LATERAL LOAD TESTS ON PRESTRESSED PILES

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SUMMARY

In this paper lateral load tests on prestressed concrete piles are described. The tests were performed in order to determine those features of the design, detailing and construction of such piles that would lead to the ductile behavior desirable under seismic conditions. The piles were tested in three configurations:- as a simple beam subjected simultaneously to lateral and axial load; as a cantilever projecting from a block of concrete simulating a pile cap and subjected to axial and lateral load; and as a concentrically loaded column stub. Eight parameters were studied, and in each test one was varied from the reference conditions.

INTRODUCTION

Tentative Provisions for the Development of Seismic Regulations for Buildings, ATC 3-06, (Ref. 1) were issued in the USA in 1978. That comprehensive document represented a concensus, current state-of-knowledge in the fields of engineering seismology and engineering practice as it pertained to seismic design and construction of buildings. However, the subsequent industry review (Ref. 2) showed that those provisions dealt inadequately with precast/prestressed concrete construction in general and with precast/prestressed concrete piles in particular. The drafters of ATC 3-06 in reality prohibited the use of precast/prestressed piles in regions of high seismicity by requiring that they not be used to resist flexure caused by earthquake motions unless it could be shown that the pile was stressed to below the elastic limit under the maximum soil deformation that would occur during an earthquake. Their concern was based on the relatively poor behavior of some-prestressed piles in the 1971 San Fernando (Ref. 3) and in the 1978 Miyagi-ken-oki (Ref. 4) earthquakes.

Precast prestressed concrete piles have been widely used in the USA even in seismic regions, and in the vast majority of instances they have proven effective and economical, thus the virtual ban imposed by ATC on their use in severe seismic environments reflects more the present lack of knowledge than any weakness inherent in the concept of such piles. However it is understandable in the light of the grave consequences of foundation failure and the difficulties associated with repairs.

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Gravity loads cause primarily axial loads in piles, whereas earthquake motions impart shear and bending moments as well. In any soil mass, the basic bedrock seismic motion may be amplified as it propagates to the ground surface, and three situations arise in which this causes severe lateral effects on the piles. In the first, a radical difference in the stiffness of adjacent soil layers may induce reversed curvature (see Fig. 1a). Secondly, if the pile cap is not embedded into the ground the head of the pile may be subjected to large shears and local bending moments (Fig. 1b). Thirdly, free-standing (marine) piles have two regions where high moment, axial force and shear may all act together. In the latter case the potential exists for collapse through instability, initiated by a loss of stiffness in the critical regions. This may be exacerbated if the ground into which the piles are driven undergoes some liquefaction, causing the effective length of the free-standing part to increase.

The lateral motion of the piles and the precise shape into which they deform are very difficult to obtain. Sogge (Ref. 5) and Davisson (Ref. 6) have analysed response to static loads assuming that behavior may be described by simple linearly elastic models. The best available knowledge (Ref. 3) suggests that piles generally move with the soil mass, although deviations from this behavior must occur locally (e.g. Fig. 1a). Analysis of a number of specific sites in the San Francisco Bay Area (Refs. 3 and 7) suggests that curvatures of 2 to 4 x 10^{-4} radians/in. may be expected in a magnitude 7 earthquake, and up to 8 x 10^{-4} in a strong (magnitude 8.25) event. The values derived in such analyses are sensitive to the mathematical modelling, especially that of the interaction between pile and soil in the critical regions, but they consititute the best basis presently available for design of the piles.

Recent tests by Park (Ref. 8) and earlier ones documented by Sheppard (Ref. 9) on 16"-18" dia. specimens show that piles which contain only light spiral reinforcement (approx. 0.2% volumetric ratio) could sustain only curvatures of 2 to 3 x 10^{-4} radians/inch before catastrophic brittle failure, whereas piles reinforced with heavy spirals (approx. 2 to 2½%) designed according to the New Zealand Code (Ref. 10) displayed curvature ductility many times greater than the maximum neccesary 8 x 10^{-4} radians/inch. The New Zealand Code requires a volumetric ratio

$$\rho_{sp} \geq 0.45\left(\frac{A_g}{A_c} - 1\right)\frac{f'_c}{f_y}\left(0.5 + 1.25\,\frac{P_e}{f'_c A_g}\right) \qquad (1)$$

