DEVELOPMENT LENGTH
OF
PRESTRESSED CONCRETE PILES

M. SHAHAWY
M. ISSA
M. POLODNA

TABLE OF CONTENTS

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

The development length for prestressing strands has been debated for some time now. Questions regarding the validity of AASHTO's equation 9-32 have been raised, based on tests conducted by Zia and Mostafa[18] . This resulted in the Federal Highway Administration (FHWA) initially requiring application of a 2.5 multiplier to AASHTO equation 9.32, while the Florida Department of Transportation (FDOT) has proposed a value of 1.5. After further deliberations, a recommendation was made to the FHWA that the multiplier should be 1.6. At a joint meeting in Philadelphia between the AASHTO Technical Committee for Prestressed Concrete (T10) and PCI Bridge Committee on October 11, 1988, the recommendation for a multiplier value of 1.6 was formally presented. It was recommended that this multiplier value should be used for strands up to and including 9/16 inch special. The FHWA accepted this recommendation.

FDOT also questioned whether the use of a multiplier was necessary in the design of piles resisting loads mainly in bending, however, the FHWA insisted on the use of a multiplier.

The FDOT position was based on the following:

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

2. 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 confined concrete in
    the footing creates a clamping force that serves to
    reduce the development length. This condition was
    confirmed by Stacher and Sozen[17]. Further, a
    prying action results when a moment is applied at
    the interface of the pile and the pier 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.

Based on the above, the FDOT has maintained that the
AASHTO. equation 9-32 might, in fact, be too conservative for
piles which are properly embedded in a pier cap and designed
to resist bending

and shear forces due to ship impact.

The question arose from consideration of the Howard Frankland Bridge which was under construction across Upper Tampa Bay connecting Tampa and St. Petersburg.

The bridge has been designed using the current AASHTO requirements and was already under construction when the development length issue arose. A retroactive application of the 2.5 multiplier would have resulted in an increase of embedment from 5' to 13.5' on the 2000 kip capacity piers and from 3.5' to 9.0' on the 1200 kip capacity piers. The subsequent reduction of the multiplier from 2.5 to 1.6, resulted in no changes to the design of 2000 kip piers, but required an increase from 3.5' to 5.0' in the embedment requirements for the 1200 kip piers.

In an effort to generate some test data on this issue, the FDOT developed and carried out a series of tests at its Structures Research Laboratory at Innovation Park, Tallahassee. The results are discussed in the following report.

1.2 OBJECTIVES AND SCOPE OF STUDY

The objectives of this study are to identify the optimum embedment length (development length) required to develop the ultimate flexural strength of a pile without any slip, to evaluate the development length by the ACI and AASHTO code equations, to

determine the mode of failure and to compare the experimental results with the analytical predictions.

1.3 TRANSFER OF PRESTRESSING FORCE

1.3.1 End Regions

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. In post tensioned concrete, bond is provided by grouting and the full compressive force is transferred to the concrete by means of end anchorages and bearing plates.

When a pretensioned beam is subjected to shear, additional bond stresses are developed. In order to prevent failure, it is necessary to calculate the level of bond stress due to loading and other effects and the maximum bond resistance which can be developed between the steel and concrete. 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, when the flexural capacity of the member is developed, is termed the "development length".

When a prestressing tendon is stressed, the elongation of the tendon is accompanied by a reduction in the diameter due to Poisson's effect. Upon release of the tendon, its diameter tends to return to the original value. This phenomenon is most pronounced at the ends of the member where little or no restraint exists, and is generally regarded as the primary factor that influences bonding of pretensioned wires to concrete. The force in the tendon is zero at the extreme end, and attains a maximum value at some distance from the end of the member. Therefore, over this transfer length, there is a gradual decrease in the diameter of the tendon, which assumes a slight wedge shape over the transfer length. This is referred to as the "Hoyer Effect" after the German engineer E. Hoyer, who was one of the first investigators to . propose this theory. Hoyer, and others more recently, have used elastic theory to calculate the transfer length as a function of

the values of Poisson's ratio for steel and concrete, the moduli of elasticity of steel and concrete, the diameter of the tendon, the-coefficient of friction between the tendon and the concrete, and the initial and effective stresses in the tendon[14]. Laboratory studies of transfer length have indicated a reasonably close agreement between theoretical and actual values. However, there can be wide variation in values of transfer length due to varying properties of concrete and steel, and the surface conditions of the tendons, which considerably affect the coefficient of friction between the two materials.

There is reason to believe that the configuration of a sevenwire strand (i.e., 6 small wires twisted about a slightly larger central wire) results in very good bond characteristics. It is believed that the Hoyer Effect is partially responsible for this. However, the relatively large surface area of a strand and its twisted configuration are also believed to have a significant effect on mechanical bond.

There has been considerable research on the magnitude of transfer length under both laboratory and production conditions. The following significant conclusions are drawn from this research.,

(1) The level of bond that can be obtained between clean three or seven-wire strands and concrete, renders such

reinforcement suitable for the majority of pretensioned concrete elements.

(2) Members that are subject to high moments near their ends, such as short cantilevers, require special consideration.

(3) Clean smooth wires of small diameter are also adequate for use in pretensioning; however, the transfer length for tendons of this type (expressed as a multiple of the diameter) can be expected to be approximately double that for seven-wire strands. (4) Under normal conditions, the transfer length for clean seven-wire strands can be assumed to be equal to 50 times the diameter of the strand.

(5) The transfer length of tendons can be expected to increase from 5 to 20% within one year after release as a result of relaxation. Also, due to relaxation, a small length of tendon (about 3 inches) at the end of a member, can be expected to become completely unstressed over time.

(6) The transfer length of tendons released suddenly by flame cutting or with an abrasive wheel, can be expected to be from 20 to 30% greater than tendons that are released gradually.

(7) Hard non-flaky surface rust, and surface indentations effectively reduce the transfer lengths required for strand and some forms of wire tendons.

(8) Concrete compressive strengths between 1500 and 5000 psi do not have a significant effect on transfer length except for strands larger than 1/2 inch, in which case larger transfer lengths are required for concrete having a strength less than 3000 psi.

(9) Except in very unusual conditions, it would seem prudent not to release pretensioned tendons, until a concrete compressive strength of at least 3000 psi is attained. Higher concrete strengths may be required at release of strands larger than 1/2" diameter.

(10) The degree of compaction of the concrete at the ends of pretensioned members is extremely important if good bond and, consequently, short transfer lengths are to be obtained.

(11) There is little if any reason to believe that the use of end blocks improves the transfer bond of pretensioned tendons, other than to facilitate the placing and compacting of the concrete at the ends. Hence, the use of end blocks is considered unnecessary in pre-tensioned beams, if sufficient care is given to placing and compaction of concrete.

(12) The presence of lubricants and dirt on the surface of tendons has a detrimental effect on the bond characteristics of tendons.

The length of a pretensioned member should be such as to

prevent the overlapping of the flexural bond region and the transfer bond zone. The ACI Code limits the design ultimate stress in pretensioned strands to a level at which some bond slippage might be expected. If inadequate development length is provided, ultimate strength will be governed by bond rather than by flexure.

Bond slippage of strands occurs in three stages, namely (a) progressive bond slip begins in the vicinity of flexural cracks, (b) general bond slip is initiated along the entire development length, and finally (c) the mechanical interlock between the strand surface and the concrete is destroyed. Kaar and Magura ([13]) pointed out that mechanical interlock is adequate to maintain considerable strand stress even after extensive bond slip has occurred. In many cases the stress in the strand reduces after general bond slip occurs, but the stress is not reduced to zero as one might expect. Thus, the final effect of inadequate development length may be a premature flexural failure at a reduced strand stress, - corresponding to a final bending moment less than the computed ultimate strength in flexure.

1.3.2 Intermediate Regions

In order to determine the bond stress existing between concrete and the tendons, two stages have to be considered: before and after cracking of concrete. Prior to cracking of concrete,

bond stress can be calculated using conventional elastic analysis. It appears that a cracked member, which is uncracked at service load, will experience no problems related to the flexural bond stress. Thus, the current ACI code, does not require a check on flexural bond.

