Effect of Pile Embedment on the Development Length of Prestressing Strands

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Nineteen 14 in. (356 mm) square prestressed concrete piles were tested to study the effect of pile embedment on the development length of 1/2 in. (13 mm) diameter prestressing strands. The primary objective of this investigation was to determine the required embedment length of strands in piles, so that the ultimate flexural moment can be developed without strand end slippage. In this study, equations for development length in the AASHTO Specifications and ACI Code were evaluated. Also examined were the general bond slip and the maximum average bond stress at failure.

Questions regarding the development length of prestressing strands and the validity of AASHTO Eq. (9-32)[1] have been raised, based on a study conducted by Cousins, Johnston and Zia.[2,3] The findings of this investigation resulted in the Federal Highway Administration (FHWA) requiring an application of a 1.6 multiplier to AASHTO Eq. (9-32).[3] The multiplier is used for strands with diameters up to and including 9/16 in. (14 mm), and is applicable to any prestressed concrete member.

It is questionable whether the use of a multiplier is necessary in the design of piles. The design requirements for a pile are substantially different from those of a beam in a structure. The pile is required to resist ship impact forces while impact would be applied only a few times in the life of the pile.

The ship impact forces used in design are somewhat arbitrary and the probability of such an event occurring is low. Also, it must be recognized that the design criteria developed for ship impact takes into account that, although the structure would suffer some damage, catastrophic collapse is not likely to occur.

Also, pile embedment into the pile cap presents a much different end condition than that encountered by a superstructure supported on bearings. In the case of the pile, shrinkage of the confining concrete in the pile cap creates a clamping force that serves to reduce the development length.

Furthermore, a prying action results when a moment is applied at the interface of the pile and the pile cap, thereby increasing the contact pressure at the junction of the pile face and the cap. The resulting increased pressure on the pile further reduces the development length. For the majority of designs, piles are embedded 12 in. (305 mm) into the pile cap and the usual design method assumes zero moment transfer (i.e., a pinned connection).

Based on the above-mentioned considerations, it is possible that the AASHTO Eq. (9-32) might, in fact, be conservative for piles which are properly embedded in a pier cap and designed to resist bending and shear forces due to ship impact.

This paper presents the results of an experimental investigation to study the effect of pile embedment on the development length of prestressing strands.

The objectives of this study were to identify the optimum embedment length (development length) required to develop the ultimate flexural strength of a pile without any strand slip and to evaluate the development length by the current ACI Code[4] and AASHTO equations.[1]

TRANSFER AND DEVELOPMENT LENGTH OF PRESTRESSING STRANDS

In pretensioned concrete, the total prestressing force is transferred to the concrete entirely by the bonding of the prestressing strand to the concrete surrounding it. When a pretensioned beam is subjected to shear, additional bond stresses are developed.

To prevent failure, it is necessary to calculate the level of bond stress due to loading and other effects. The tendency for the strand to slip is resisted by a combination of adhesion, friction and the Poisson effect or lateral swelling of steel in the transfer zone.

These factors provide the mechanism for the transfer of the prestressing force to the concrete upon release of the strand. The length over which the initial prestressing force is transferred to the concrete is termed the "transfer bond length."

Another type of bond mechanism, termed "flexural bond," is mobilized when the member is subjected to bending as a result of externally applied loads. As these loads increase, the stress in the strand also increases. The additional length over which the resulting increase in strand force is transferred is known as the "flexural bond length."

The sum of the transfer length and flexural bond length at the full flexural capacity of a member is termed the "development length." If inadequate development length is provided, ultimate strength will be governed by bond rather than by flexure. A direct result of inadequate development will be a premature failure.

Early investigations on the mechanism of bond were conducted in the 1950s.[5,6,7] These tests concluded that the strand diameter, the method of releasing the strand and the physical condition of the strand are all parameters that influence the development length.

Tests by Hanson and Kaar[5] were performed on specimens prestressed with clean ¼, ⅜ and ½ in. (6.3, 9.5 and 13 mm) diameter strands, and having a wide range of steel percentages. The strands were slowly released, instead of being suddenly released by flame cutting.

In most of the specimens, there were significant increases in the load carrying capacity between the point at which first bond slip was detected and final bond failure. The difference in load carrying capacity was believed to be due to mechanical interlock of the strand. The ACI Code equation approximates the average value of all the points representing first bond slip and final bond failure.

Results of tests performed by Kaar, LaFraugh and Mass[8] indicated that, although higher strength concrete could develop 75 to 80 percent of the transfer bond in a shorter distance than lower strength concrete, the total distance required to develop 100 percent of the transfer bond was approximately the same regardless of concrete strength.

Martin and Scott[9] proposed a transfer length of 80 diameters for strands of all sizes, and a flexural bond length of 160, 187 and 200 diameters for the ¼, ⅜ and ½ in. (6.3, 9.5 and 13 mm) diameter strands, respectively. These values are considerably higher than those specified by the current ACI Code and AASHTO Specifications.

On the other hand, based on the results of a test program of 36 pretensioned hollow-core units, Anderson and Anderson[10] concluded that the current ACI Code requirement on the development length is adequate.

In a comprehensive study to critically review past research data published by other investigators, Zia and Mostafa[11] proposed the following expressions for transfer and development length:

$$l_t = (1.5\, f_{si}\, d_b / f'_{ci}) - 4.6 \tag{1}$$

$$l_b = 1.25\,(f_{ps} - f_{se})\, d_b \tag{2}$$

$$l_d = l_t + l_b \tag{3}$$

where

f_{si} = stress in prestressing steel at transfer, ksi
f'_{ci} = compressive strength of concrete at time of initial prestress, ksi
f_{se} = effective stress in prestressed reinforcement (after allowance for all losses), ksi
f_{ps} = stress in prestressed reinforcement at nominal strength, ksi
l_t = transfer length of prestressing strand, in.
l_b = flexural bond length of prestressing strand, in.
l_d = development length of prestressing strand, in.
d_b = nominal strand diameter

Eq. (2) is based on the theoretically derived expression:

$$(f_{ps} - f_{se})\, d_b l_b = 4u_{ave} \tag{4}$$

where u_{ave} is the average bond stress within a length l_b.

Note that in the current ACI Code, it is implied that u_{ave} = 250 psi (1.7 MPa). Eq. (2) assumes a value of u_{ave} = 200 psi (1.38 MPa).

The Zia-Mostafa equation for transfer length is applicable for concrete strengths ranging from 2000 to 8000 psi (14 to 55 MPa). It accounts for effects of strand size, the initial prestress and the concrete strength at transfer. The equation for transfer length gives comparable results to those specified in the ACI Code, particularly for cases where the concrete strength at transfer is low.

The current ACI provisions for development length of prestressing strand are

Table 1. Details of test program.

Specimen number	Number and size of strands (in.)	Section	Embedment length (in.)	Concrete strength (ksi)
A-1E	8 – ½	End	36	7.10
A-2E	8 – ½	End	36	5.84
A-3I	8 – ½	Interior	36	6.59
A-4I	8 – ½	Interior	36	5.60
B-1E	8 – ½	End	42	6.70
B-2E	8 – ½	End	42	6.45
B-3E	8 – ½	End	42	5.98
B-4E	8 – ½	End	42	7.80
B-5E	8 – ½	End	42	6.48
B-6I	8 – ½	Interior	42	6.48
C-1E	8 – ½	End	48	6.96
C-2E	8 – ½	End	48	6.50
C-3I	8 – ½	Interior	48	7.76
C-4I	8 – ½	Interior	48	6.50
C-5I	8 – ½	Interior	48	6.50
C-6E	8 – ½	End	48	6.50
D-1E	8 – ½	End	60	7.20
D-2I	8 – ½	Interior	60	6.50
D-3E	8 – ½	End	60	6.50

Metric (SI) conversion factors: 1 in. = 25.4 mm; 1 ksi = 6.895 MPa.

contained in Section 12.9 of ACI 318-89. Section 12.9.1 states that three or seven-wire pretensioning strand shall be bonded beyond the critical section for a development length not less than:

$$l_d = [f_{ps} - (2/3)\, f_{se}]\, d_b \tag{5}$$

In Eq. (5), the expression in parentheses is used as a constant without units.

Where bonding of a strand does not extend to the end of a member, the development length specified in Section 12.9.1 shall be doubled.

The equation for the development length can be rewritten as follows:

$$l_d = (f_{se}/3)d_b + (f_{ps} - f_{se})d_b\, /4\, u_{ave} \tag{6}$$

where l_d and d_b are in inches, and f_{ps} and f_{se} are in kips per sq in.

The first and second terms in this equation represent transfer length and flexural bond length, respectively.

The effective steel stress, f_{se},

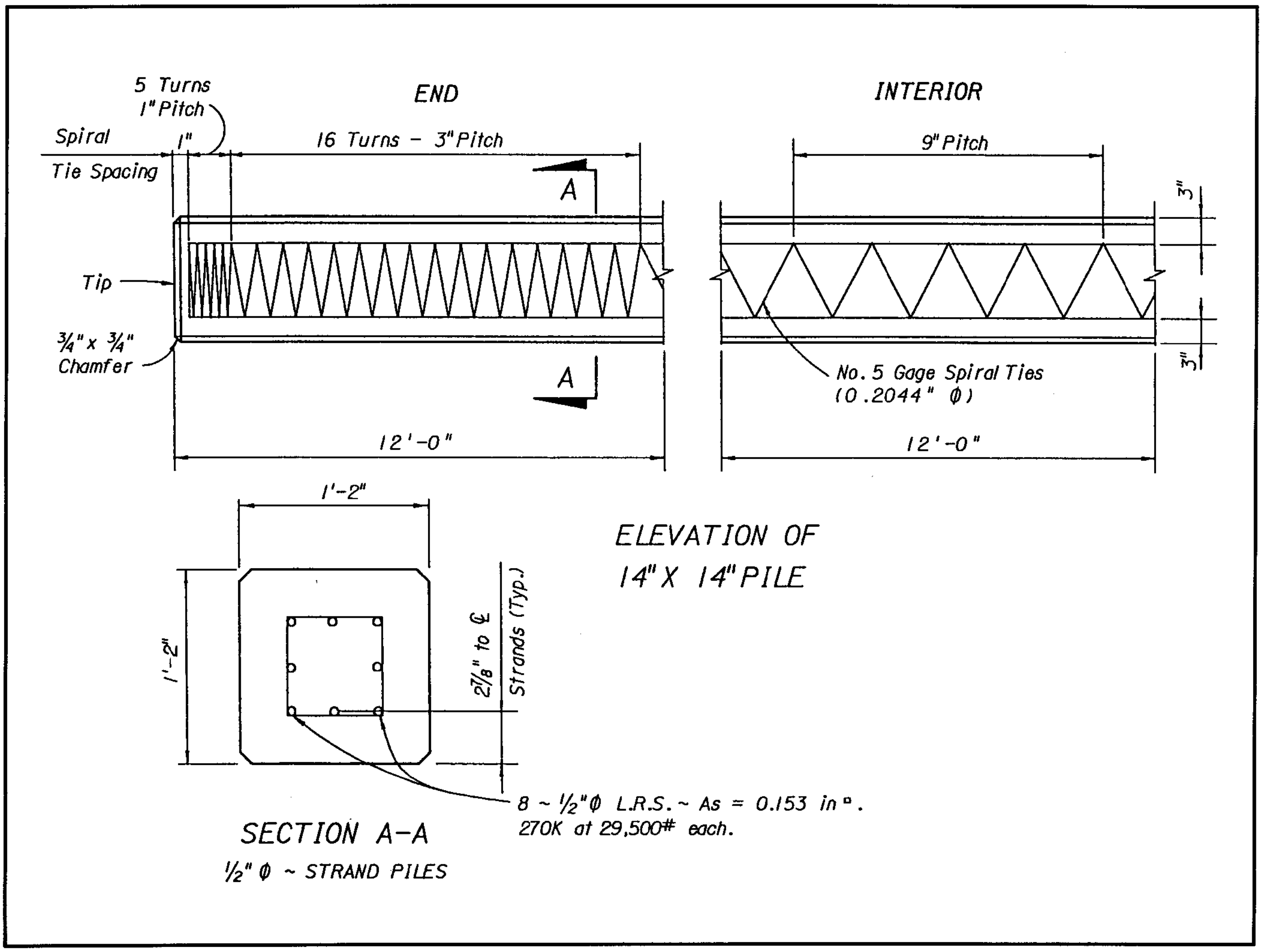

Fig. 1. Details of test specimens.