seeking to replace the axial strength lost on spalling of the cover by increased core strength due to confinement. The 1978 Model CEB/FIP Code recommended

$$\frac{A_{sp}}{s} \geq \left(c + 7d_b\right)\frac{f_r}{f_y} \qquad (2)$$

where c is the cover, d_b is the spiral wire diameter, s its pitch and f_r is the modulus of rupture of the concrete. The volumetric ratio was also to be

$$\rho_{sp} \geq 0.007 \tag{3}$$

Eq. (2) appears to be designed to prevent an explosive bursting failure when longitudinal cracks destroy the tensile hoop strength of the concrete shell, assuming that such cracks penetrate 7 bar diameters inside the cover. Eq. (2) and (3) were recommended by the ATC 3-06 review committee for piles which were free-standing, hollow core, or subject to severe operating conditions.

In addition to spiral reinforcement other factors have been suggested as leading to higher curvature capacities. Among them are: Using a greater number of slimmer piles, reducing the axial load below the usual rated level, using a higher prestress level, using stronger concrete, incorporating non-prestressed longitudinal steel, and reducing the concrete cover. Furthermore, simple analyses suggested that hollow piles might present special problems which could not be cured by adding spiral if the concrete inside the spiral were to fail by spalling inwards, in which case the spiral would not be efffective in providing confinement.

A program of testing was conducted at the University of Washington to investigate which of the foregoing parameters exerted a significant influence on pile behavior.

TEST PROGRAM

In 1981 the National Science Foundation initiated a project at the University of Washington under the direction of Professor Stanton on the "Performance of Prestressed Piles Under Earthquake Loads." That project is a cooperative effort of the University and Concrete Technology Corporation of Tacoma, Washington, the major Puget Sound Region prefabricator and precaster. To date data are available from monotonic and reversed cyclic loading tests on 14 piles (Ref. 11). Cross-sectional details for a typical pile are shown in Fig. 2 and properties of the 12 specimens discussed in this paper are shown in Table 1. The test piles were manufactured in Concrete Technology's plant using their standard procedures and equipment.

The 14 inch diameter octagonal piles had an effective prestress level of 750 psi provided by six half-inch diameter 270 ksi strands. Concrete strengths at testing ranged from 5,560 psi to 7,710 psi, covers over the spiral were 1.00 or 2.00 inches, additional axial loads applied to the pile ranged from 182 to 364 kips (1,469 to 2,312 psi), and additional non-prestressed reinforcement consisting of either 6 No. 4 or 6 No. 8 Grade 60 bars was used in two specimens. The spiral reinforcement consisted of W3.5 or W5.5 Grade 65 wires (0.211 or 0.264 in.) at pitches varying between 3 and 8 inches so that the spiral reinforcement ratios varied between 0.18% and 0.73%. Most piles contained 2-1/2 in. diameter central holes but two contained 7 in. diameter central holes intended to simulate at reduced scale the 24" hollow octagonal piles popular for larger capacities. In one pile, No. 4, the 2-1/2 in. hole was central at the pile ends but at midspan was accidentally displaced 2 in. transverse to the axis of bending in a plane perpendicular to the plane of bending.

Specimens were tested in a horizontal plane using the setup shown in Fig. 2. The pile was placed within a self-reacting frame and loaded as a simple beam of 11'-6" span. It was subjected to two-point loading at locations spaced 1'-3" either side of its midspan. The axial load was simulated by unbonded, post-tensioned strands placed in the pile's central hole, thus, P-delta effects due to the axial load were minimized. Measurements were made of the transverse loads applied to the pile, strains at the 1/4 points and a deflection profile along the pile. The axial load was continuously monitored using a load cell at each end of the tendon, and was kept constant at its prescribed value. The lateral load was monotonic on specimen 5 and reversed cyclic on all the others, following the sequence shown in Fig. 4. Predetermined deformations rather than loads were applied, based on the maximum pile deflection. These were obtained from the curvatures of Fig. 4 assuming linearly elastic behavior. Thus, in the nonlinear response range, the maximum curvature at midspan was greater than the nominal value, while the values in the end regions were concomitantly less. The procedure was adopted to simplify control.