After cracking, the calculation of bond stress in a member becomes more complicated. The magnitude of the bond stress changes suddenly at the crack locations due to the abrupt transfer of stress from concrete to steel at these locations. The results of analyses based on reasonable assumptions, do show that there is a significantly higher bond stress in the regions adjacent to cracks. However, results obtained from laboratory testing of beams, or from testing of actual structures, indicate that there is no problem with this high flexural bond stress. The occurrence of local bond failure is not significant in the overall safety or serviceability of a beam. Special attention should however be given to members_ that are subject to fatigue loading, since any cracking at service load renders the bond problem more serious. `

The ACI code deals with bond in both reinforced and prestressed concrete beams in terms of development of reinforcement . rather than bond stress. Most prestressed concrete beams are designed to be uncracked at the service load. The above discussion

shows that flexural bond stress is very low at this stage, and the member need not be checked for serviceability. Design will therefore be generally based on development length as discussed below.

The bond between steel and concrete at the ends of members is considerably different from that at intermediate length of a beam, where the bond stress is produced by external shear or by the existence of cracks. Where there are no cracks and no external shear force, the bond stress is zero.

The nature of transfer bond at the end of a member is entirely different from that of flexural bond produced by shear or cracking. At intermediate points along a beam, the bond stress is resisted by adhesion between steel and concrete, aided by mechanical resistance provided by corrugations in the steel when deformed bars are used. At end anchorages, the prestressed tendons almost always slip and sink into the concrete at transfer. This slippage destroys most of the adhesion over the length of transfer and part of the mechanical resistance due to the corrugations. This results in the bond stress being resisted mainly by friction between steel and concrete.

In Figure 1, at end A, the tendon will have zero stress, immediately after transfer, and its diameter will be restored to

the original unstressed diameter. At the inner end, B, of the transfer length, the tendon will develop almost full prestress, and, owing to Poisson's ratio effect, its diameter will be smaller than that in the unstressed state. Thus, along the length of transfer, between points A and B, there is an expansion of the tendon diameter, which results in radial pressure being developed between the steel and the surrounding concrete. The frictional force resulting from such pressure serves to transmit the bond stress between steel and concrete. In other words, a sort of wedging action takes place within the length of transfer.

Hoyer[8] has shown that the length of transfer varies directly with the diameter of tendon and inversely with the coefficient of friction.
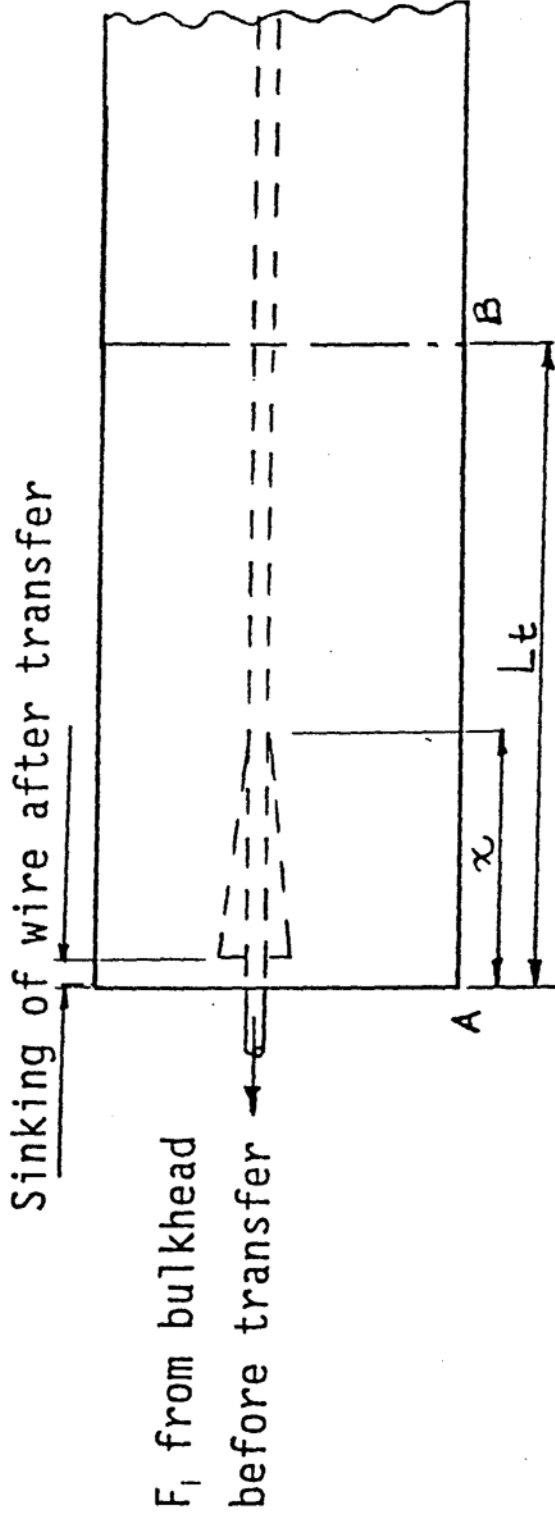
Figure 1 — Prestress Transfer at end of Pretension Beam.

CHAPTER 2

LITERATURE REVIEW

2.1 Bond Parameters

Since the initial bond studies by Hoyer[8], more than thirty such investigations have been reported in the literature[18]. Most of the earlier studies dealt with transfer length of small wires of different sizes either plain, twisted, crimped, indented, or deformed. Bond studies in the United States and Great Britain have dealt mainly with multi-wire strands. These include studies by Base[4] in England, and by the Portland Cement Association[14], and Anderson and Anderson[2] in the United States.

From these tests[18] it has been concluded that the most significant parameters affecting transfer length of prestressing steel are:

* type of steel (e.g., wire, strand).

* steel size (diameter).

* steel stress level.

* surface condition of steel - clean, oiled, rusted.

* concrete strength.

* type of loading (e.g., static, repeated, impact).

* type of release (e.g., gradual, sudden (flame

cutting, sawing)).

* confining reinforcement around steel (e.g.,
   helix or stirrups).

* time-dependent effects.

* consolidation and consistency of concrete
   around steel, and.

* amount of concrete cover around steel.

It is generally agreed that transfer length is longer for larger steel sizes, higher prestress levels, and lower concrete strengths. Also, strands develop some mechanical bond with concrete in addition to friction; thus the transfer lengths of strands are shorter than those of smooth wires of comparable diameter.

If repeated loading is applied outside the transfer zone, no significant effect on the transfer length is observed. However, if applied within the transfer zone, repeated loading can cause. early bond failure if a crack develops within or just outside the transfer length. The use of some reinforcement to'resist the bursting stress near the end of the prestressing steel reduces the transfer length, but the effect is not significant.

In most test specimens[18] there was observed to be a significant increase in load carrying capacity between the point

at which first bond slip occurred and final bond failure. The difference in load carrying capacity was assumed to be due to mechanical interlock of the strand.

Based on the results of tests on thirty-six pretensioned hollowcore units, Anderson and Anderson[2] concluded that the existing ACI Code requirement for development length was adequate provided, that the free-end slip of the strand, upon transfer of prestress, does not exceed an empirical value of approximately 0.2 times the strand diameter.

2.2 Evaluation of ACI Provisions

Current ACI Specifications deal with development of prestressing strands only, reflecting current practice in North America.

Early investigations on the nature of bond were conducted in the 1950's 7,10,11. 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-Kaar tests[7] were performed on specimens prestressed with clean 1/4, 3/8, and 1/2 inch diameter strands, and having a wide range of steel percentages. The strands were released slowly, instead of being cut by flame or saw. In most of the specimens there were significant increases in the load

carrying capacity between the point at which first bond slip was detected by strain gauges 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[15].

Results of tests performed by Kaar, LaFraugh, and Mass[12] greatly added to the knowledge concerning transfer length. Tests were performed on members with varying strand diameters and concrete strengths. The results 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 irrespective of concrete strength.

In recent years, several researchers have proposed new equations for transfer and development lengths. Martin and Scott,[15] in a statistical evaluation of the early test performed by Hanson and Kaar[8], proposed the following expressions:'
For l_x less than or equal to 80 d_b:

$$f_{ps} < l_x/(80d_b(135/d_b^{1/6} + 31)) \qquad (4)$$

where l_x is the distance from the end of the member to the section under consideration, in inches.

and for l_x greater than 80 d_b:

$$f_{ps} < (135/db^{1/6} + 0.39\ l_x/d_b) \qquad (5)$$

It was specified that in no case should f_{ps} be greater than that given by Eq. (18-3) of ACI Code 318-83 or that obtained from a determination based on strain compatibility.

The above expressions provide a n approach to designing precast, pretensioned units for spans too short to provide an embedment length that will develop the full strength of the strand.

Martin and Scott[3] 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 1/4, 3/8, and 1/2 inch diameter strands respectively. These values are considerably higher than those specified by the current ACI Code.