depends on the initial prestress, f_{si}, and the amount of prestress loss. Zia and Mostafa have pointed out that the denominator "3" in the expression for transfer length represents a conservative average concrete strength in ksi. In the expression for flexural bond length, given in the ACI Code, a denominator of 1 ksi (6.9 MPa) is implied, which represents an average bond stress of 250 psi (1.7 MPa) within the development length.

According to the ACI Code requirement, the transfer length and the flexural bond length would be, respectively, 47 and 105 nominal strand diameters for 250 ksi (1720 MPa) grade strand, assuming $f_{si} = 0.7f_{pu}$, $f_{se} = 0.8f_{si}$ and $f_{ps} = 0.98f_{pu}$ (where f_{pu} is the specified tensile strength of prestressing strand, ksi).

Similarly, for 270 ksi (1860 MPa) grade strand, the transfer length would be 51 strand diameters, and the flexural bond length would be 113 strand diameters. Note that the value of 50 strand diameters is mentioned as the assumed transfer length in Section 11.4.3 of ACI 318-89.

TEST PROGRAM

Nineteen 14 in. (356 mm) square pretensioned prestressed concrete piles were tested in this investigation. All specimens were prestressed with eight ½ in. (13 mm) diameter prestressing strands confined by 5-gauge spiral reinforcement. A summary of the test program is presented in Table 1. Fig. 1 shows a typical pile cross section and reinforcement details.

The test specimens were obtained by cutting sections from 80 ft (24.4 m) long prestressed concrete piles which were left over from a previous bridge construction project. Thus, the spiral reinforcement varied along the length of the test specimen as shown in Fig. 1.

The end sections were provided with more spiral reinforcement than the middle sections. Each test specimen was approximately 12 ft (3.66 m) in length. Sections cut from the ends and from the middle of each pile were tested to study the effect of the shear confinement provided by the spiral reinforcement on the development length.

Cores of 6 in. (152 mm) diameter were taken from all test specimens and tested to determine the compressive strength of concrete. The results of these tests are shown in Table 1.

Test Setup and Instrumentation

Load testing of the piles required a test frame that would simulate the behavior of a pile cap. The frame should restrain the pile against translation and rotation at the junction of the pile and the frame. A reaction frame was built from several HP 14 x 73 in. (356 x 1854 mm) steel sections. Fig. 2 shows the details of the test frame which was anchored to the structural floor.

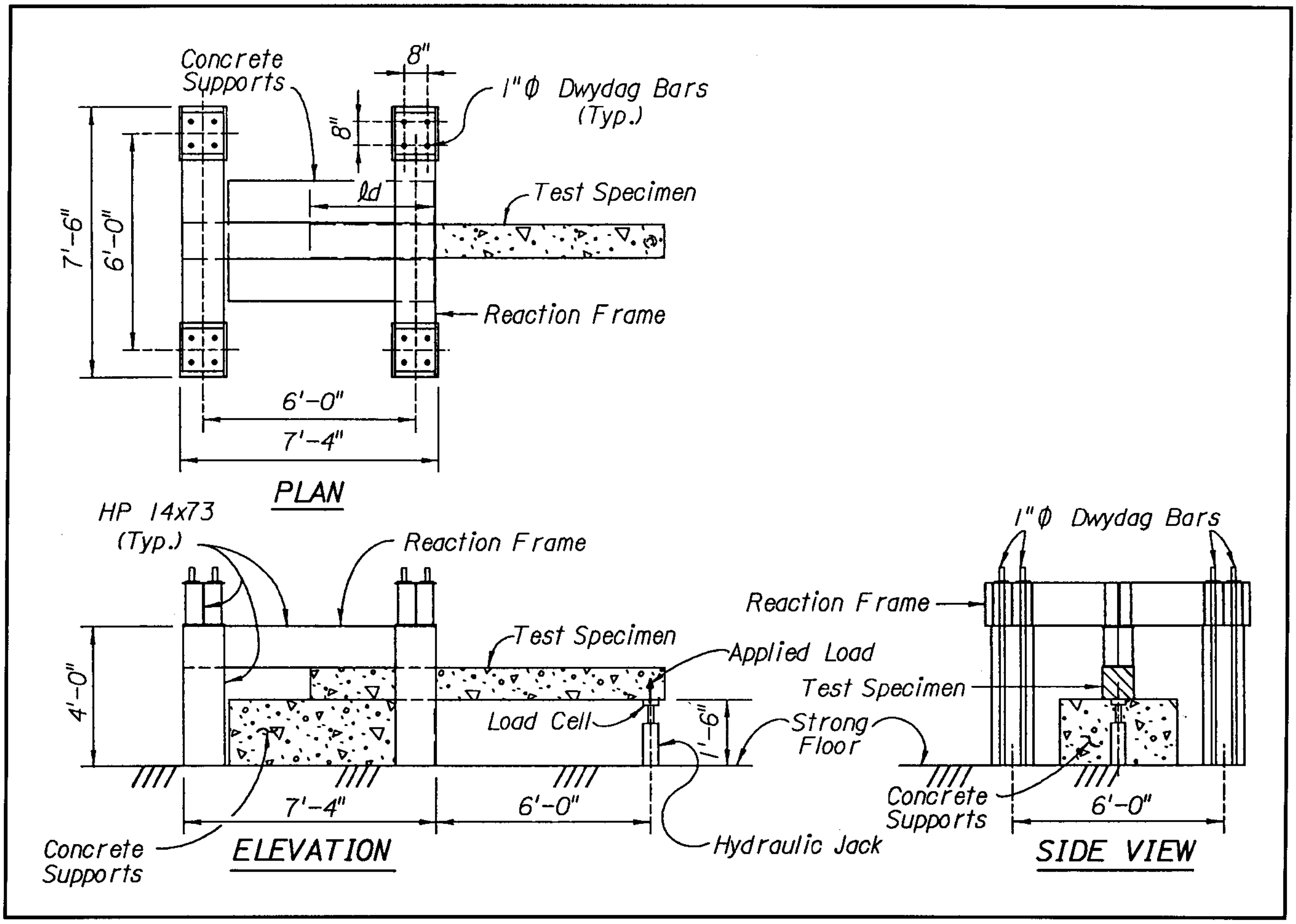

Fig. 2. Details of test setup.

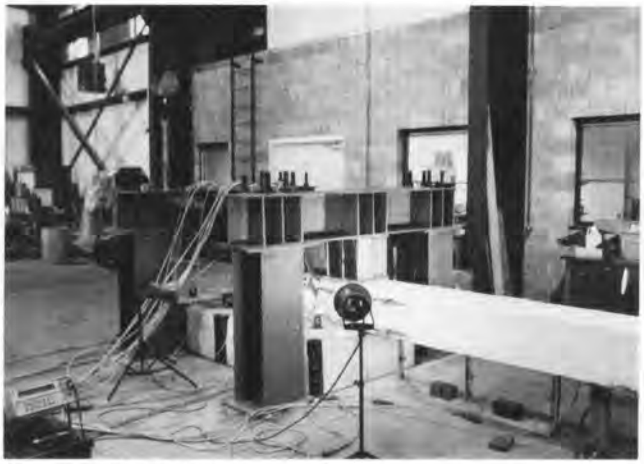

Fig. 3(a). Pile test setup.

Fig. 3(b). Closeup of pile test setup.

A hydraulic jack, supported on the floor, was used to apply the load to the pile at a distance of 6 ft (1829 mm) from the face of the supporting frame. The frame provided restraint against translation and rotation in the vertical and horizontal directions to simulate the actual conditions. In actual conditions, the pile is fully restrained by the clamping force resulting from shrinkage of the confined concrete in the pier cap.

In the test, a clamping force was applied to the upper and lower faces of the embedded length of the pile. This force was applied by tensioning the Dywidag bars at each corner of the test frame to a value of 50 kips