After these pile-body tests had been completed, 10 of the broken pieces were turned round and cast in pairs into blocks of concrete simulating pile caps (Fig. 5). The ends of the piles had been equipped with ducts for dowels or the strand had been left projecting by 5', in order to permit different embedment conditions to be studied. The damaged end of each specimen (which had formed the midspan region during the pile body tests) was cut off clean and square with a concrete saw to permit axial loading by post-tensioning as before. Each cantilever specimen projecting from the pile cap was tested independently. This procedure allowed good use to be made of the available specimens and permitted comparison of the performance of the same physical specimen in two test configurations. The load on specimen 04 (west end) was montonic while the others were reversed cyclic following the aforementioned deformation sequence. Finally 28" x 14" octagonal stub columns were sawn from those specimens which permitted it and they were tested under concentric axial load. This was done in order to obtain information about the strength and ductility under axial load alone which could be compared with data from the combined flexure and axial load tests.

RESULTS AND DISCUSSION

Tables 1-3 provide a summary of the numerical test results. Of the pile body tests it can be seen that none of the specimens exhibited behavior appropriate for use in extreme seismic environments. Scatter in the results is evident from piles 1A and 03, which were nominally identical, but displayed markedly different results.

In all cases cracking was observed first, followed in a subsequent cycle by some spalling on the compression side, and in specimens other than 3, 4, 5 and 9, failure occurred on the way to the first loading peak of cycle 6 (to a maximum nominal curvature of 4×10^{-4} rad/in.). The maximum curvature in the central region was computed from the central deflection and 1/4 point curvature (obtained from strain gages), assuming a linear variation of curvature outside the central region. The failure was in all cases sudden and explosive, and was caused by bursting of the spiral in all specimens but No. 13. In the solid piles the broken ends were conical and the pretensioned strands buckled symmetrically, in a manner suggestive of a pure compression failure. Failure always occurred in the central region, probably because compression at the load point provided some local confinement.

The values of maximum load and curvature were hard to obtain because discrete rather than continuous readings were taken. However, the absolute peaks are of less interest than the maximum values at which reasonably stable hysteresis loops were possible. The observed cracking moments and curvatures were (but for specimen 12) consistently higher than their theoretical counterparts, but this may be attributed in part to difficulties in observing the first appearance of a crack. The results of Table 1 suggest that reduced axial load (specimen 04) and increased spiral (Spec. 09) are the only two factors which permitted the specimen to withstand more than the median level of 6-1/4 cycles. (The first 4 specimens were tested at the Concrete Technology laboratory, and it is believed that their apparently superior performance may be due in part to slight variations in test procedure.) Specimen 08 (light spiral) withstood the smallest number of load cycles. Addition of longitudinal reinforcing bars increased the initial (elastic) stiffness, but contributed negligible extra ductility or energy dissipation. If the spiral had been stronger and its bursting delayed, then the bars might have been more effective. The hollow pile (13) with the heavy spiral recommended by CEB/FIP and the ATC review committee failed without bursting of the spiral. After the cover started to spall, the increase in stress on the remaining concrete inside the spiral was great enough to cause a failure by spalling inwards. Specimen 12 failed by bursting outwards, suggesting that a threshold level of spiral exists beyond which addition of more steel is useless.

The pile cap specimens were all much more ductile then the pile bodies (see Table 2), which can only be explained by the additional confinement afforded by the pile cap itself. The monotonic test (04 W) also had a light axial load and failed in diagonal tension. The others all failed in combined flexure and axial load and displayed the same conical failure surface as did the pile bodies. All piles were embedded 21" (1-1/2 pile diameters) into the cap, and no pull-out was observed. It was not expected in view of the axial compression. Once again the specimens with light axial load and heavy spiral performed the best.

The column stub test results showed considerable scatter, partly due to the failure type. Specimen 09 (heavy spiral) failed prematurely by debonding of the spiral at the ends, while 12 E (hollow) failed by shearing on a diagonal plane. The others failed by bursting of the spiral.