On the other hand, based on the results of a test program of thirty six pretensioned hollow-core units, Anderson and Anderson[2] concluded that the current ACI Code requirement on the development length is adequate.

Zia and Mostafa[18], in a comprehensive study of all past research, proposed the following expressions for transfer length:

$$l_t = 1.5f_{si}d_b/f'_{ci} - 4.6 \qquad\qquad (6)$$

$$l_b = 1.25(f_{ps} - f_{se})d_b \qquad\qquad (7)$$

$$l_d = l_t + l_b \qquad\qquad (8)$$

where

f_{si} = stress in prestressing steel at transfer, ksi

f_{ci} = compressive strength of concrete at time of initial prestress, ksi

i_t = transfer length of prestressing strand, in.

l_b = flexural bond length of prestressing strand, in.

Eq. (7) is based on the theoretically derived expression:

$$l_b = (f_{ps} - f_{se})/4u_{ave}$$

where u_{ave} is average bond stress within l_b. Note that in the current ACI Code, it is implied that u_{ave} = 250 psi . Eq. (7)
_ assumes a value of u_{ave} = 200 psi.

The Zia-Mostafa equation for transfer length is applicable for concrete strength 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.

2.3 Industry Survey

As a part of the broader investigation of which this study forms a part, American and Canadian prestressed concrete producers were surveyed about their concerns with ACI Code requirements governing the design and manufacture or precast prestressed elements. One of the questions included in the survey was: "Do the provisions governing the development of prestressing strand (Section 12.9) pose any hardship"? The answers were 10 yes and 29 no's. Of the 10 yes answers, 8 related to doubling the development length of sheathed strands;

1. Section 12.9.3 is too severe (two respondents).

2. Section 12.9.3 does not make any sense. Why should l_d be doubled? Does it make any difference if the strand is debonded in 6 in. (150 mm) length or say 10 ft. (3m) length? Per this section debonding will cause problems in most prestressed members of moderate 20 to 30 ft. (6 to 9 m) length.

3. Doubling the development length for wrapped strands.

4. Seems excessive; otherwise not a problem for our members.

5. For sheathed strand the extended bond development is too great based on our observations. Otherwise, I do not consider the 20

strand development provisions a "hardship".

6. Masking is a real problem if complying with Section 12.9.3

Other comments claiming hardship were:

1. On very short span members the development length creates a theoretical problem in flexural strength.

2. Difficulties are experienced on heavily loaded short spans.

3. Development length is long and poses some difficulties when holes are cut in hollow-core floor slabs. Research to prove that the ultimate tensile strength of strand can be developed in a shorter length would be welcome. Not a problem insofar as double tees are concerned.

4. Section 12.9.1 of ACI 318-83 needs 170 d_b development length.

5. The term $(f_{ps} - f_{se})d_b$ in the Code equation for development length is excessive. However, this requirement is generally a problem in short simple span members in which case the strand diameter must be reduced. Experience with railroad ties seems to indicate the conservative nature of this requirement.

6. Our experience shows that the prevention of splitting during detensioning merits the use of short length ((95)) (5 ft)

(1.5 m) shear reinforcing in the ends in the development

length region regardless of Code provisions.

7.   Generally, double tees have long spans and
development is not a problem.

2.4 CURRENT ACI CODE PROVISIONS

The current ACI provisions for development length of
prestressing strand are contained in Section 12.9 of ACI
318-83. The provisions are as follows:

Section 12.9.1 - Three or seven-wire pretensioning
strand shall be bonded beyond the critical section for a
development length, in inches, not less than:

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

where

f_{ps} = stress in prestressed reinforcement at nominal strength,
ksi

f_{se} = effective stress in prestressed reinforcement (after .
allowance for all losses), ksi

d_b = nominal strand diameter, in.

The expression in parentheses is used as a constant without
units. Section 12.9.2 - Investigation may be limited to
cross sections nearest each end of the member that are
required to develop full design strength under specified
factored loads.

Section 12.9.3 - Where bonding of a strand does not extend to the end of member, and design includes tension at service load in precompressed tensile zone as permitted by Section 18.4.2, development length specified in Section 12.9.1 shall be doubled. The equation for the development length can be rewritten as follows:

$$l_d = (f_{se}/3)d_b + (f_{ps} - fse)d_b$$

Where l_d and d_b are in inches, and f_{ps} and f_{se} are in kips per square inch.

The first term represents the transfer length of the strand, i.e., the distance over which the strand must be bonded to the concrete to develop the effective prestress, f_{se}, in the strand. The second term represents the additional length over which the strand must be bonded so that a stress, f_{ps}, may develop in the strand at nominal strength of the member. The variation of strand stress along the development length of the strand is shown in Figure 2.

The effective steel stress, f_{se}, obviously 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.

Similarly, in the expression for flexural bond length, in the above equation, 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 110 nominal strand diameters for 250 ksi grade strand, assuming f_{si} = 0.7fpu and f_{ps}=0.8fpu (where f_{pu} is the specified tensile strength of prestressing strand, ksi).

Similarly, for 270 ksi grade strand, the transfer length would be 51 strand diameters, and the flexural bond length would be 119 strand diameters. Note that the value of 50 strand diameters is mentioned as the assumed transfer length in Section 11.4.3 of ACI 318-83.
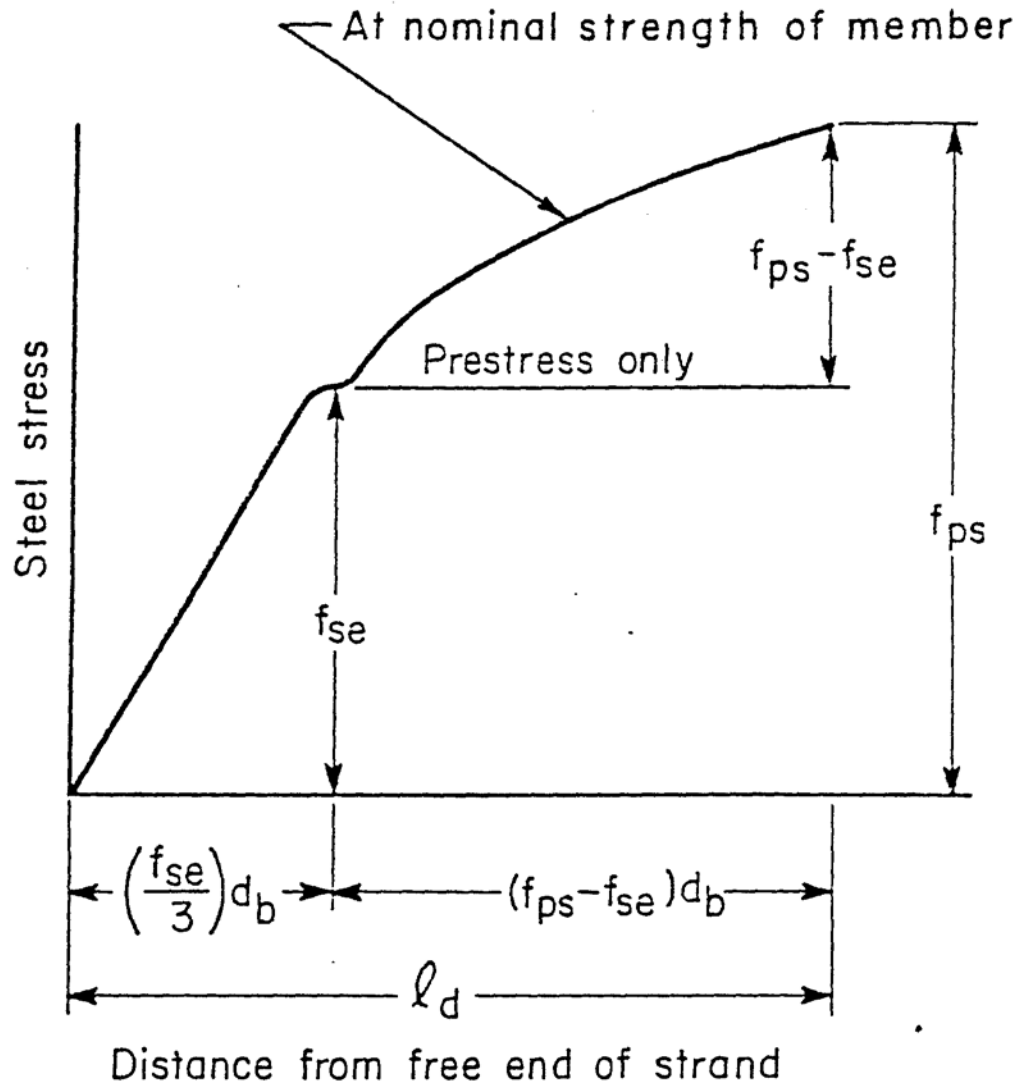
Fig. 2 – Variation of steel stress with distance from free end of strand

CHAPTER 3

EXPERIMENTAL INVESTIGATION

3.1 GENERAL

The test program was conducted to investigate the effect of embedment length of a pile in a pile cap on the ultimate moment strength of the pile. The main objective of this study was to identify the optimum embedment length (development length) required to develop the ultimate moment strength of a pile without any slip, and to compare this with the value of the development length, l_d, specified in the ACI and AASHTO Codes.