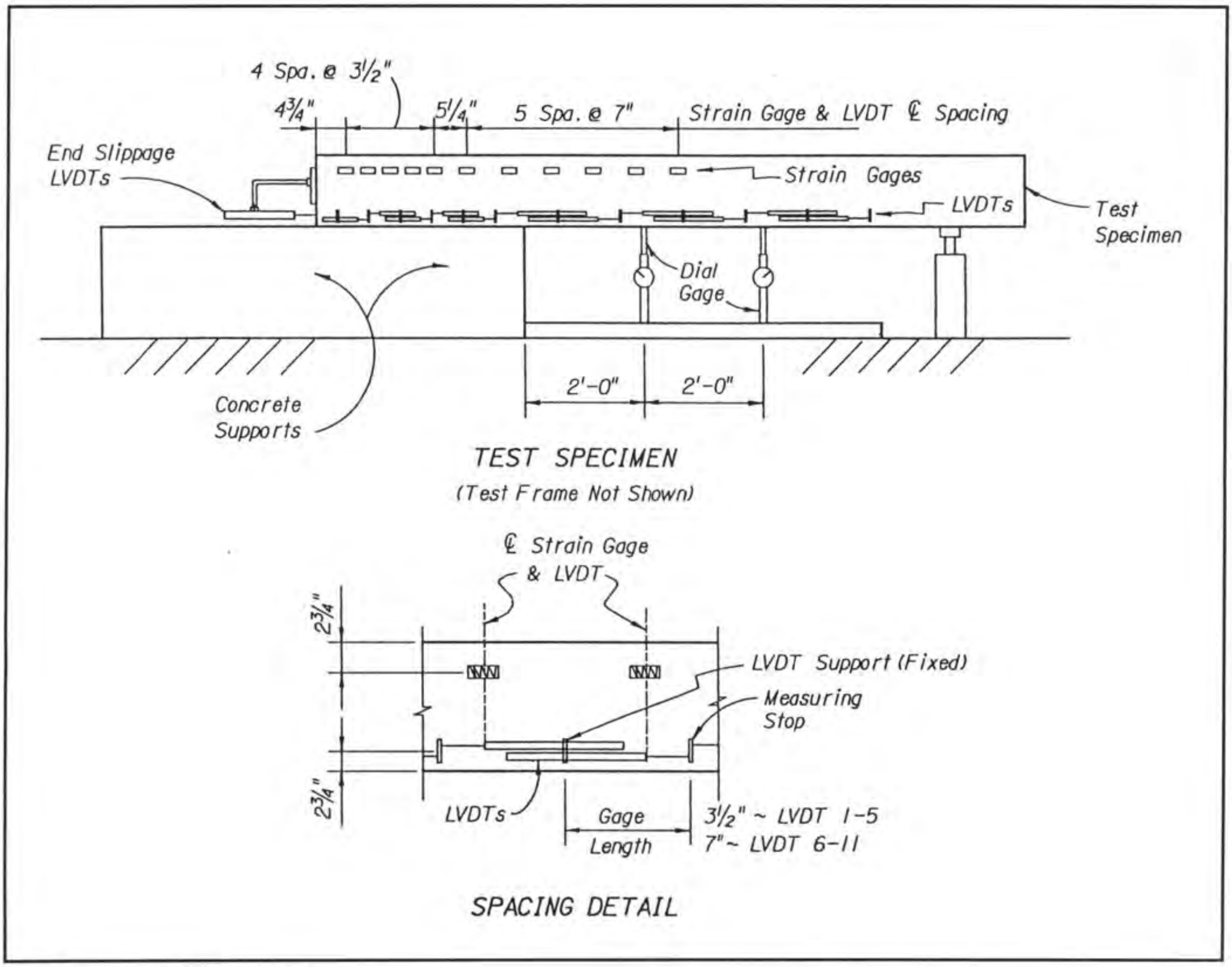

Fig. 4. Location of instrumentation.

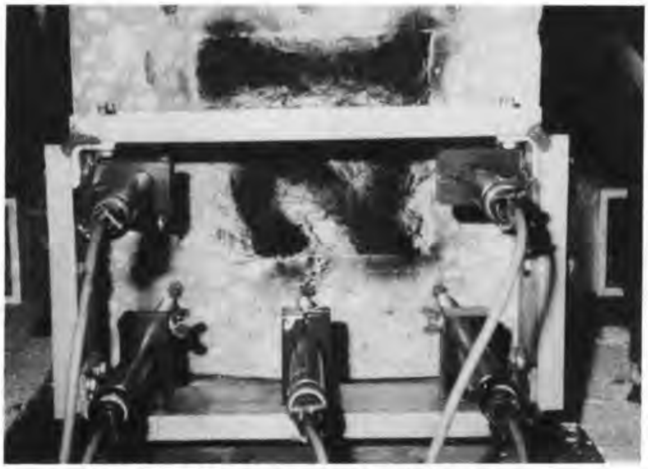

Fig. 5. LVDT locations to measure end slippage.

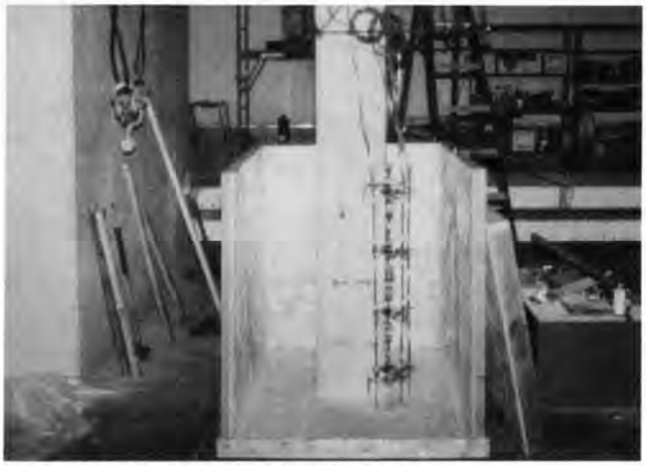

Fig. 6. Pier cap prior to concrete casting.

(222 kN). In total, a force of 200 kips (888 kN) was applied. This force was translated into contact pressure applied to the pile surface. Figs. 3(a) and 3(b) show a view of the pile specimen in the testing frame.

The clamping stress resulting from the constant force [200 kips (888 kN)] varied depending on the length of pile embedment. The pile embedment length varied between 36 to 60 in. (914 to 1524 mm) with corresponding clamping stresses of 397 and 238 psi (2.74 and 1.64 MPa), respectively. The value of the constant clamping force was established based on initial tests to measure the effect of concrete shrinkage around a pile cap or a footing.

Vertical deflection at the free end of the pile, strains in the concrete and slip of the prestressing strands at the restrained end of the pile were measured using dial gauges, electrical resistance strain gauges (ERSGs) and linear voltage differential transducers (LVDTs).

The strain gauges were mounted near the upper side of the pile to measure the compressive strain in the concrete. The LVDTs were mounted

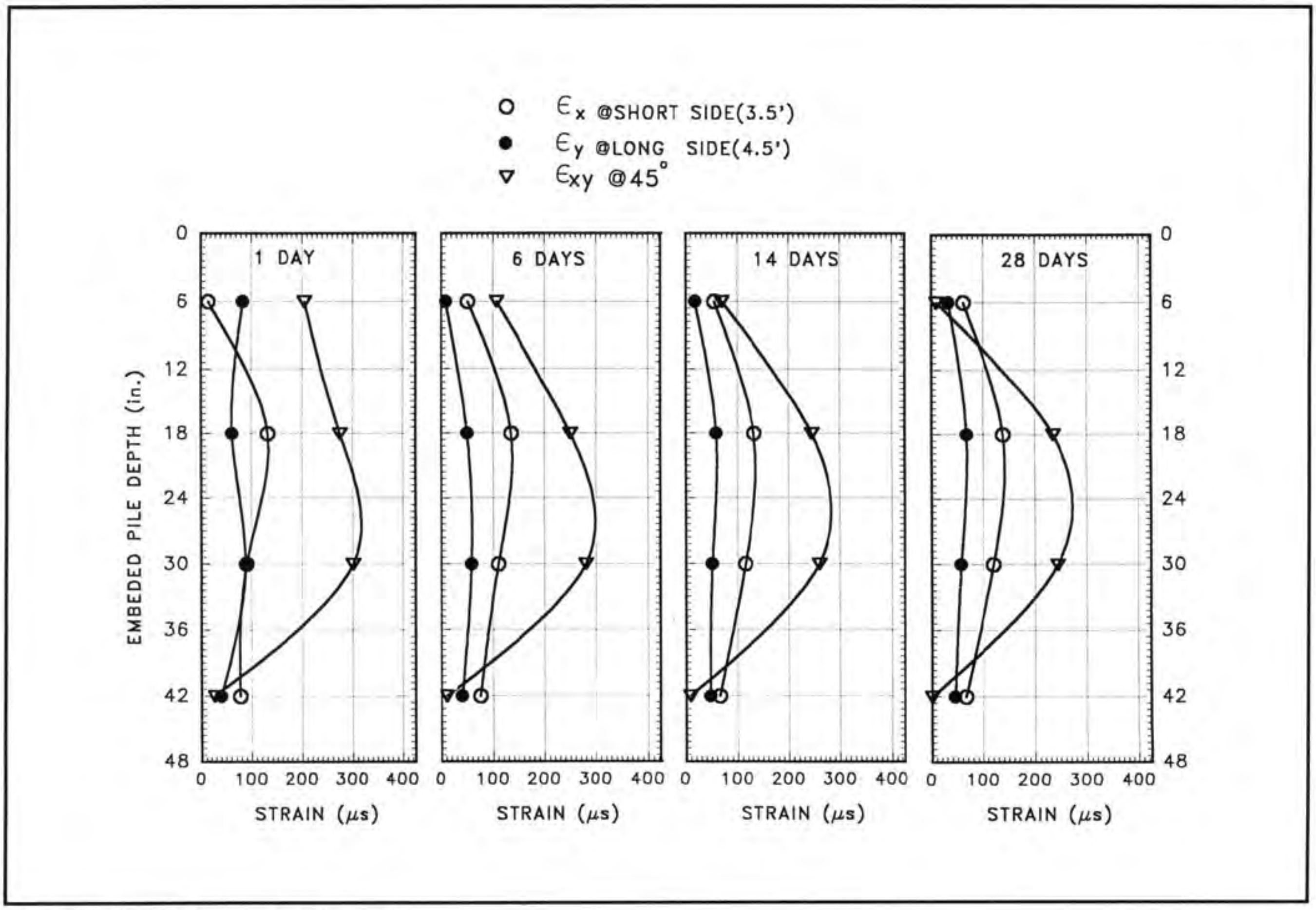

Fig. 7(a). Pile confinement strain results.

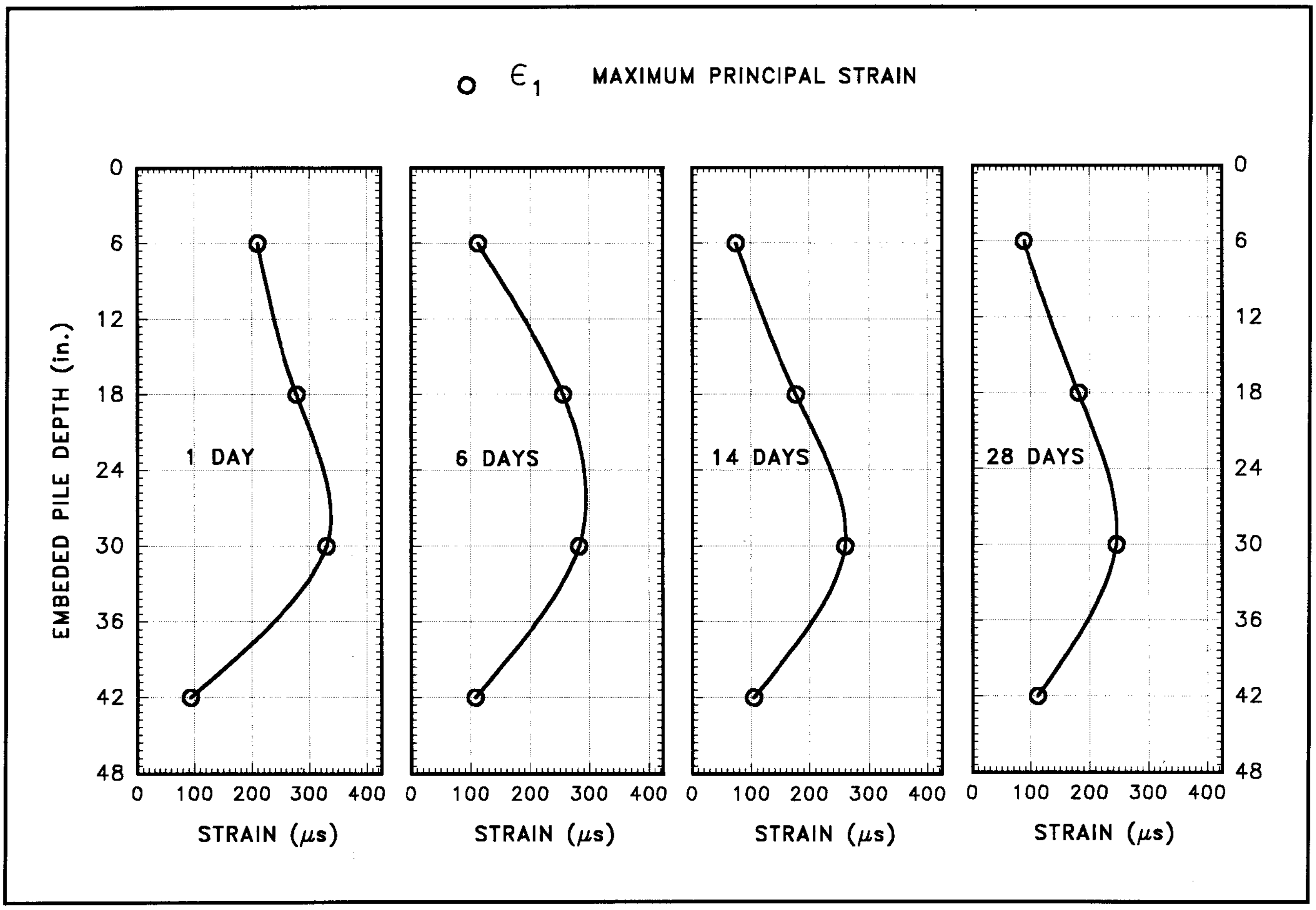

Fig. 7(b). Pile confinement strain results.

