Behavior of a large scale pile in silty clay

Comportement d'un pieu à grande échelle dans l'argile silteuse

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SYNOPSIS A large scale pile test was performed to obtain information for the design of an offshore platform in 700 feet (213 m) of water. The pile was tested under a variety of loading conditions. This report discusses the results of the load test program as they relate to pile foundation response under various load rates and cyclic load conditions.

INTRODUCTION

A large scale pile load test was conducted in Long Beach, California, to better determine the axial behavior of the piles designed for a 700 foot (213 m) water depth platform located 15 miles (24 km) offshore of the test site. The Long Beach location was chosen because it is onshore and has soil properties similar to the offshore site at a deep enough penetration to show significant differences between various axial pile capacity predictive methods. This test has been referred to as the Beta Pile Test.

The 30-inch (762 mm), 1.5-inch (38 mm) wall thickness test pile was lowered through a cleaned out 42-inch (1067 mm) sleeve pile which had been driven to 190 feet (58 m) and later cleaned out. The test pile was then driven with a Delmag D-62-12 diesel hammer from a penetration of 190 feet (58 m) to 263 feet (80 m) below ground level. The soil at the test penetration is a very stiff to hard silty clay with frequent sand seams below 256 feet (78 m). Details of the pile test setup, soil conditions, static pile capacity and pile capacity predictive results are discussed by Pelletier and Doyle (1982). The purpose of this report is to present further results of the load tests.

TEST PLAN AND OBJECTIVES

Three load test series were conducted. Test Series One was conducted as soon as the pile was driven to grade in order to examine the ability of pile driving analysis procedures to compute short and long term pile capacity and to acquire an additional pile capacity value to compare with effective stress analyses. After this initial test, the pile was driven to a final penetration of 264 feet (80.5 m) to remove any possible testing effects. The second and most complex load test series started 60 days after driving, when piezometers indicated that 95% of the excess pore pressures had dissipated. This series was conducted to determine: (1) long term static capacity, (2) load rate effects, (3) the influence of previous load history on capacity, (4) pile capacity under both one-way and two-way cyclic loading, and (5) tension versus compression capacity. The last test series was started 87 days after driving to determine if pile capacity regained with time following cyclic degradation.

TEST PROGRAM RESULTS

Loading was applied to the pile at the +8 foot (2.4m) elevation. Pile top loads and displacements were measured at that elevation. Pile top load was measured by three independent full strain gage bridges. Each bridge consisted of four pairs of strain gages spaced 90 degrees around the pile at the same elevation in order to cancel bending strains. Displacement at the pile top was measured by a calibrated linear slidewire which was attached to an overhead reference beam. A Microhead Level and an engineer's level were also used whenever possible to verify the displacement readings.

The pile was loaded by hydraulic jacks which were arranged so that either tension or compression loads could be incrementally or continuously applied to the pile top.

A summary of the designated test numbers and test procedure is given in Table I. The measured maximum pile top load and pile top load rate is also given for each test. In addition, pile top load versus pile top displacement is plotted in Figure 1 for each test. This seven part figure shows the load-deflection data for each day of testing. A zero displacement corresponds to the pile location at the start of each load test series.

Test Series One

The first load series started 71 minutes after driving and lasted two hours. The first test (1-1) was run to obtain the short term static capacity under both incremental and continuous loading conditions. The load rate was increased in the second test (1-2) to investigate load rate effects.

Test Series Two

The second test series was started 60 days after driving and lasted three days. Seven tests were conducted. Three load tests were performed on the first day. The first test (2-1) was similar to Test 1-1 and its purpose was to obtain the long term static capacity. Test 2-2 was run at a fast rate to investigate load rate effects while the third test (2-3A) was conducted to determine whether there was any change in capacity since Test 2-1.