In order to minimize the number of specimens to be tested in the current research program, it was decided that parameters that had not been found to affect development length significantly would not be varied.

3.2 Test Program

Nineteen (19) 14" square prestressed concrete piles were tested in this study. Seventeen (17) of the test specimens were prestressed with 8-1/2" diameter prestressing strands.' All piles contained by 5 gage spiral reinforcement diameter prestressing, and the remaining two (2) were prestressed with 12-7/16" diameter prestressing strands. A summary of the test program is presented in Table 1. Figure 3 shows a typical pile cross section and

BEAM NO.	NO. AND SIZE OF STRANDS	SECTION	EMBEDMENT LENGTH (in.)	CALCULATED DEVELOPMENT LENGTH (ACI)	f'_c (ksi)	STRAND SLIP
A-4	8-1/2"	END	36	47.2	7.10	NO
B-2	8-1/2"	END	36	46.9	5.84	NO
B-7	8-1/2"	INT.	36	47.1	6.59	YES
B-9	8-1/2"	INT.	36	47.1	5.59	YES
A-1	8-1/2"	END	42	49.8	6.70	YES
A-5	8-1/2"	END	42	49.8	6.45	NO
A-6	8-1/2"	END	42	49.0	5.98	YES
B-1	8-1/2"	END	42	50.1	7.80	YES
B-3	8-1/2"	END	42	49.5	6.48	NO
B-8	8-1/2"	INT.	42	50.0	6.48	YES
A-2	8-1/2"	END	48	52.7	6.96	YES
B-4	12-7/16"	END	48	52.5	6.50	NO
B-5	8-1/2"	INT.	48	52.9	7.76	NO
B-6	12-7/16"	INT.	48	52.7	6.50	NO
C-3	8-1/2"	INT.	48	52.7	6.50	NO
C-4	8-1/2"	END	48	52.7	6.50	NO
A-3	8-1/2"	END	60	58.9	7.20	NO
C-1	8-1/2"	INT.	60	58.5	6.50	NO
C-2	8-1/2"	END	60	58.5	6.50	NO

TABLE 1   DETAILS OF TEST PROGRAM

reinforcement details.

The test specimens were obtained by cutting sections from eighty (80) feet length 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 Figure 4. The end sections were provided with more spiral reinforcement than the middle sections. Each test specimen was approximately 12 feet in length. Sections cut from the ends and from the middle of each pile were tested to study the effect of the shear confinement on the development length.

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

3.3 Test Setup

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 steel sections. Figure 5 shows the details of the test frame which was anchored to the 3 foot thick reinforced concrete floor.
A hydraulic jack, supported on the floor, was used to apply the

ELEVATION OF 14" X 14" PILE

Spiral Tie Spacing

5 Turns 1" Pitch

16 Turns – 3" Pitch

9" Pitch

16 Turns – 3" Pitch

5 Turns 1" Pitch

Head

Tip

3/4" by 3/4" Chamfer

3/4" by 3" Chamfer

No. 5 Gage Spiral Ties (0.2044" φ)

SECTION A–A

12 – 7/16 φ S.R. – As = 0.115 in². 270K at 21,000# ea. spaced 2.83" ctrs.

7/16 "φ – STRAND PILES

Strands (Typ)

2 3/4" to ℄

1'-2"

SECTION A–A

8 – 1/2" φ L.R.S. – As = 0.153 in². 270K at 29,500# each.

1/2"φ – STRAND PILES

Strands (Typ)

2 1/8" to ℄

1'-2"
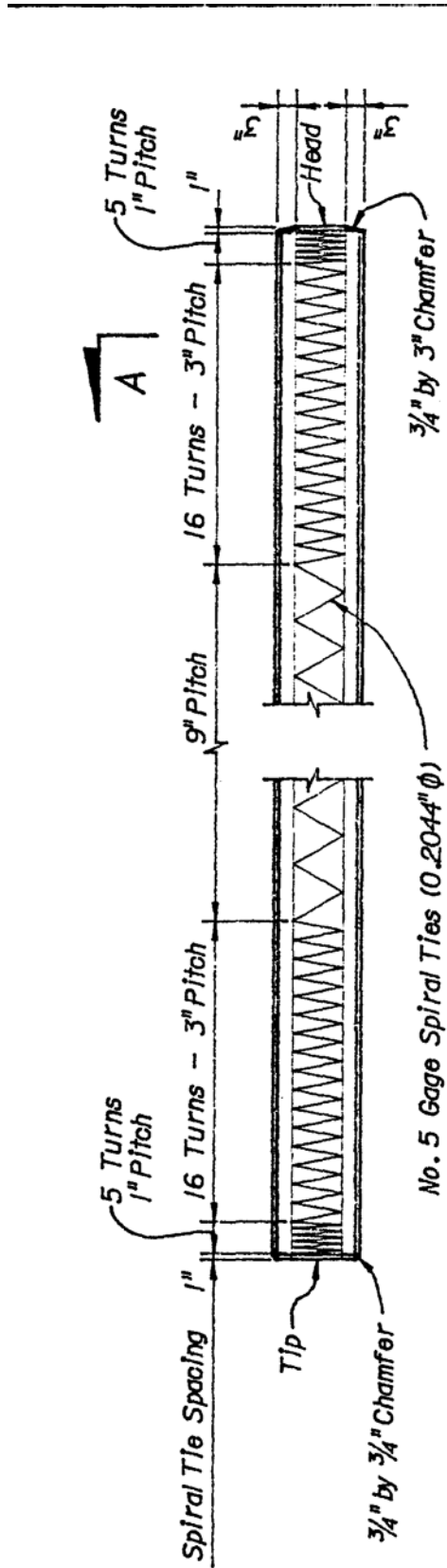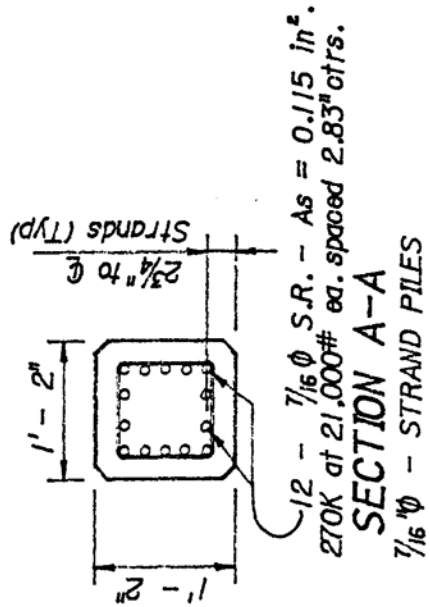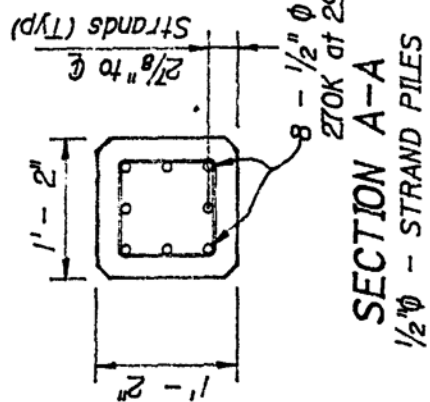
Figure 3   Details of Piles
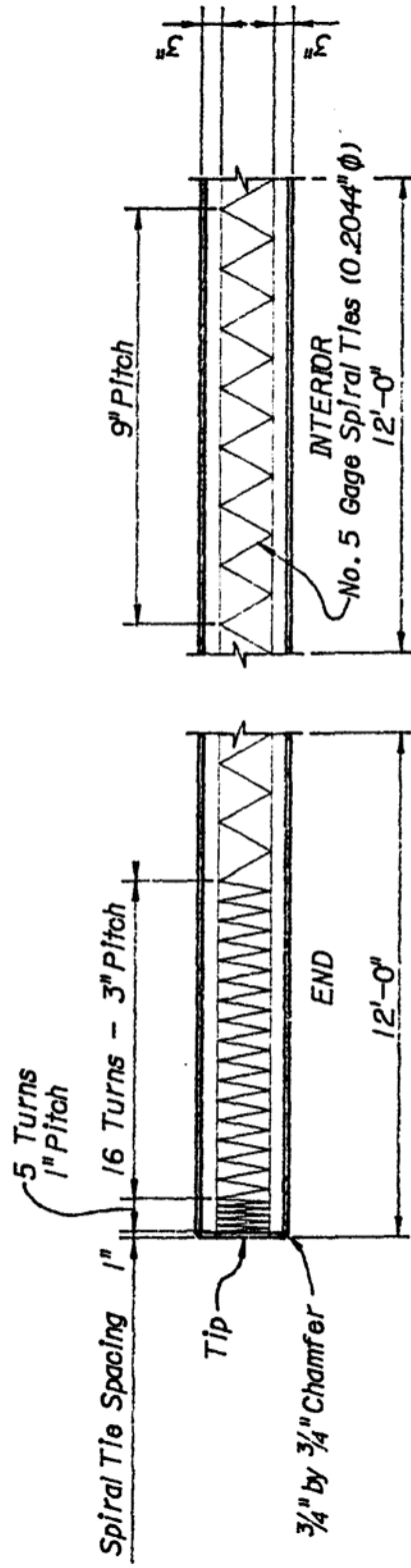
Figure 4 Details of Test Specimens