near the lower face to measure tensile strain in the concrete. Strain gauges and LVDTs were, respectively, mounted at 2¾ in. (70 mm) along the pile length in order to measure the strains at the level of the prestressing tendons. Fig. 4 shows the locations of instrumentation on a test specimen.

Dial gauges were used to measure pile deflection at 2 and 4 ft (0.61 and 1.22 m), respectively, from the face of the frame. Horizontal LVDTs were mounted at the embedded end of the pile to measure any slip in the lower prestressing tendons as shown in Fig. 5. A load cell placed between the hydraulic jack and the pile was used to measure the force applied to the pile.

Test Procedure

First, the pile was placed in the test frame and the 200 kips (888 kN) clamping force was applied. The pile was then loaded incrementally at the free end until failure occurred. Failure was defined as slip of the prestressing strands or flexural failure due to yielding of the steel and crushing of the concrete at the face of the support. Deflection and strain measurements were recorded at specified load increments during the test.

Loading was applied by means of the hydraulic jack in increments of 3 kips up to 18 kips (13.34 kN to 80.07 kN). The load was then applied in smaller increments up to failure. Cracks were marked at each load step to follow their development.

EFFECT OF CONCRETE SHRINKAGE ON PILE CONFINEMENT

An initial test was conducted to study the effect of shrinkage of concrete around an embedded length of a pile in a footing or in a pile cap. The objective of this test was to determine a realistic value for the clamping force to be used throughout the experimental investigation. This force, as discussed earlier, represents the confining stress exerted on a precast concrete pile embedded in a footing or a pier cap due to the shrinkage of the concrete mass around the pile.

A wood form for a section of a pier cap was constructed. This section represents an end segment of a pier cap and has a cross-sectional dimension of 42 x 54 in. (1067 x 1372 mm) and a depth of 48 in. (1219 mm). A 12 ft (3658 mm) section of a 14 x 14 in. (357 x 357 mm) square pile was then placed at the center of the form.

Prior to casting, the concrete pile section was instrumented with vibrating wire strain gauges placed at 12 in. (305 mm) spacing along the entire length of embedment at different directions as shown in Fig. 6. The gauges were used to measure the strain variation along the entire length of embedment as the concrete hardened around the pile.

The strain readings were automatically recorded every 30 minutes for the first 7 days, and then every 60 minutes for the duration of the test.

The measured strain results along the embedment length at different times are

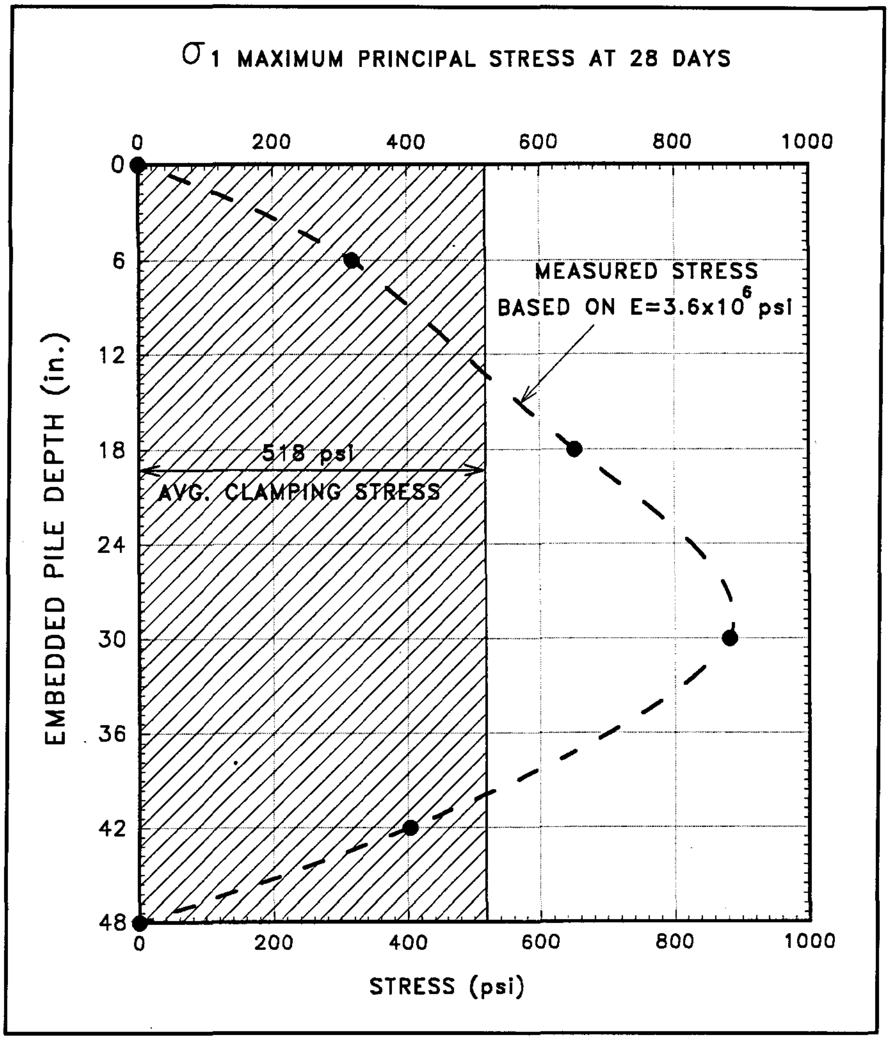

Fig. 8. Average clamping stress.

shown in Fig. 7(a). These results were used to calculate the principal strains along the length of embedment at different times. The principal strain distributions along the embedment length are shown in Fig. 7(b).

It can be seen from Fig. 7(b) that, after an age of 28 days, the distribution of the principal strains followed a parabolic curve with a maximum measured value of 245 microstrains, at a depth of 30 in. (762 mm). The results also indicated that most of the strain changes occurred in the first 3 weeks of the investigation, after which the measured strains were constant.

Fig. 8 shows the stress distribution along the embedded length at 28 days based on an assumed conservative value for the concrete modulus of elasticity, E_c, of 3.6 x 10^6 psi (24800 MPa). An average confining stress of 525 psi (3.6 MPa) is obtained by dividing the area under the curve by the 48 in. (1219 mm) embedment length.

This average stress value was considered to be an upper limit for the applied contact stress to any test specimen. In all the tests, the maximum applied clamping stress was 397 psi (2.74 MPa) for an embedment length of 36 in. (914 mm). This value repre-

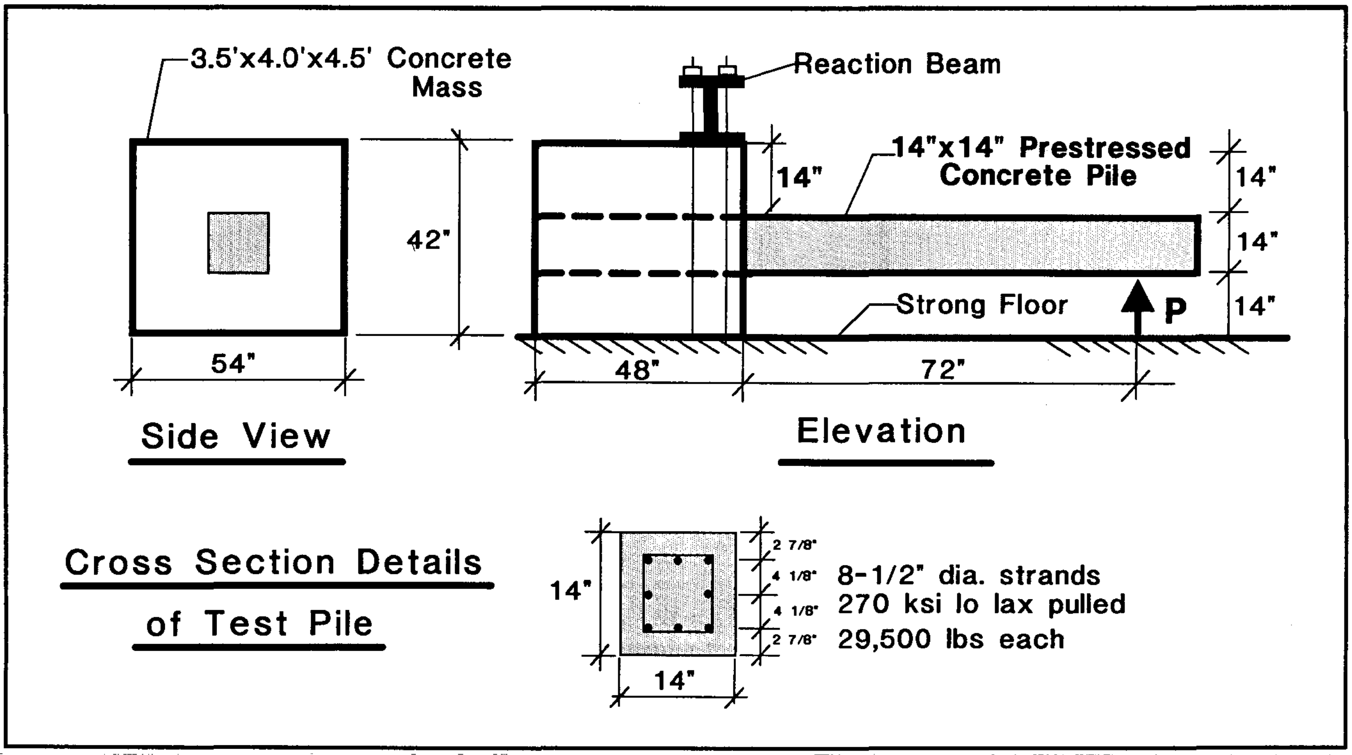

Fig. 9(a). Test setup to simulate a pile embedded 48 in. (1219 mm) into a pier.