CONCLUSIONS

The tests showed that presently-used spirals are not adequate to ensure ductile behavior. Four more piles with heavier spirals were cast and are now being tested. Until results are available the following are recommended: (1) in critical regions spiral design should be based on the ACI 318-77 requirements for columns (Eq. 10-5), (2) in critical regions the central void in hollow piles should be filled with concrete, and (3) research is needed to define with greater certainty the maximum pile curvatures which must be accommodated.

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TABLE 1.

SPECIMEN NO. (TEST NO.)	f'c (psi)	AXIAL LOAD (kips)	NONPRESTRESSED LONGITUDINAL REINFORCEMENT	TRANSVERSE REINFORCEMENT	VOLUMETRIC SPIRAL RATIO (%)	CONCRETE COVER (in)	THEORETICAL M_{cr} (kip-in)	ACTUAL M_{cr} (kip-in)	MAXIMUM M (kip-in)	ACTUAL K_{cr} (10^{-6} rad/in)	MAXIMUM K (10^{-6} rad/in)	CYCLES TO FAILURE
01A (REF1)	6665	309		W3.5 @ 4"	.35	2.0	1079	1350	1650	183	260	6.50
03 (REF2)	6205	283		W3.5 @ 4"	.35	2.0	1021	1000	1640	185	640	8.50
04 (A1)	6245	241		W3.5 @ 4"	.35	2.0	939	1350	1735	255	710	9.00
05 (L1)	6440	288		W3.5 @ 4"	.35	2.0	1059	1200	1750	115	650	0.25
06 (R1)	6147	287	6 #4	W3.5 @ 4"	.35	2.0	1059	1070	1643	141	340	6.25
07 (R2)(2)	7013	327	6 #8	W3.5 @ 4"	.35	2.0	1262	1425	1800	202	450	6.25
08 (S1)	5849	269		W3.5 @ 8"	.18	2.0	983	1250	1610	164	330	4.75
09 (S2)	6008	282		W5.5 @ 3"	.73	2.0	1014	1100	1525	145	470	6.75
10 (C1)	7367	347		W3.5 @ 4"	.29	1.0	1165	1100	1743	98	310	6.25
11 (F1)	7709	364		W3.5 @ 4"	.35	2.0	1205	1350	1907	150	410	6.25
12 (G1 (1)	6957	235		W3.5 @ 4"	.35	2.0	1008	800	1702	149	365	6.25
13 (G2)(1)	5560	182		W5.5 @ 3"	.73	2.0	858	882	1250	118	340	6.25

(1) 7 in. dia. central hole   (2) 2.5 in. hole eccentric by 2 in. at mid span

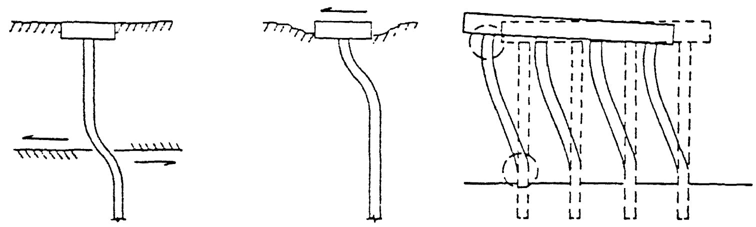
TABLE 2

SPECIMEN NO. (TEST NO.)	f'c (psi)	AXIAL LOAD (kips)	TRANSVERSE REINFORCEMENT	EMBEDMENT DETAILS	THEORETICAL M_{cr} (kip-in)	ACTUAL M_{cr} (kip-in)	MAXIMUM M (kip-in)	ACTUAL K_{cr} (10^{-6} rad/in)	MAXIMUM K (10^{-6} rad/in)	CYCLES TO FAILURE
01AE (REF1)	7280	328	W3.5 @ 4"	6 #6 bars 36" long	1221	1796	2300	157	730	21.25
01AW (REF2)	7060	318	W3.5 @ 4"	6 #6 bars 36" long	1196	1957	2380	309	1000	21.25
03E (E2)	6600	316	W3.5 @ 4"	6 #8 bars 54" long	1228	2202	2050	337	1450	25.25
03W (E1)	7040	316	W3.5 @ 4"	nothing	1100	1468	2250	121	1495	25.25
04E (A1)	6245	241	W3.5 @ 4"	5' strand extension	1017	1613	2180	203	1950	10.25
04W (L1)	6245	241	W3.5 @ 4"	5' strand extension	886	1100	2200	300	1100	0.25
05E (REF3)	7320	316	W3.5 @ 4"	6 #6 bars 36" long	1197	1896	2340	163	1450	25.25
05W (E3)	8040	314	W3.5 @ 4"	5' strand extension	1113	1712	2190	173	1030	23.25
08E (S1)	6376	291	W3.5 @ 8"	6 #6 bars 36" long	1125	1341	1850	182	700	12.25
09E (S2)	6008	278	W5.5 @ 3"	6 #6 bars 36" long	1090	1366	1600	221	2100	13.75