PLAN

ELEVATION

ELEVATION
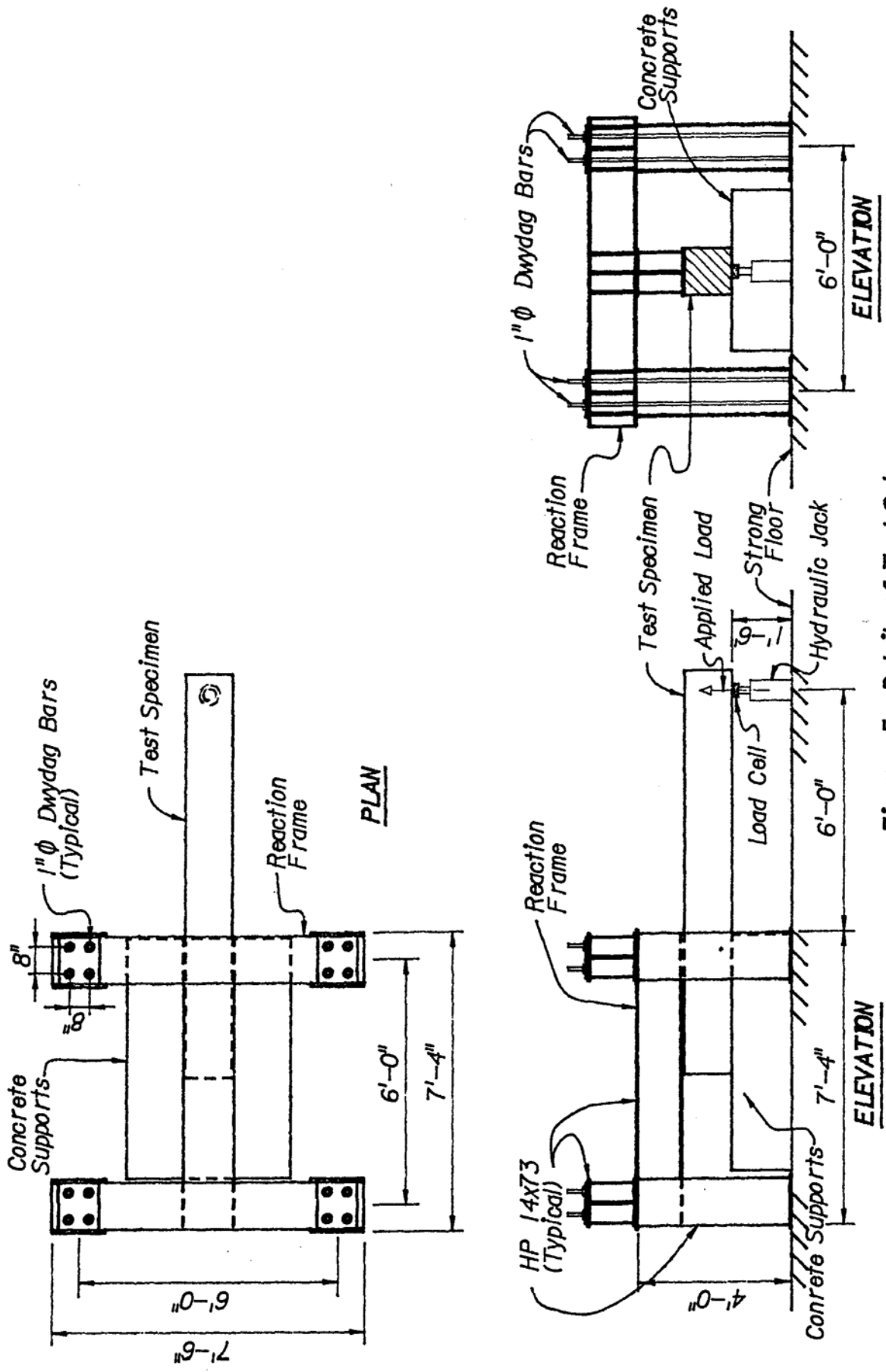
Figure 5 Details of Test Setup

load to the pile at a distance of 6 feet from the face of the supporting frame. The frame provided restraint against translation and rotation in the vertical direction which was felt to be more severe than 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 which result in reduction of the development length. Figure 6 shows a view of the test specimen in the testing frame.

3.4 Instrumentation and Data Acquisition System

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. These observations were made using dial gages, electrical resistance strain gages(ERSG's) and LVDT's(Linear Voltage Differential Transducers).

The electrical resistant strain gages and LVDT's used in the tests were connected to a data acquisition recorded manually.

The strain gages were mounted near the upper side of the pile to measure the compressive strain in the concrete. The LVDT's were mounted near to the lower face so as to measure tensile strain in concrete. Strain gages and LVDTs were respectively mounted 2 3/4"

along the pile length in order to measure the strains at the level of the prestressing tendons. Figures 7 through 10 show the locations of instrumentation on a test specimen: Dial gages were used to measure pile deflection at 2 and 4 feet respectively from the face of the frame. Three horizonal dial gauges or LVDT'S were mounted at the embedded end of the pile to measure any slip in the lower prestressing tendons (see figure 10) . A load cell placed between the hydraulic jack and the pile was used to measure the force applied to the pile.

3.5 Test Procedure

The pile was placed in the test bed, and 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/or crushing of the concrete at the face of the support. Deflection and strain measurements were taken at specified load increments during the test.   -