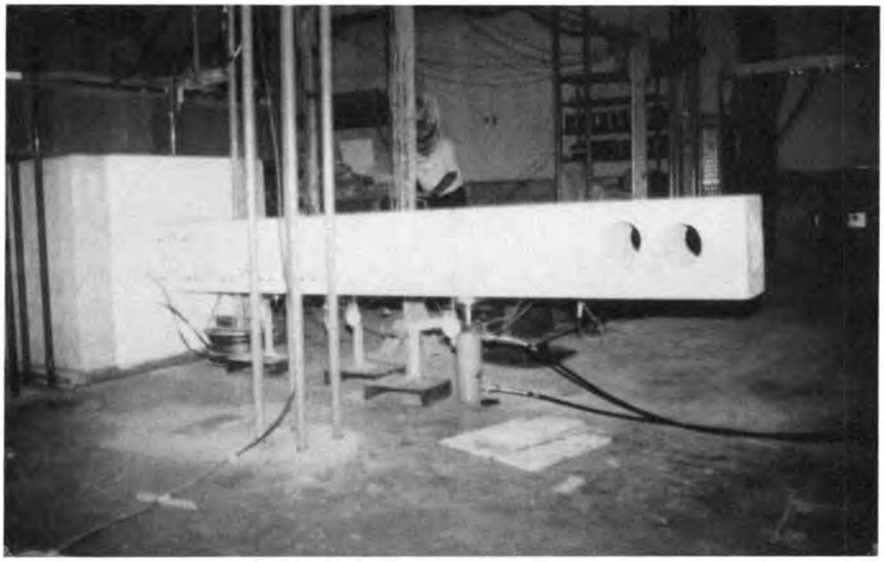

Fig. 9(b). Test setup.

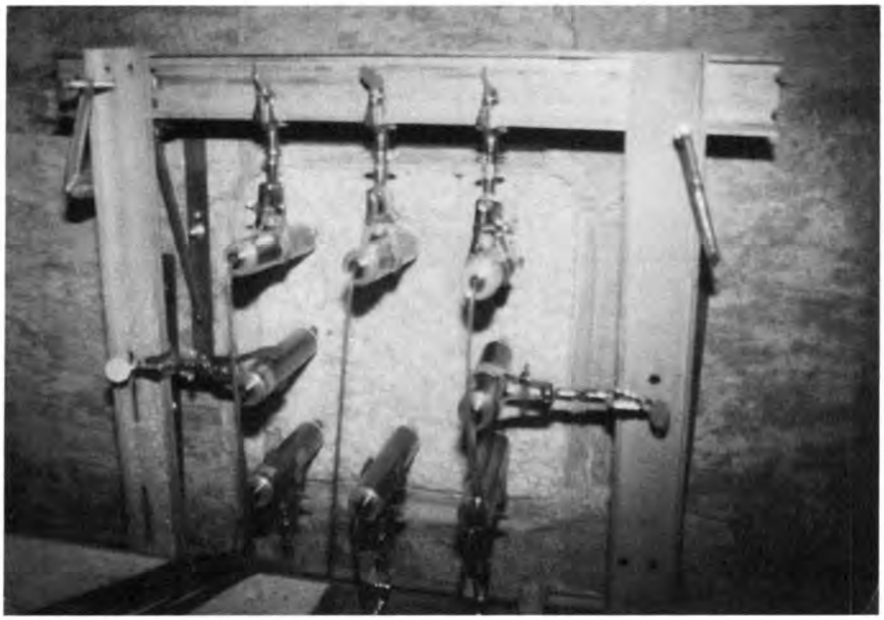

Fig. 10. Slip measurements.

sents a conservative 75 percent of the measured confining stress.

The pile cap test specimen was then instrumented and placed in the test frame as shown in Figs. 9(a) and 9(b). An initial force of 5 kips (22.24 kN) was applied at two points along the face of the support to hold the test specimen in position.

Horizontal LVDTs were mounted at the end of the pile to measure any slip in the prestressing tendons, as shown in Fig. 10. A hydraulic jack, supported on the floor, was used to apply the load to the pile at a distance of 6 ft (1829 mm) from the face of the supporting frame.

The pile was then loaded incrementally at the free end until failure occurred. The mode of failure was flexure at the interface between the pile cap and the pile as shown in Figs. 11(a) and 11(b). No strand slippage or any signs of bond deterioration was observed.

Fig. 12 shows the strain along the pile at different load stages. These results indicated that a 48 in. (1.22 m) embedment was adequate to develop the ultimate moment capacity of this section without any strand slippage.

ANALYTICAL STUDY

A nonlinear material analytical model was used to analyze the prestressed concrete piles. The program uses a numerical procedure to simulate the material and conducts a geometric nonlinear analysis of plane prestressed

Fig. 11(a). Overview of mode of failure.

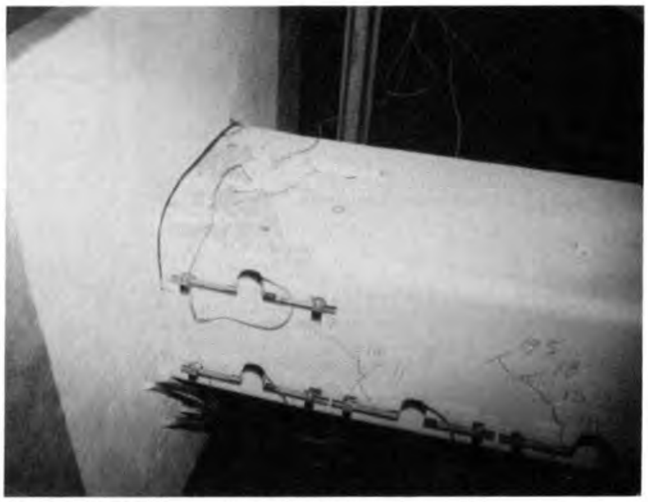

Fig. 11(b). Closeup of mode of failure.

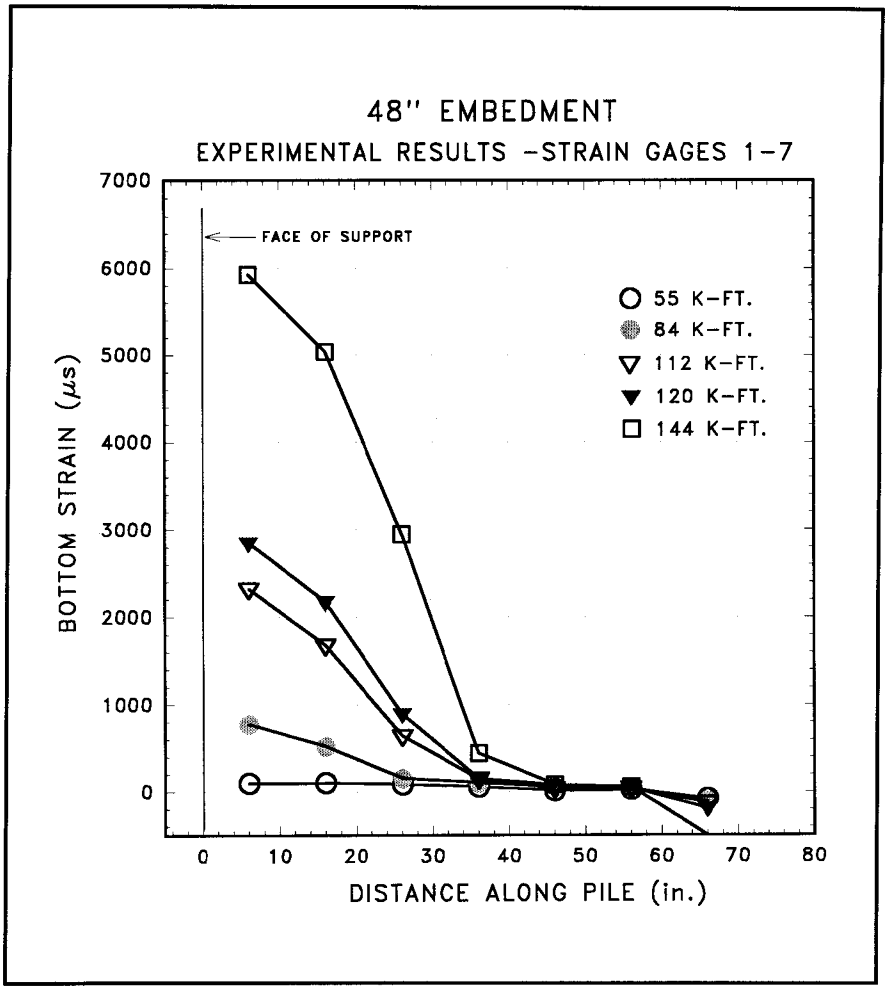

Fig. 12. Tension strain along pile.

Table 2. Test results.

Specimen number	Embedment length (in.)	Measured ultimate moment (kip-ft)	Theoretical ultimate moment (kip-ft)	Measured ult. mom. / Theoretical ult. mom.	Strand slip
A-1E	36	153.0	130.05	1.176	No
A-2E	36	150.0	122.02	1.229	No
A-3I	36	129.0	127.11	1.015	Yes
A-4I	36	129.0	120.15	1.074	Yes
B-1E	42	135.0	127.78	1.056	Yes
B-2E	42	156.0	126.25	1.235	No
B-3E	42	147.0	123.06	1.194	Yes
B-4E	42	130.0	133.48	0.974	Yes
B-5E	42	153.0	126.43	1.210	No
B-6I	42	132.6	126.43	1.044	Yes
C-1E	48	126.0	129.28	0.975	Yes
C-2E	48	141.0	126.57	1.110	No
C-3I	48	147.0	133.31	1.103	No
C-4I	48	138.0	126.57	1.090	No
C-5I	48	140.4	126.57	1.106	No
C-6E	48	142.2	126.57	1.122	No
D-1E	60	144.0	130.57	1.103	No
D-2I	60	144.0	126.57	1.138	No
D-3E	60	135.0	126.57	1.067	No

Metric (SI) conversion factors: 1 in. = 25.4 mm; 1 kip-ft = 1356 N-m.

concrete frames. The computer program considers the time-dependent effects due to load history, temperature history, creep, shrinkage and aging of concrete, and relaxation of prestress.

The response of a structure can be calculated by the program through the elastic and inelastic ranges up to the ultimate load. At each load level, nonlinear equilibrium equations, which are valid for the current geometry and material properties, are derived using the displacement formulation of the finite element method. The equations are then solved by an iterative procedure.

The main objective of the analytical study was to develop a reliable computer program for use in future parametric studies.

RESULTS AND DISCUSSION

The main experimental and theoretical results are presented and discussed in this section. Only typical diagrams are presented to discuss the behavior. The test results for all specimens are given in Ref. 12.

Ultimate Moments

Calculation of the ultimate moment at failure provided information on ultimate load and the net steel stress in prestressing steel. The ACI strain compatibility analysis was used to determine the effective strand stress, the average bond stress and the ultimate moment capacity. The measured external moment, producing failure, was compared to the calculated ultimate load in Table 2.