TABLE 3

SPECIMEN NO. (TEST NO.)	SPECIMEN GEOMETRY	TRANSVERSE REINFORCEMENT	MAXIMUM STRESS (ksi)	AVERAGE STRAIN ST MAXIMUM STRESS (in/in)
04E (REF1)	14" octagon	W3.5 @ 4"	7.36	0.0055
04W (REF2)	14" octagon	W3.5 @ 4"	5.65	0.0038
08E (S1)	14' octagon	W3.5 @ 8"	5.78	0.0041
09E (S2)	14" octagon	W5.5 @ 3"	4.72	0.0042
13EL (G1)	hollow core	W3.5 @ 4"	4.91	0.0039
13E2 (G2)	hollow core	W5.5 @ 3"	5.04	0.0046
12E (G3)	hollow core (1)	W3.5 @ 4"	3.74	0.0048

(1) Hollow core offset from member's longitudinal axis



(a) (b) (c)

Fig. 1. Critical Locations in Laterally Loaded Piles.

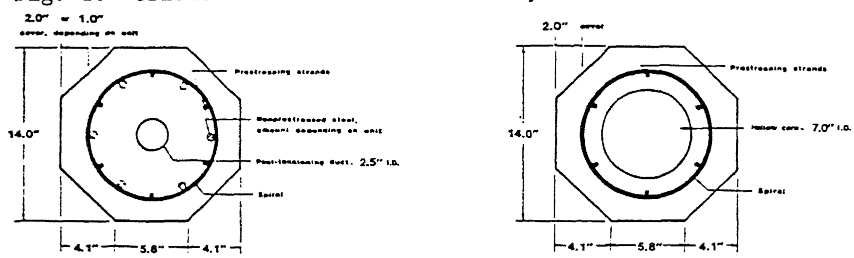


Fig. 2. Cross-sections Used in Tests.

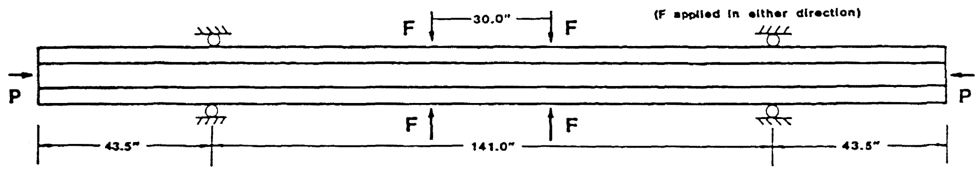
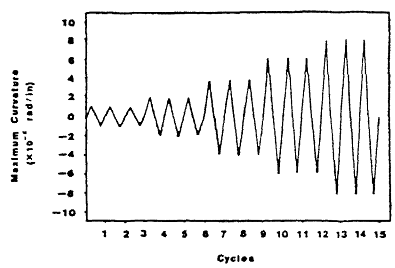


Fig. 3. Pile Body Test Set Up.

Fig. 4. Load Sequence.



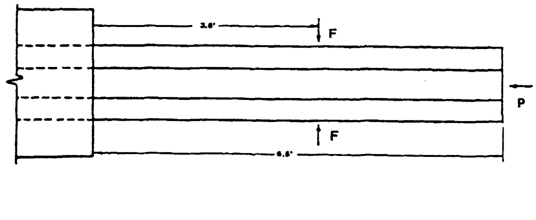


Fig. 5. Pile Cap Test Set Up.