In order to load a pile in the test frame, the top part of the frame was first removed, and the pile was placed in the frame to satisfy the specified length of embedment. The top of the frame was replaced and secured to the floor. The pile was thus sandwiched between the frame top and the concrete support, and was effectively fixed against translation and rotation at the fixed
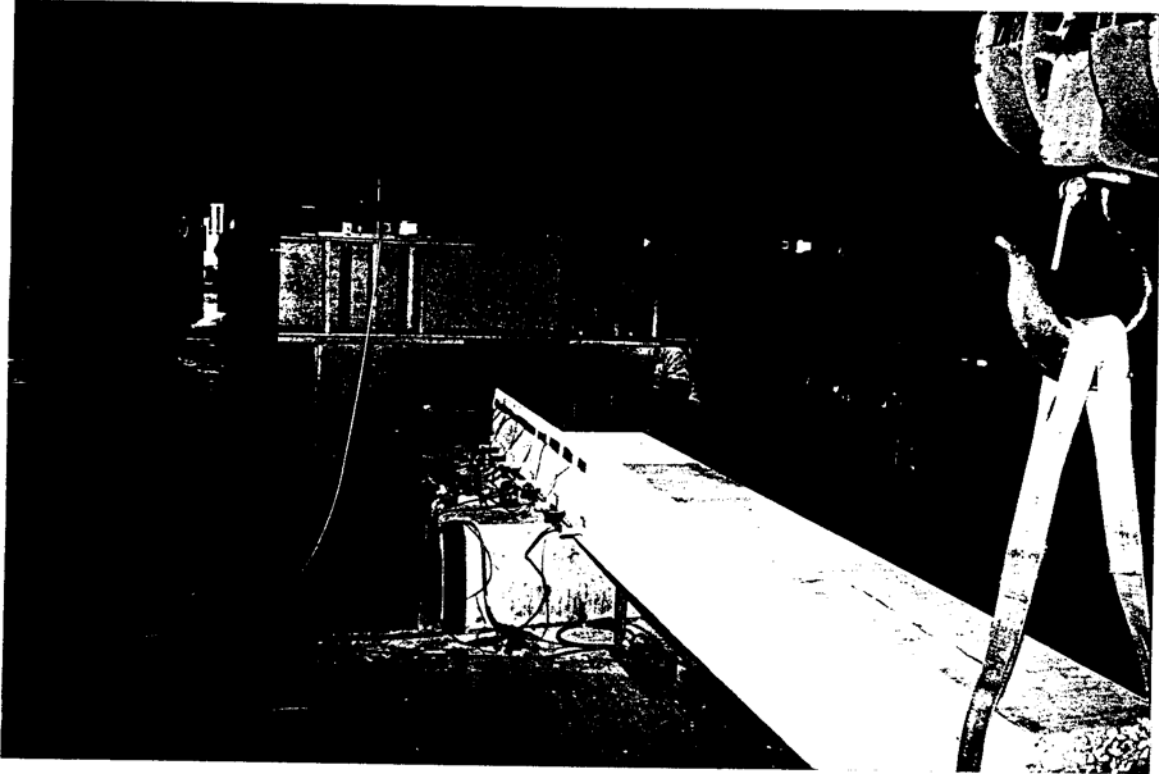
FIGURE 6 (A)
PILE TEST SET UP
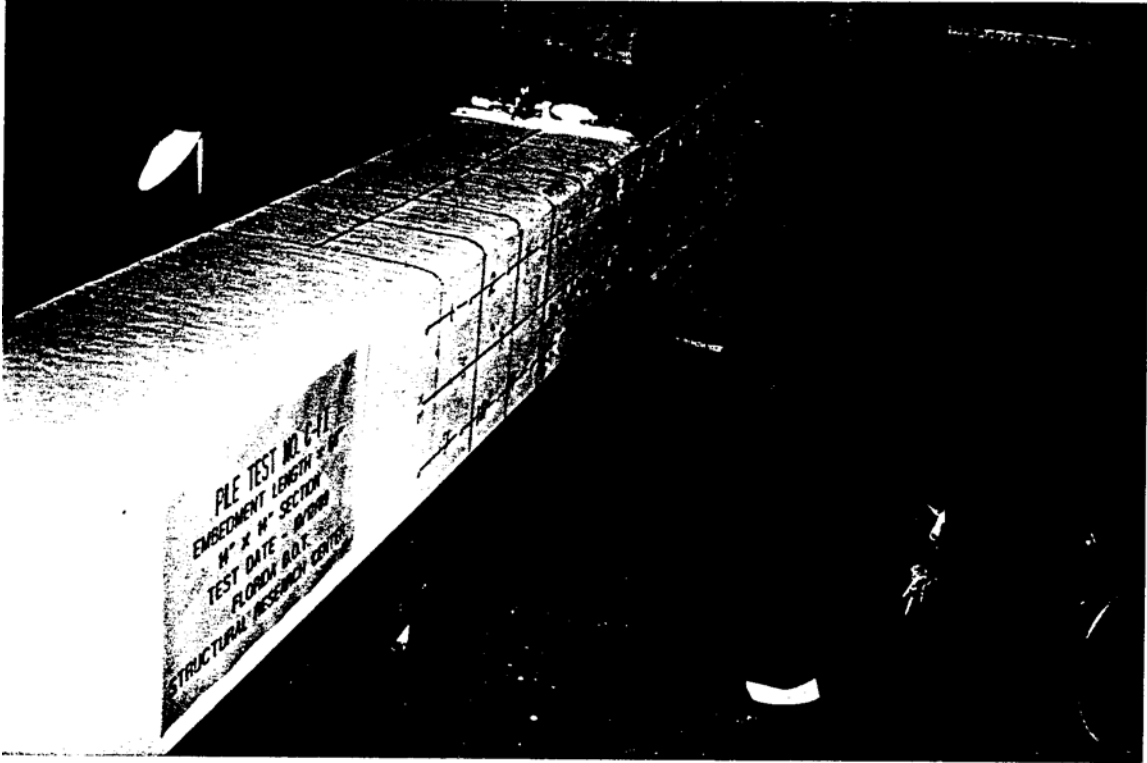
FIGURE 6 (B)
PILE TEST SETUP
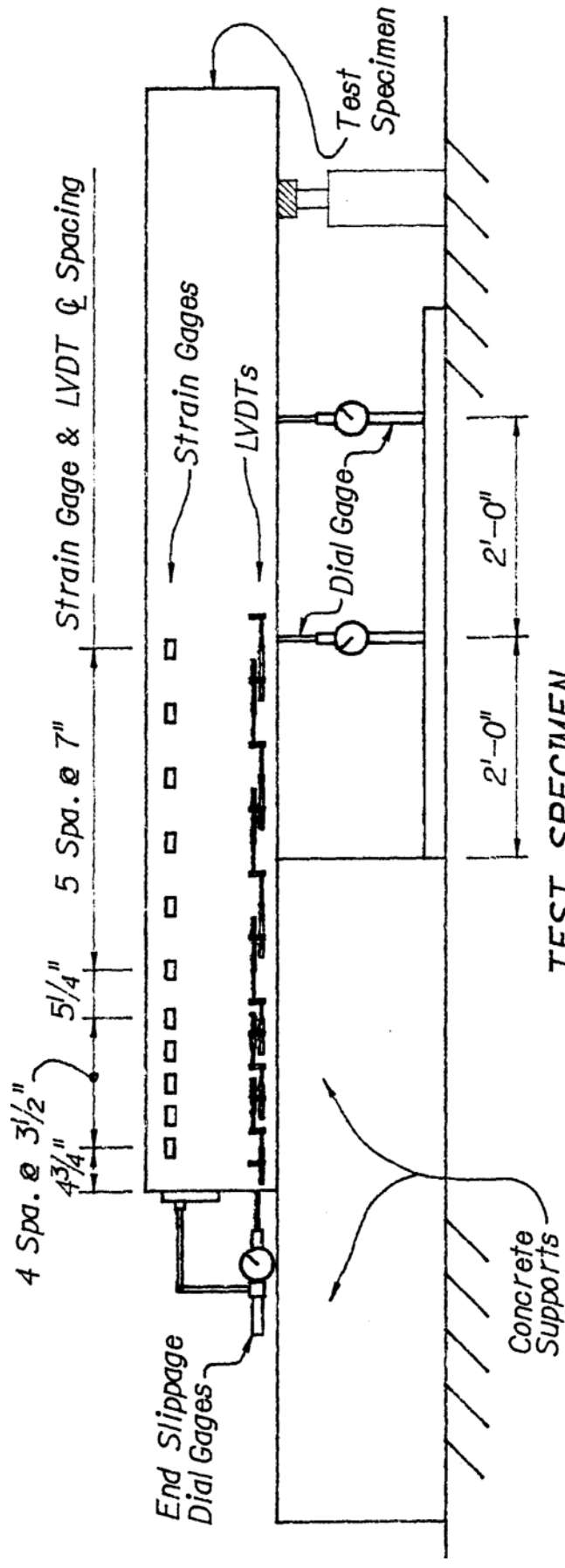
TEST SPECIMEN
(Test Frame Not Shown)
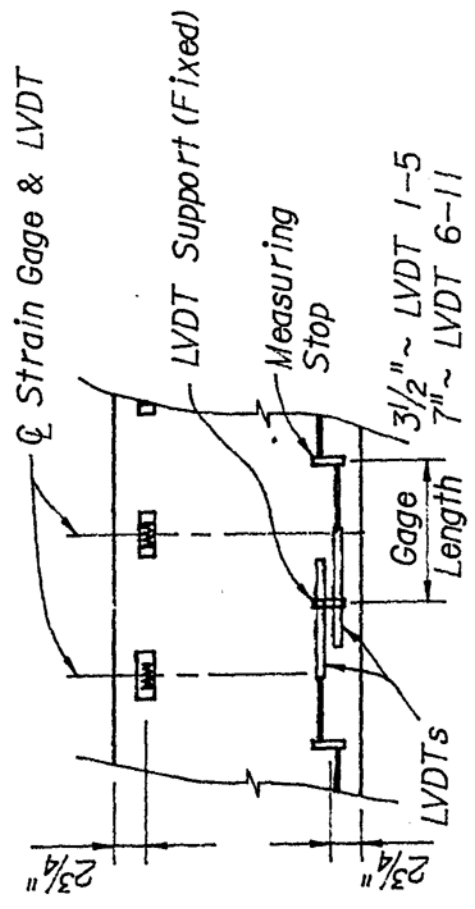
SPACING DETAIL