Seventeen of the test specimens failed between 2 and 20 percent higher than the theoretical ultimate moment, and the remaining two specimens failed at 2 percent less than the theoretical ultimate moment. These results show that eight of nine piles with 48 in. (1.22 m) embedment or higher reached their ultimate theoretical moment without any slip.

Fig. 13 shows a comparison between analytical and experimental results for a typical applied moment-deflection curve. Typical compressive strain behaviors along the embedment

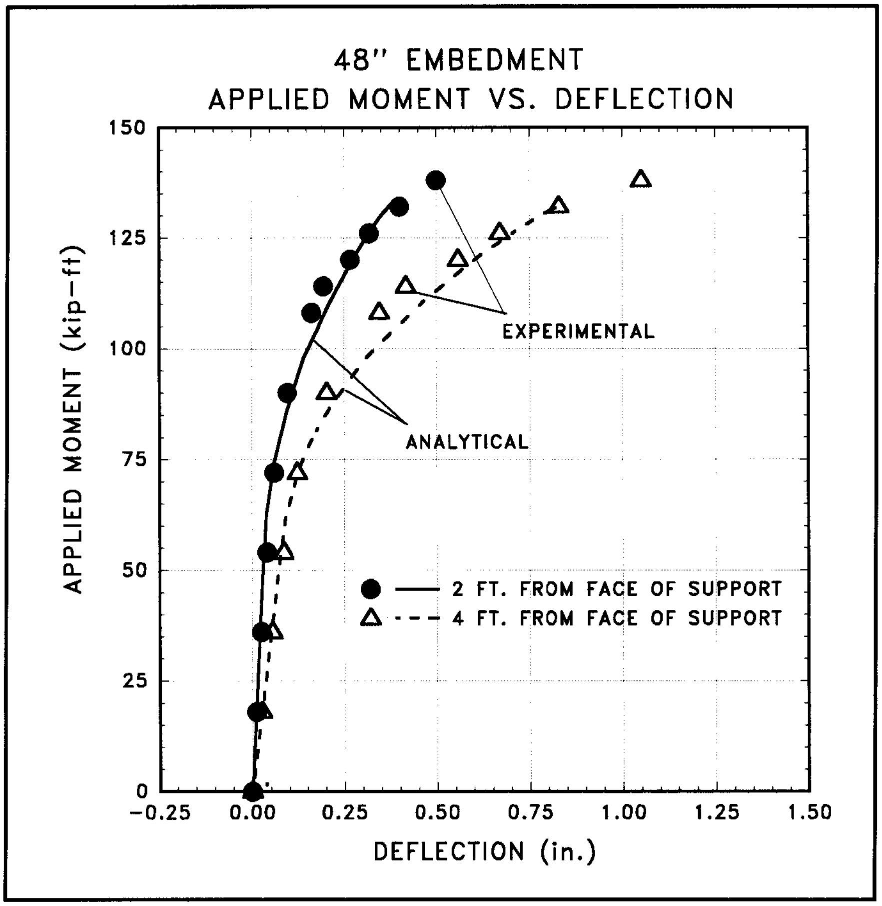

Fig. 13. Comparison between experimental and analytical results (Pile C-2E).

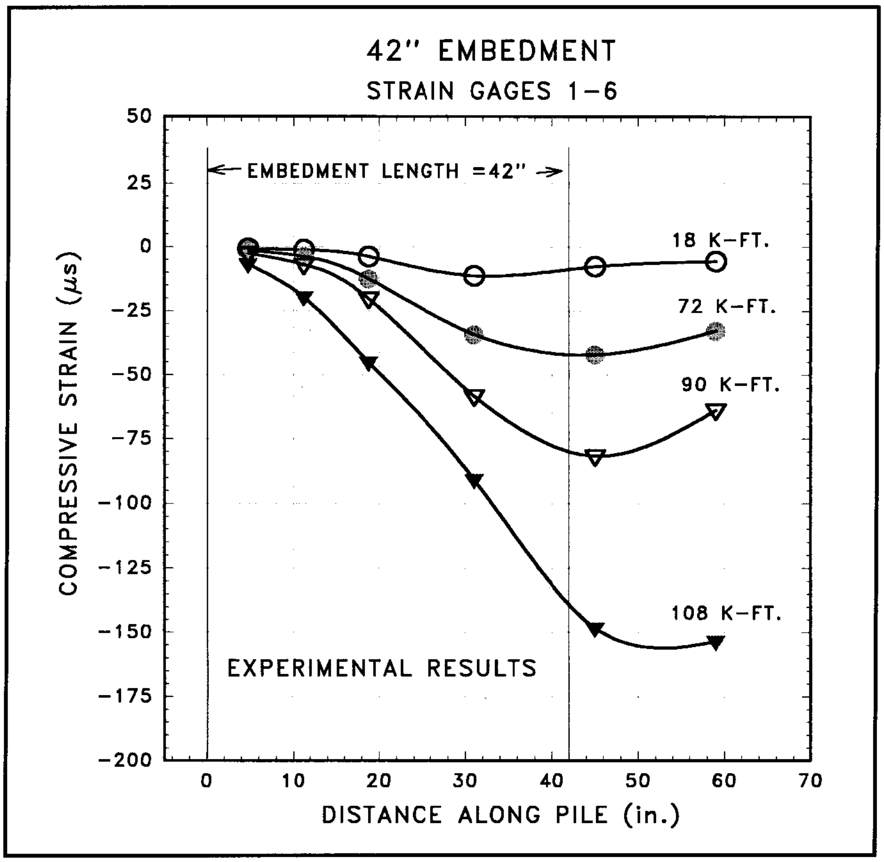

Fig. 14. Top strain vs. distance along pile (Pile B-2E).

length of Specimen B-2E at different applied moments are shown in Fig. 14. The same pile, with a typical plot of the tension strain along the embedment length at different applied moments, is shown in Fig. 15.

Fig. 16 shows the applied moment-strain relationship along the pile at different locations. In general, good agreement is obtained between the experimental and analytical results. It can be seen from the figure that the ultimate moment can be predicted by the computer model.

Fig. 17 shows the variation in stress along the length of the prestressing tendon at various levels of applied moments for Specimen C-1E. This behavior was typical for all the test specimens.

The extent and patterns of cracking in the test specimens, as predicted by the nonlinear analysis and as observed in the pile test at failure, are presented in Ref. 12. The agreement between the observed and predicted crack patterns is excellent.

Fig. 18 illustrates the relationship between average bond stress and embedment length for ½ in. (13 mm) diameter strands. The relationship shown in this figure supports the generally accepted assumption that average bond stress is proportional to embedment length in the experimental and analytical results.

Effect of Shear Confinement Steel

The spiral shear reinforcement varied along the length of the pile as shown in Fig. 1. The test specimens from the pile end section were provided with more shear reinforcement than those from the interior section. Fig. 19 shows that the shear confinement at the end section generally increased the moment capacity of piles by about 6 percent (see Table 2). Also, it can be seen in the figure that the effect of the shear reinforcement is more significant for short embedment lengths, i.e., 36 and 42 in. (914 and 1067 mm).

Pile Embedment Length

The effect of variation of pile embedment length on the moment at general bond slip and on the ultimate moment of resistance of the 14 in. (356 mm) square piles is presented in Table 2. It can be seen in the table that only one specimen having an embedment length of 48 in. (1.22 m) or higher showed strand slip at 98 percent of the theoretical ultimate moment. Seven test specimens showed strand slip prior to failure, and four of these specimens failed at moments higher than their design ultimate moments. The remaining two specimens failed within 2.5 percent of their ultimate capacities.

The maximum bond stresses at failure were calculated based on the measured steel stresses using the following equation:

$$U = (P - T)/\pi\, l_e d_b$$

where

P = resisting steel strength
T = resisting concrete strength
l_e = available embedment length
d_b = nominal strand diameter

The calculated average bond stresses for the 36, 42, 48 and 60 in. (914, 504, 1219 and 1524 mm) embedments are 694, 604, 526 and 424 psi (4.78, 4.16, 3.63 and 2.92 MPa), respectively. These stresses were based on a zero resisting concrete strength (T = 0) at ultimate after cracking.

The required development length for each specimen was calculated using the ACI formula [Eq. (6)] with the actual calculated bond stress and the measured steel stress values. The ultimate steel stress, f_{ps}, was calculated by projecting the total strain (initial strain plus experimental strain at failure) on the stress-strain curve of the ½ in. (13 mm) diameter strand.

A comparison between the calculated development length results and the provided embedment length is shown in Table 3. Comparing the results from Tables 2 and 3, it can be concluded that an embedment length of 48 in. (1.22 m) or higher is sufficient to develop the full ultimate moment without strand slip.

Table 4 shows the calculated development length based on Eqs. (3), (5) and (6). The calculated values using Eqs. (3) and (5) show a minor variation in the calculated development

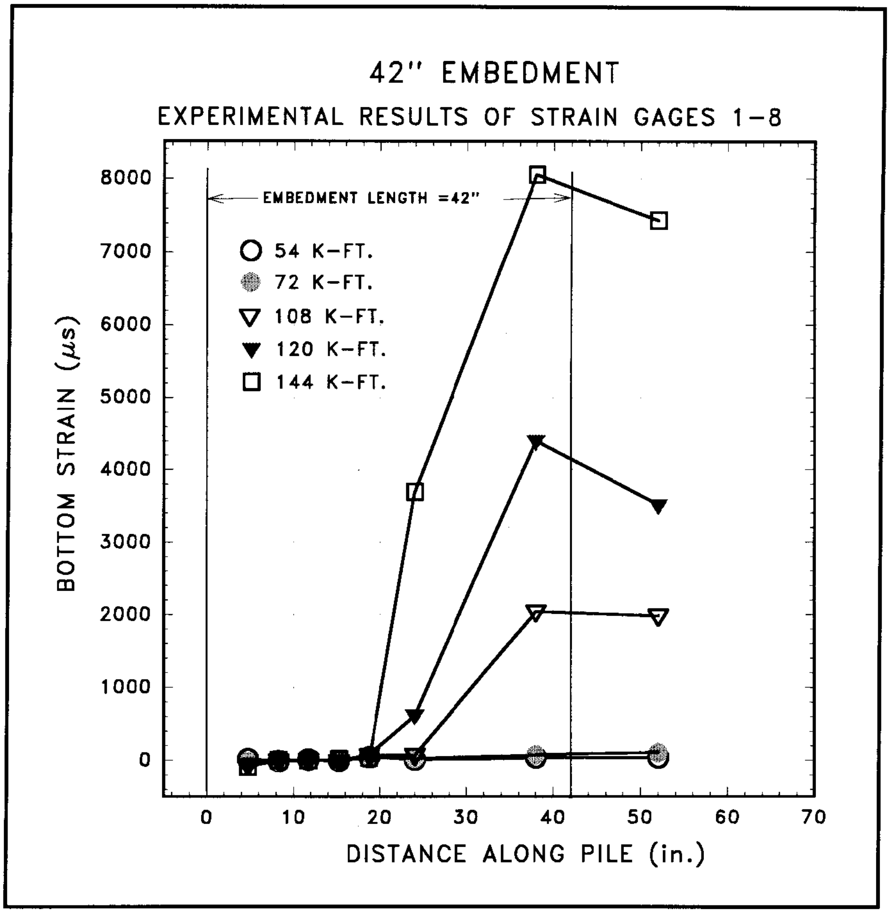

Fig. 15. Tension strain along pile (Pile B-2E).