Three tests (2-3B, 2-4, 2-5) were conducted on the second day. Test 2-3B was a duplication of Test 2-3A and was conducted to provide a reference capacity for the subsequent test (2-4). Test 2-4 was a cyclic

TABLE I
Test Program Load Series

Test Number	Procedure	Maximum Load, kN	Load Rate mm/min
1-1	-Load in 100 kip increments	3123	-
	-Bleed off some hydraulic pressure		
	-Load continuously to failure then unload to zero	3634	0.8
1-2	-Load continuously to failure then unload to zero	3870	1.8
2-1	-Load in 100 kip increments	10707	-
	-Load continuously to failure	11040	1.5
	-Increase load rate	11877	17.8
	-Shut in hydraulic pressure and allow pile to creep then unload to zero	10809	-
2-2	-Load continuously to failure then unload to zero	11935	17.8
2-3A	-Load continuously to failure then unload to zero	11806	7.6
2-3B	-Load continuously to failure then unload to zero	14417	1.3
2-4	-One-way cyclic tension test. Ten cycles each at percentage levels of: 25-30, 25-40, 25-50, 25-60, 25-67, 25-83, and 5 cycles at 25-100 (where 100% is the Test 2-3B capacity)	-	12.7
	-Continue last load cycle to large deflection then unload to zero	14337	12.7
2-5	-Load continuously to tension failure, then conduct 16 cycles of fully reversed (two-way) loading (see Table II)	13816	4.1
2-6	-Load continuously in compression to failure then unload to zero	-10249	15.2
	-Load continuously to failure until pile is pushed into virgin soil below tip	-11285	15.2
	-Conduct 6 cycles of fully reversed loading — cycle 1	10409 -8932	
	cycle 6	9052 -8060	
3-1	-Load continuously to failure	11596	1.3
	-Increase load rate	11730	12.7
	-Shut in hydraulic pressure and allow pile to creep then unload to zero	11846	-
3-2	-Load continuously to failure	13096	12.7
	-Conduct 9 cycles of fully reversed loading then 6 cycles of progressively less loading — cycle 1	+13096 -10578	
	cycle 9	+10542 -9595	
	-Load continuously to compression failure	-8661	12.7
3-3	-Load continuously to failure	10542	7.6

(Unless otherwise stated, all load values in this report include the pile weight and, for Test Series One, the hammer weight. Positive values denote a tension test.)

tension test where the pile was cycled at several tension bias load levels. A tension test to failure at the end of Test 2-4 was run to determine if cyclic tension loading caused any degradation in pile capacity. The final test of the day was a fully reversed (tension and compression) cyclic test. Shown in Figure 1 is the pile top-displacement relationship for cycles 1, 5, 10 and 16.

The last day consisted of a compression test and a fully reversed cyclic test. During the compression part of the fully reversed cyclic test, the displacement slidewire bottomed out at -7.8 cm. Thus, that portion of the curve in Figure 1 is not correctly shown.

Test Series Three

Test Series Three was conducted 30 days after test series two. This last series lasted three days and consisted of three tests (3-1, 3-2, 3-3).

Test 3-1 was a static test conducted for the purpose of determining whether capacity had recovered since completing Test Series Two.

Test 3-2 was a fully reversed cyclic test. After nine fully reversed load cycles to failure, the level of loading was progressively reduced over the next six cycles in order to develop a backbone curve. A compression test immediately followed to determine if degradation continued during the reduced cyclic tests.

The last test (3-3) of the program was a static test to failure.

DISCUSSION

The testing program was developed to provide information about static and cyclic capacity, load history and rate effects. These are discussed below.

Static pile capacity

The static capacity was determined in Test 2-1 (Figure 1). A variety of load application methods was used during Test 2-1. The pile was initially loaded in tension in 100 kip (445 kN) increments. At a pile top load of 2407 kips (10707 kN), the pile began to pull out of the ground. A slow continuous load was then applied at a rate of 0.06 inches/minute (1.5 mm/minute). The pile reached a maximum load of 2482 kips (11040 kN). This load, minus the pile weight of 144 kips (640 kN), has been taken by Pelletier and Doyle (1982) as the static pile capacity. The loading rate was then increased to 0.7 inches/minute (17.8 mm/minute) and the load reached 2670 kips (11877 kN). The hydraulic jacks were then shut off and the pile was allowed to creep. The pile load dropped to 2430 kips (11809 kN) before creep movement ceased. For this test, the difference between incremental capacity, slow continuous capacity and post-failure creep capacity was less than 3%.