Figure 7 Location of Instrumentation
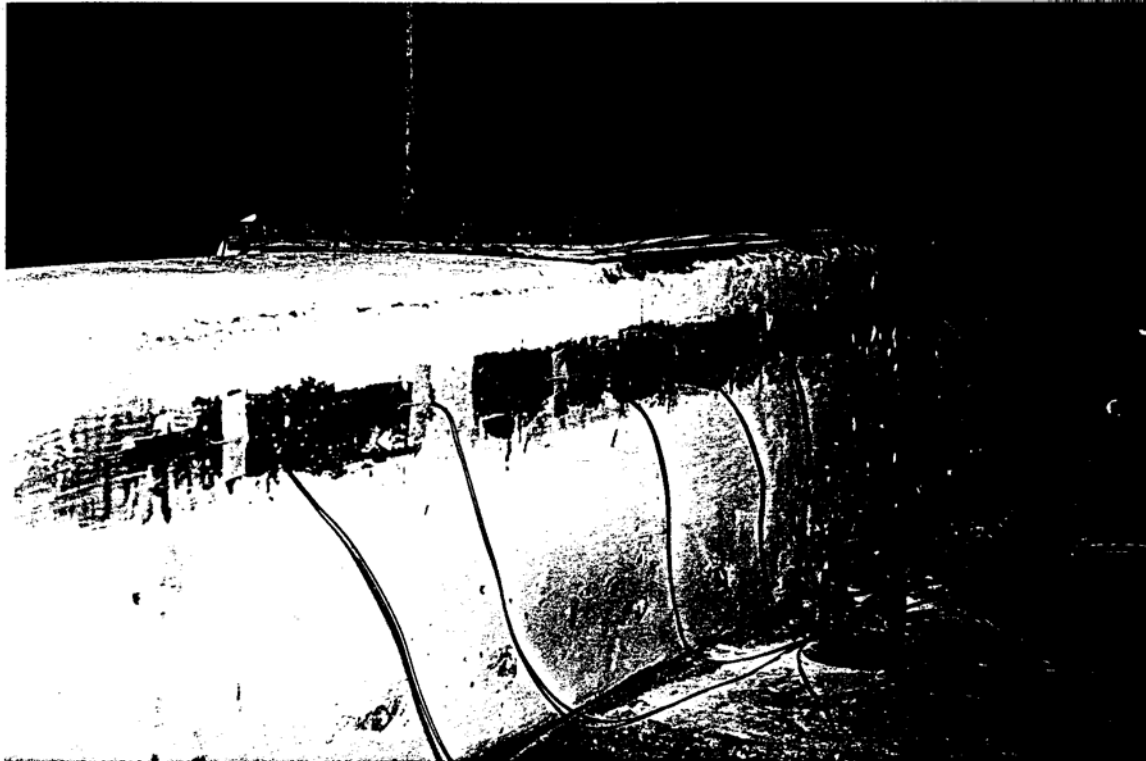
FIGURE 8
LOCATIONS OF STRAIN GAGES (TOP) ALONG PILE
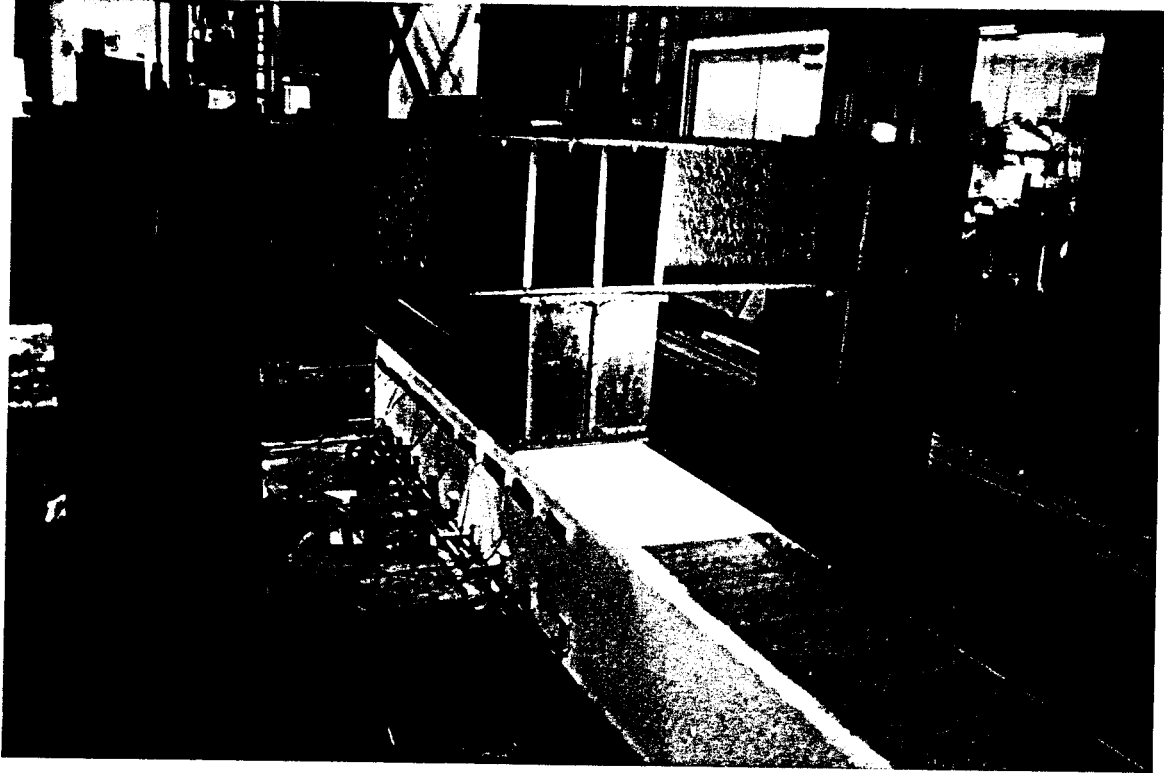
FIGURE 9
LOCATION OF LVDT's ALONG THE PILE
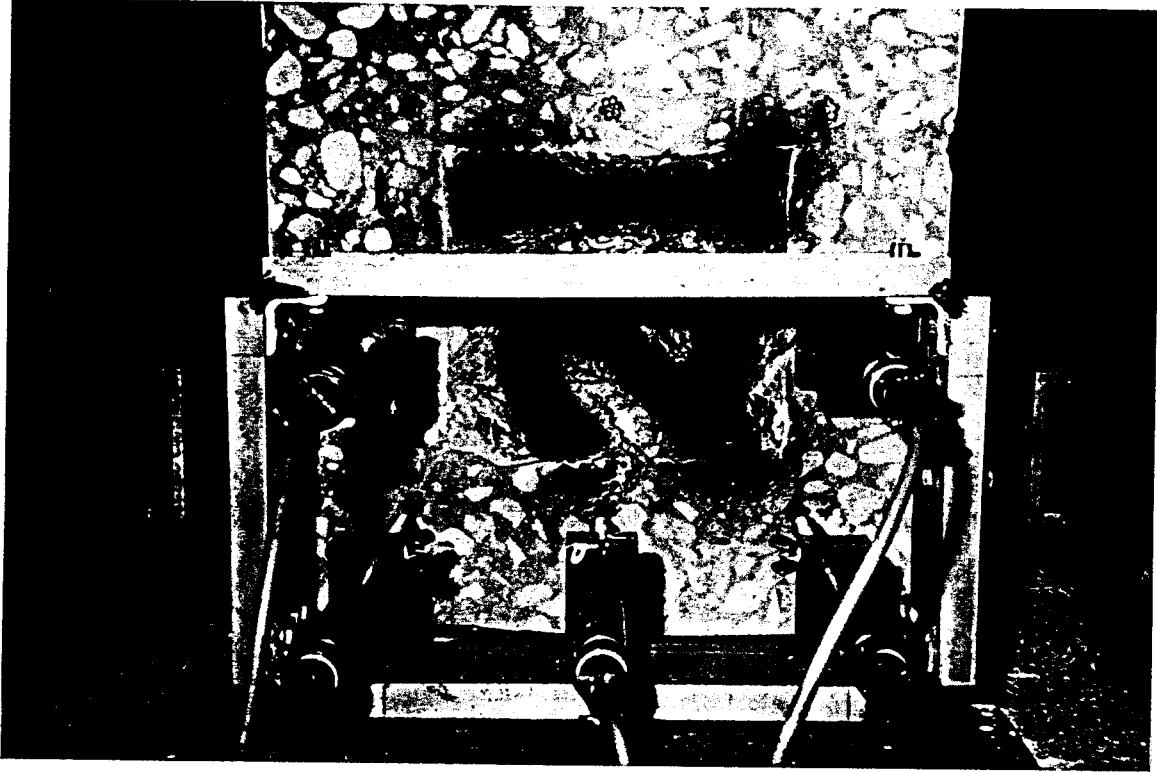
FIGURE 10
TO MEASURE END SLIPPAGE

end. The pile was instrumented with strain gages prior to its being placed in the test frame.