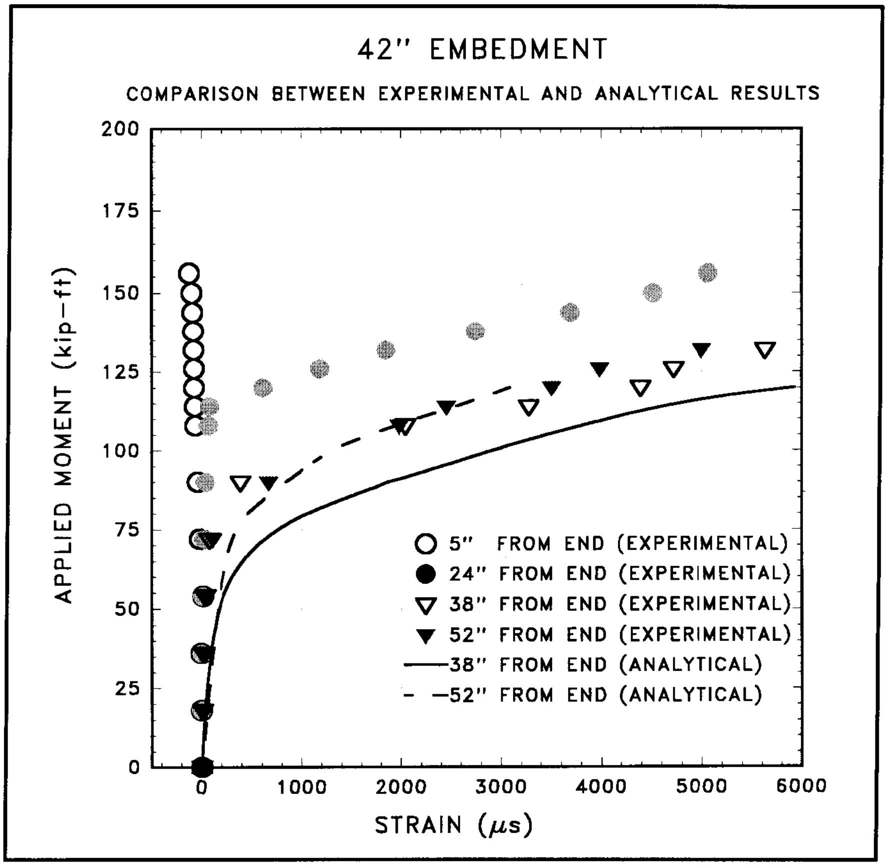

Fig. 16. Applied moment vs. bottom strain (Pile B-2E).

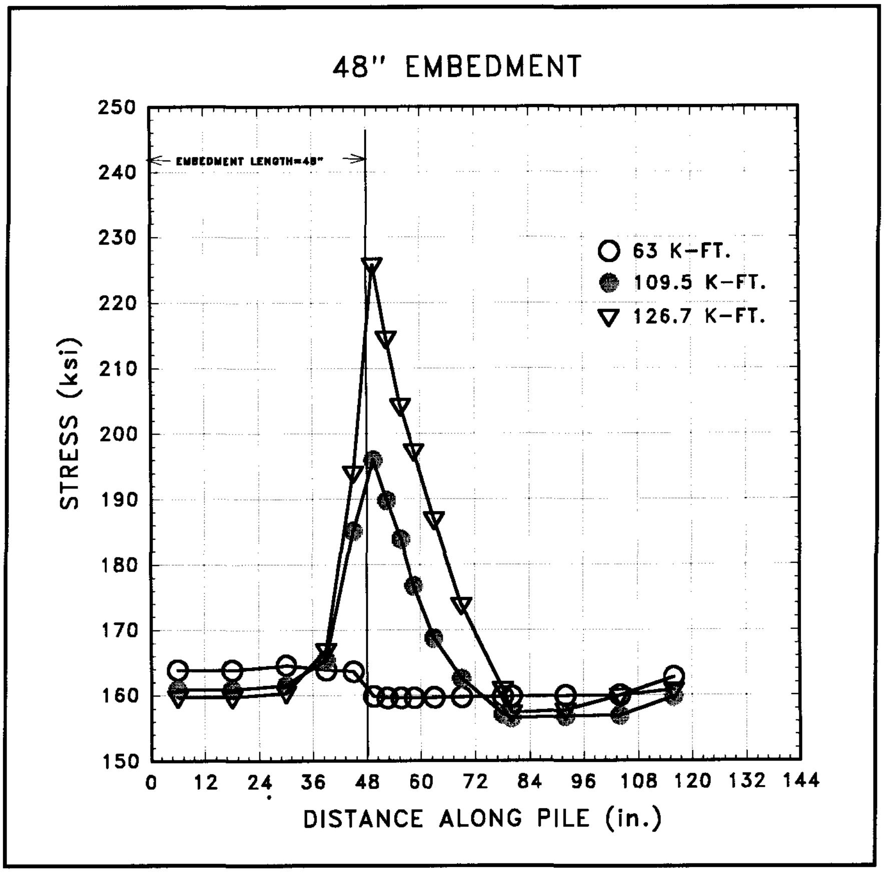

Fig. 17. Strand stress along pile at different moments (Pile C-1E).

length. This is because these two equations use a constant value for the average bond stress at different pile embedments. The values obtained using Eq. (6) show some variation in the development length for different embedment lengths, which is consistent with the actual bond stress experimental results shown in Table 3.

It can be seen in the table that the provided embedment lengths agree very well with the values derived from Eq. (6). It should be further noted that, with the exception of one specimen, none of the piles with 48 or 60 in. (1.22 or 1.52 m) embedment lengths slipped or failed by bond.

Effect of Pile Clamping Force

A constant clamping force of 200 kips (888 kN) was applied to the test specimen by Dywidag bars at the four corners of the test frame. This force was transferred to the pile side as a distributed force (contact stress) depending on the pile embedment length.

For example, a 36 in. (914 mm) pile embedment will produce a contact stress of 397 psi (2.74 MPa). An increase in the contact stress due to shorter embedment length will enhance the confinement at the clamped end of the pile.

Fig. 20 shows that an increase of the confinement stress will result in an increase in the average bond stress. This increase in the average bond stress will reduce the development length of piles.

Table 3. Experimental results.

Specimen number	Embedment length L_e (in.)	Measured steel stress at failure f_{ps} (ksi)	Maximum bond stress (psi)	Average bond stress (psi)	Calculated development length L_d (in.)	Actual $\frac{L_e}{L_d}$	Average $\frac{L_e}{L_d}$
A-1E	36	256.07	693		44.12	0.816	
A-2E	36	262.77	711	694	44.82	0.803	0.815
A-3I	36	253.97	687		43.89	0.820	
A-4I	36	253.45	686		43.84	0.821	
B-1E	42	261.78	607		47.96	0.876	
B-2E	42	260.92	605		47.87	0.877	
B-3E	42	256.88	596	604	47.37	0.887	0.879
B-4E	42	260.35	604		47.80	0.877	
B-5E	42	262.61	609		48.07	0.879	
B-6I	42	259.14	601		47.65	0.874	
C-1E	48	260.28	528		51.01	0.881	
C-2E	48	258.29	523		50.90	0.943	
C-3I	48	261.68	531	526	51.21	0.937	0.942
C-4I	48	257.73	522		50.90	0.943	
C-5I	48	260.06	528		50.99	0.941	
C-6E	48	257.66	523		50.65	0.948	
D-1E	60	261.16	424		57.62	1.041	
D-2I	60	260.90	425	424	57.58	1.042	1.043
D-3E	60	259.68	422		57.37	1.046	

Metric (SI) conversion factors: 1 in. = 25.4 mm; 1 psi = 0.006895 MPa.

Therefore, the development length equation calculated by the ACI Code is conservative for piles since it does not consider the effect of confinement and restraint by the pile cap.

Modes of Failure

Twelve of the piles failed in flexure without prior slippage of the strand along its entire embedment length. The remaining piles failed in flexure after a general bond slip of the strands. The moment sustained at general bond slip was between 2 to 5 percent lower than the nominal moment. A comparison between the experimental and the analytical ultimate flexural moment is presented in Table 2.

Flexural or bond failures occurred long after the cracking moment of a specimen was reached. Flexural cracking was observed before a bond failure, which occurred after considerable end slip of strands was recorded. A flexural failure was characterized by considerable flexural cracking, yielding of steel and finally crushing of the concrete in the compression zone at the point of maximum moment.

Failure by slippage of the strands was observed to occur in two stages, namely, initial general slip of the strand along its whole embedment length and then destruction of the mechanical interlocking effect between the strand surface and the surrounding concrete. In the case of piles with short embedment lengths [36 in. (914 mm)], a small increase of load was seen between these two stages. Fig. 21 shows plots of end slippage of the bottom strands vs. applied moment. No strand slippage was measured during a flexural failure.

CONCLUSIONS

The primary objective of this investigation was to determine the required embedment length of strands in piles so that the ultimate flexural moment can be developed without strand slippage. In this study, equations for development length in the AASHTO Specifications and ACI Code were evaluated. Also examined were the general bond slip and the maximum average bond stress at failure.

Table 4. Calculated development length.

Specimen number	Embedment length (in.)	Development length (in.)		
		ACI Eq. (6) *	ACI Eq. (5) †	Zia-Mostafa Eq. (3)
A-1E	36	44.1	74.2	91.8
A-2E	36	44.8	72.2	89.8
A-3I	36	43.8	73.2	90.8
A-4I	36	43.8	72.2	89.8
B-1E	42	47.9	73.2	91.8
B-2E	42	47.8	73.2	90.8
B-3E	42	47.3	72.2	89.8
B-4E	42	47.8	74.2	92.8
B-5E	42	48.0	73.2	90.8
B-6I	42	47.6	73.2	90.8
C-1E	48	51.0	74.2	91.8
C-2E	48	50.9	73.9	91.7
C-3I	48	51.2	74.2	92.8
C-4I	48	50.9	73.8	91.6
C-5I	48	51.0	73.2	90.8
C-6E	48	50.6	73.2	90.8
D-1E	60	57.6	74.2	91.8
D-2I	60	57.5	73.2	90.8
D-3E	60	57.3	73.2	90.8

Metric (SI) conversion factors: 1 in. = 25.4 mm; 1 psi = 0.006895 MPa.
* Based on the measured average bond stress from Table 3.
† Based on an average bond stress of 250 psi.