Short-term versus long-term capacity

The first load test (Figure 1) started 71 minutes after driving. The maximum load measured during the continuous portion of the test was 760 kips (3381 kN). Subtracting the pile weight of 144 kips (641 kN) and weight of hammer of 33 kips (147 kN) gave a net pile load of 583 kips (2593 kN). A piezometer located at a penetration of 239 feet (73 m) and 17.8 feet (5.4 m) from the pile wall showed an excess pore pressure at the time of testing of 30 psi (207 kPa) above hydrostatic of 93 psi (641 kPa). The test capacity for the first test (2-1) after 60 days showed a net pile top load of 2338 kips (10400 kN). Thus, the ratio between the long-term and short-term capacity (set up factor) is four. It is likely that the "immediate" capacity would have been less than 583 kips (2593 kN) if the pile was tested sooner than 71 minutes after driving since some pore pressure dissipation was observed before the test started.

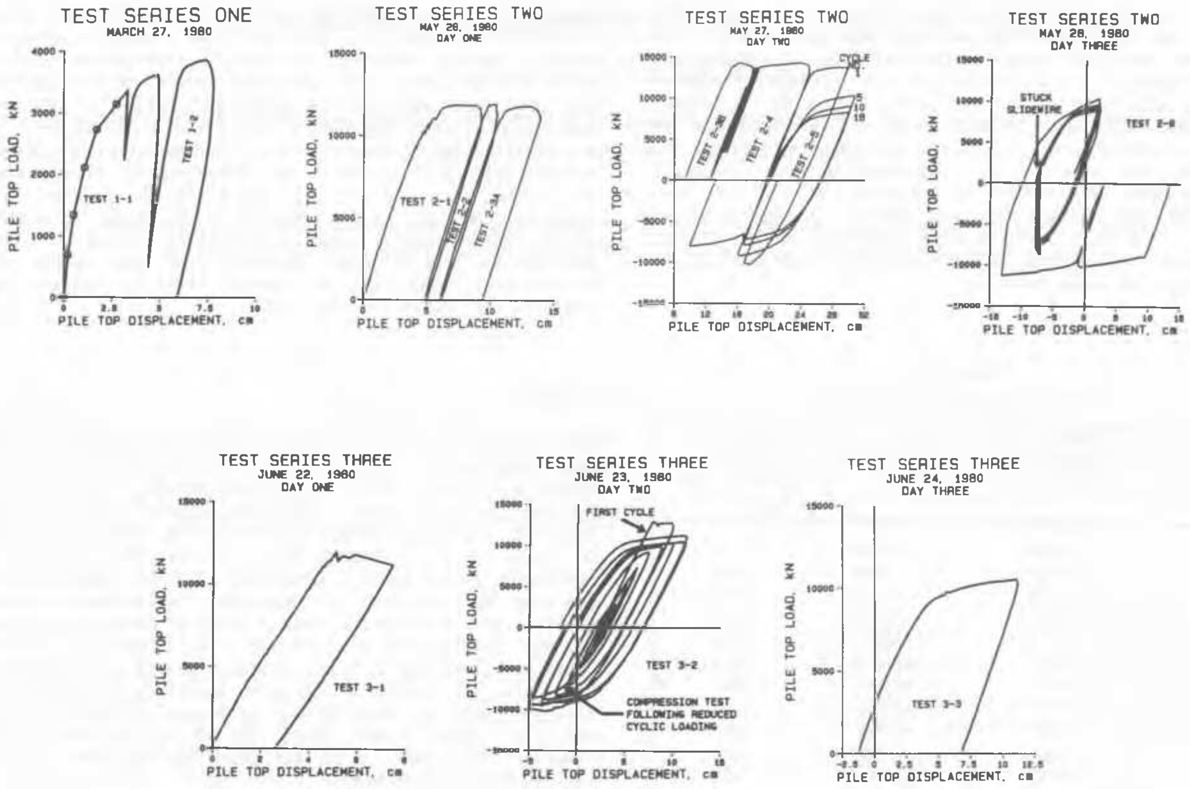

Fig. 1 - Pile top load versus pile top displacement diagrams.

Rate effects

The pile was tested under a variety of continuously applied loading rates which varied from 0.03 to 0.7 inches/minute (0.76 to 17.8 mm/minute). In terms of load rates, these correspond to 100% to 1500% of ultimate capacity per hour in Test Series Two, for example. The loading rates are indicated in Table I. Pile capacity increased approximately 8% per tenfold increase in strain rate in Test Series Two, while the capacity increased by 8% when the strain rate was doubled in Test Series One. The large increase in Test Series One may have been affected by the dissipation of pile driving induced pore pressures along the pile wall during testing.

Load history effects

One of the objectives of Test Series Two was to determine set up capacity and to investigate effects of load history on measured pile capacity. As shown in Table I, the fast rate portion of Test 2-1 and Test 2-3A had essentially the same capacity. These tests were conducted on day one. Thus, the accumulation of large displacements from Test 2-1 through Test 2-3a did not affect capacity. The next morning (day two), the first test (2-3B) showed a 32% increase in capacity as compared to the first test on day one (Test 2-1). Tests 2-1 and 2-3B were run at similar load rates. Even after one-way cycling (Test 2-4), the capacity was unchanged from that mornings's first test. The overnight increase in capacity may be due to dissipation of shear induced pore pressures which were generated during the previous day's testing. For example, during Test Series Two, near field excess pore pressures showed a gradual rise of 25 psi (172 kPa) during the first day's testing. These had dissipated by the next morning.

Test Series Three was conducted in a similar manner as Test Series Two except that the pile loading rates were 0.5 inches/minute (12.7 mm/minute) for each load test. The load test results were similar to Test Series Two except the effect was smaller. Pore pressures rose 10 psi (69 kPa) during the first day of testing, while the overnight capacity increased 9%.

Cyclic pile capacity

Tests were conducted under both one-way and two-way loading conditions. For the one-way tests (Test 2-4), cyclic load was applied ten times each between levels of 25-30%, 25-40%, 25-60%, 25-67%, and 25-83% of the ultimate capacity. No permanent pile deformations occurred until the pile was cycled five times between 25-100% of ultimate capacity. The measured pile top capacity prior to the one-way cycling was 3241 kips (14417 kN) as indicated in Table I, Test 2-3B. Following cycling at ultimate capacity, the measured pile top load of 3232 kips (14377 kN) was essentially unchanged. Because of the limited number of cycles, however, no criteria were developed to predict the response of deep penetration piles under a large number of cycles.

Displacement controlled two-way cyclic tests to failure were conducted. Tests 2-4, 2-5 and 3-2 consisted of 16, 7 and 9 fully-reversed cycles, respectively. The cycling at ultimate capacity resulted in a significant reduction in pile capacity. Degradation began as soon as two-way loading occurred. Degraded values ranged between 61% to 85% of the precyclic values. The measured pile top loads for Test 2-5 are shown in Table II. By the last three cycles, the degraded capacity was 61% of the original capacity and the rate of degradation had decreased. The reduction in ultimate tension capacity approximates a straight line on a semi-logarithmic plot for cycles 2-14. This behavior is similar to the degradation model proposed by Idriss, et al (1976) for clay. The degradation was, however, temporary and had healed by the end of the 30 days separating Test Series Two and Three, as indicated by the first load test results from each test series (Table I). Thus, for these soils, capacity degradation is substantial but temporary.