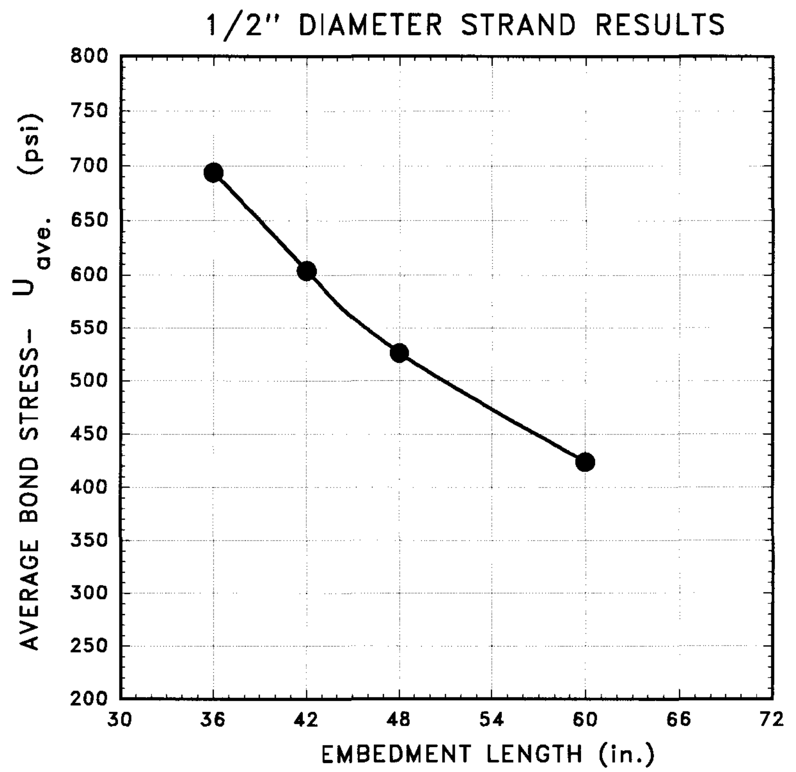

Fig. 18. Average bond stress vs. embedment length.

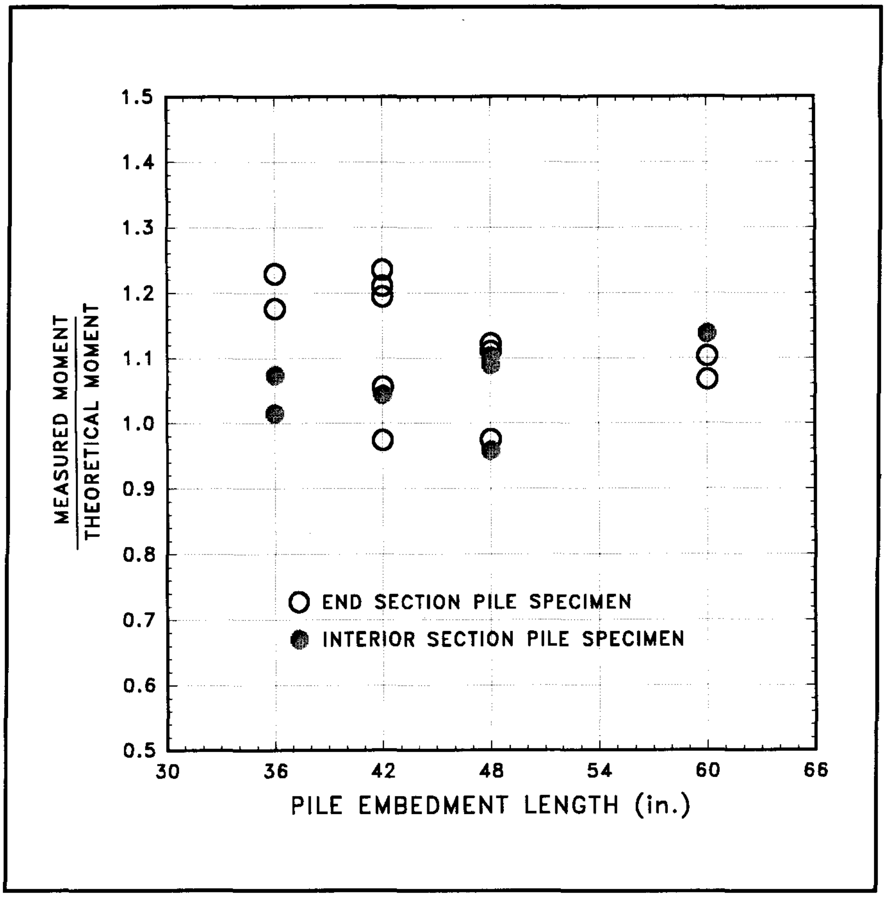

Fig. 19. Comparison between interior and end pile specimens.

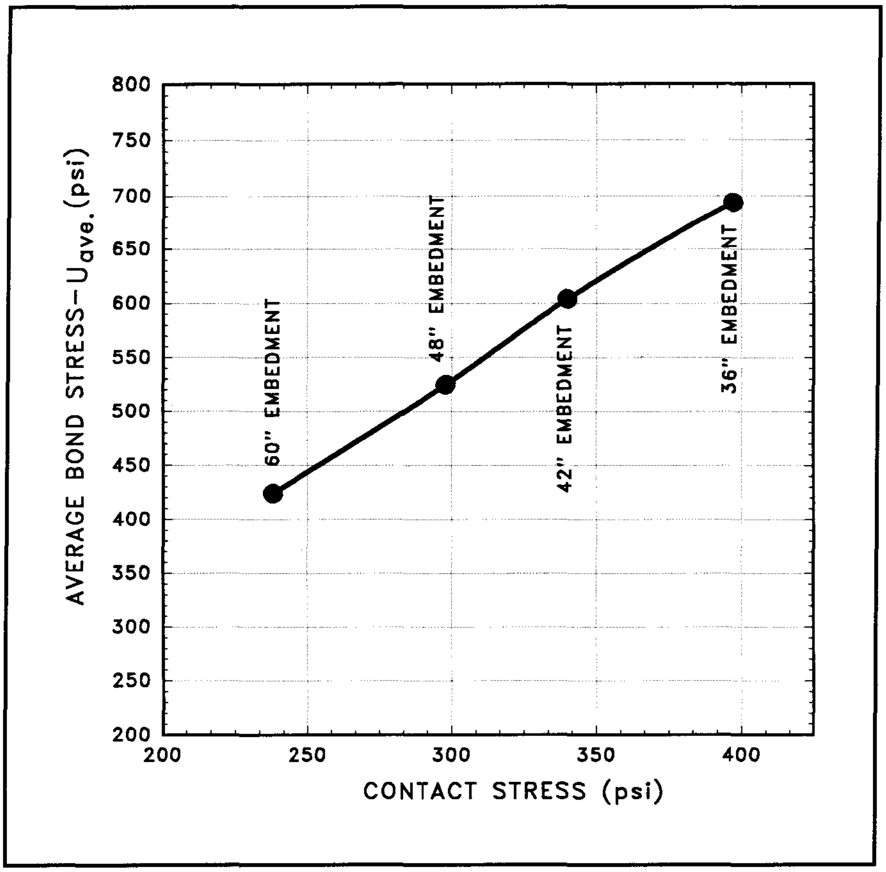

Fig. 20. Contact stress vs. average bond stress.

The following conclusions apply only to piles that are embedded for a certain length in a pier cap or a footing. The results can also apply to other structural members with similar end conditions.

1. The length of pile embedment in a footing or a pier cap has a marked influence on the value of the average bond stress at which general bond slip occurs.

2. Eight piles with 48 in. (1.22 m) embedment or higher failed in flexure without any bond slip. Only one pile with an embedment length of 48 in. (1219 mm) showed a strand slippage at 98 percent of the ultimate moment.

3. The shear confinement steel at the end section generally increased the moment capacity of piles by about 6 percent (see Fig. 17).

4. Shrinkage of the confining concrete in the pile cap creates a clamping force that serves to reduce the development length. A maximum confining principal strain of 245 microstrains occurred at a depth of 30 in. (762 mm) from the face of the support.

5. The current code equation (ACI/AASHTO) for calculating the development length in the case of prestressed piles is very conservative and needs to be modified to reflect the actual end conditions.

6. There appears to be no justification for the application of a multiplier to the development length equation in the current AASHTO Specifications as required by FHWA.

RECOMMENDATIONS

Based on the test results, it is recommended that in prestressed concrete piles:

1. An embedment length of 50 in. [$100d_b$ for ½ in. (12.5 mm) diameter strands] be used. This embedment length is adequate to develop the flexural strength of such piles without slippage of the strands.

2. In the expression for flexural bond length given in the ACI Code and AASHTO Specifications, the implied denominator of 1 ksi (6.9 MPa) should be increased to 2 ksi (13.8 MPa), which represents an average bond stress of 500 psi (3.45 MPa) within the development length.

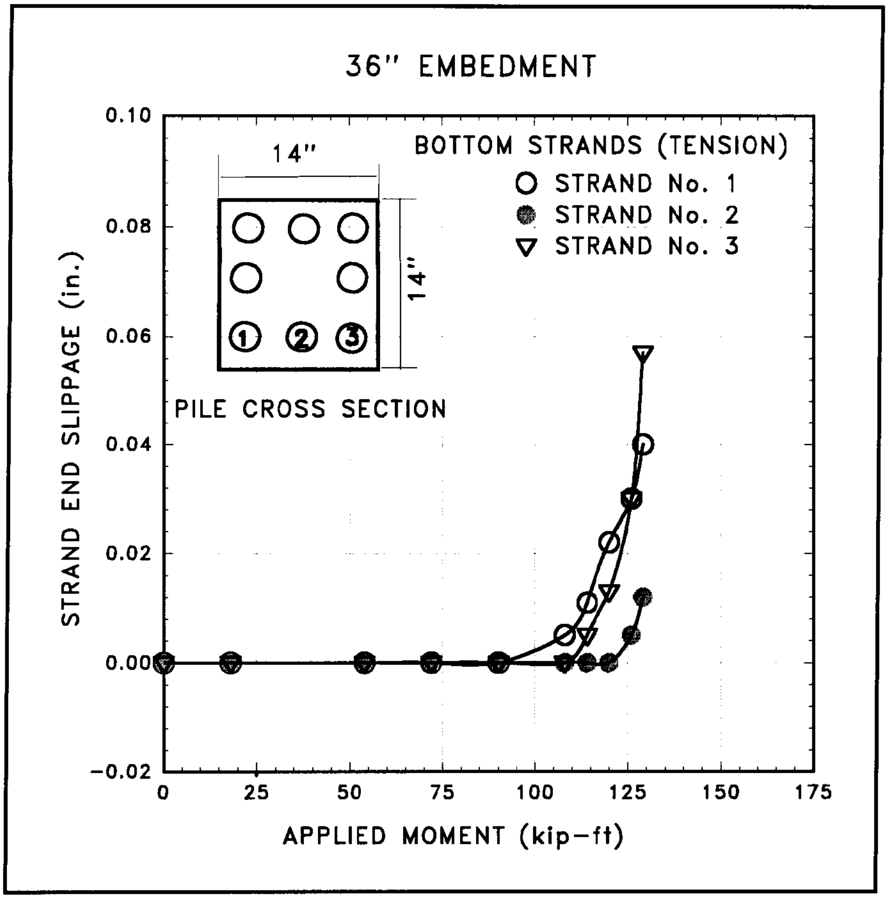

Fig. 21. Applied moment vs. strand end slippage (Pile A-4I).

The above recommendations are only valid for prestressed concrete piles that are embedded for a certain length in a pier cap or a footing.

The opinions, findings and conclusions expressed in this paper are those of the authors and not necessarily those of the Florida Department of Transportation.

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