TABLE II
Pile Top Loads For Test 2-5

Cycle Number	Peak Tension (kN)	Peak Compression (kN)	Half Of Absolute Difference (kN)
1	13816	-10338	1739
2	11921	-10449	743
3	11338	-10133	605
4	10818	-9693	565
5	10382	-9274	556
6	10035	-8928	556
7	9755	-8821	467
8	9635	-8394	623
9	9323	-8105	609
10	9128	-7887	623
11	8928	-7762	583
12	8776	-7562	609
13	8630	-7326	654
14	8460	-7388	538
15	8478	-7322	578
16	8429	-7162	636

After cycle nine of Test 3-2, two-way cycling was continued except that maximum tension and compression loads were progressively reduced. One of the purposes for running this test was to determine if degradation continued under reduced two-way cycling. The loads were reduced to .93, .88, .75, .52 and .27 of the last full cycle failure load. The pile was then loaded to failure in compression. If no degradation had occurred under the reduced cycling levels, the load-deflection curve obtained from the last compression test should have gone through the tips of the previously run reduced cycling curves. As shown in Figure 1 the reload went through the tips of the .27 and .52 load level curves but was progressively less at the higher load levels. At the .93 load level, the load was about 8% less on the reload cycle. About one-third of that was caused by the continued degradation of the last reload. The rest of the degradation was, therefore, caused by the reduced load levels. It appears that load levels less than about 50% of the maximum did not cause further reduction in the load-displacement curve. Above that level, the three reduced level cycles contributed to the degradation at about the same level as the fully reversed failure load cycles.

Tension versus compression capacity

The two-way cyclic tests provide a means to examine the question of tension versus compression capacity of large scale piles. Since the pile tip had been pulled up by previous tension loadings, pile tip effects were minimal. If tension and compression capacities are the same, then the midpoint of a symmetric load-displacement hysteresis loop would be biased by the weight of the pile and the plug (if the pile was plugged during the test). During driving, the plug was monitored and the pile did not plug. The measured weight of the pile was 144 kips (640 kN) and the submerged weight of the plug was about 30 kips (133 kN). The results of Test 2-5 are shown in Table II where half of the absolute difference between the peak tension and compression values after the first few cycles is about equal to the pile weight. During cyclic loading, the plug apparently played no measureable part in the pile capacity. Thus, the two-way cyclic data indicate that the tension and compression capacities are equal. Similar results were obtained for Tests 2-6 and 3-2.

SUMMARY

The results of Beta Pile Test contribute to the understanding of the behavior of axially loaded piles under a variety of loading conditions. Pile set up factors are at least four for these soils. Pile capacity increased about 8% for every tenfold increase in load rate. During Test Series Two and Three, overnight pile capacity increases of 9-32% were measured and may be explained by measured dissipation of shear induced pore pressures. For a limited number of cycles, one-way loading did not degrade pile capacity. Two-way loading at ultimate load capacity caused a significant but temporary reduction in pile capacity. After fully reversed cycling, degradation continued to occur at two-way load levels above about 50% of the ultimate load. Finally, the capacity of the pile was the same in both tension and compression.

ACKNOWLEDGEMENTS

This study, conducted by Shell Development Company, Fugro-Gulf, Inc. and Raymond International, was funded by Shell Oil Company and its partners in the Beta Oil Field. Special thanks are due Messrs. T. K. Hamilton, D. W. Bogard, R. L. Boggess and H. Matlock for their design and installation efforts during the pile test.

REFERENCES

Pelletier, J. H. and Doyle, E. H. (1982). Tension capacity in silty clays - Beta pile test. Proc. 2nd Int. Conf. Numerical Methods in Offshore Piling, 163-182, Austin, Texas.

Idriss, I. M., Dobry, R., Doyle E. H. and Singh, R. D. (1976). Behavior of soft clays under earthquake loading conditions. Preprints, Offshore Technology Conf, Paper 2671, 605-616, Houston, Texas.