A steel plate was epoxied to the end of the pile in the frame in order to provide a mounting surface for the three horizontal dial gauges that were to measure slip of strand.

The load was applied to the pile after setting and recording of the initial readings of all gages. Loading was applied by means of the hydraulic jack in increments of 3 kips up to 18 kips. Readings of all instruments were taken and recorded at the end of each load stage. The load was then applied in increments of 1 kip up to failure. Cracks were highlighted with a marker in order to follow their development. A total of nineteen piles were tested.

CHAPTER 4

ANALYTICAL STUDY

4.1 GENERAL

A nonlinear material analytical model was used on the computer to analyze the prestressed concrete piles. The program is a modified version of the program PCFRAME, which is based on the finite element analysis approach.

4.2 Description of The Analytical Model

The program uses a numerical procedure to simulate the material, and conducts a geometric nonlinear analysis of plane prestressed concrete frames, including 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 means of an iterative procedure.

As shown in Figure 11, a parabolic approximation was used for the stress-strain relationship for concrete. Biliner and multilinear approximations of stress-strain curves were used for

non-prestressed steel and prestressed steel, respectively.

The program accommodates the different material properties within an element of the structure by using a composite concrete and non-prestressed steel layer system. Concrete and nonprestressed steel are assumed to be perfectly bonded. Each prestressing steel tendon is idealized as a discrete number of elements with a constant force over the length of an element. The eccentricity of the tendon in an element is assumed constant and is taken as the average of the eccentricities at the two ends of the element, namely

$$e = (e_i + e_j)/2$$

as shown in Figure 12.

Pretensioned as well as bonded and unbonded post-tensioned concrete frames can be analyzed by the program. Perfect bond between the concrete and the prestressing steel is assumed for bonded beams.

The models are analyzed for the effects of applied load, by considering the concrete and non-prestressed steel separately from the prestressing steel, and superimposing the results to determine the combined effect of the total load. Figures 13 and 14 show the computer models of the piles for 8-1/2" strand diameter and 127/16" strand diameter respectively.
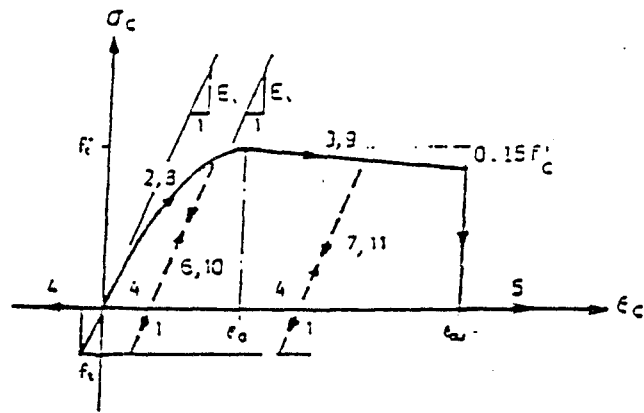
(a) Concrete
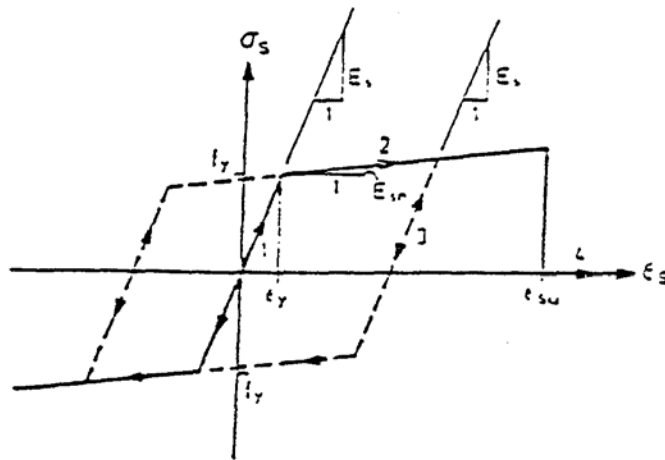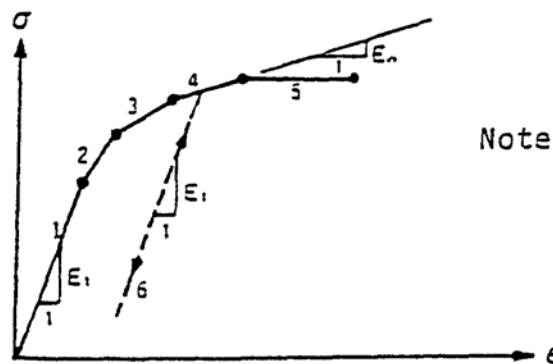
(b) Non-Prestressed Steel

Note: Numbers on the Stress-
Strain Curves refer
to Material Code

(c) Prestressed Steel

Figure 11 - Material Stress-Strain Curves.

43
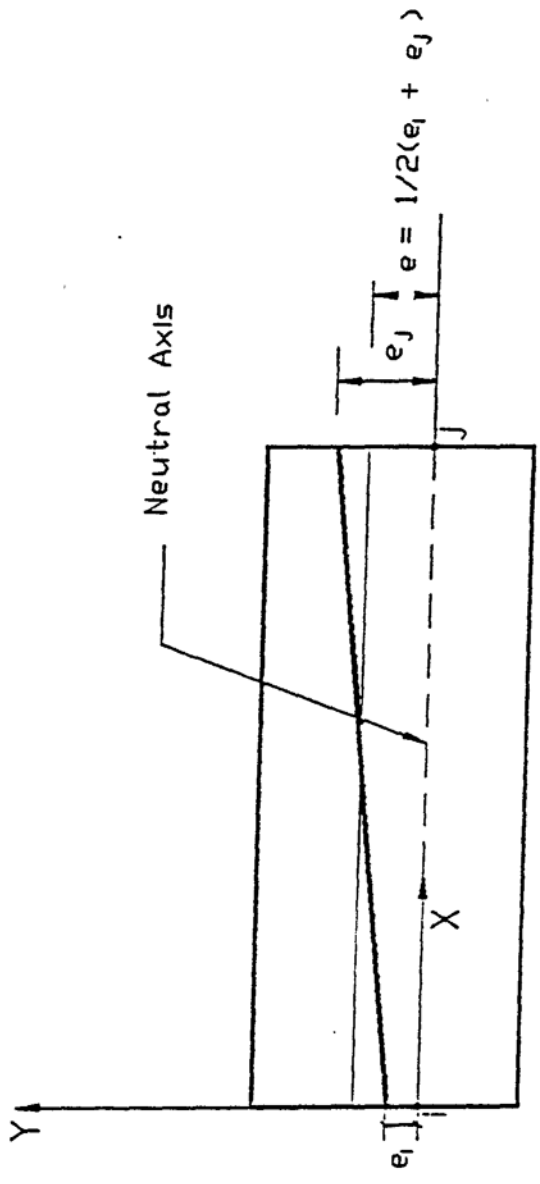
Figure 12 Prestressing steel segment in an Element

COMPUTER MODEL

CONCRETE SECTIONS

SECTION NO.	Y(in)	Area(in)^2
1	6.5	14
2	5.5	14
3	4.5	14
4	3.5	14
5	2.5	14
6	1.5	14
7	0.5	14
8	-0.5	14
9	-1.5	14
10	-2.5	14
11	-3.5	14
12	-4.5	14
13	-5.5	14
14	-6.5	14
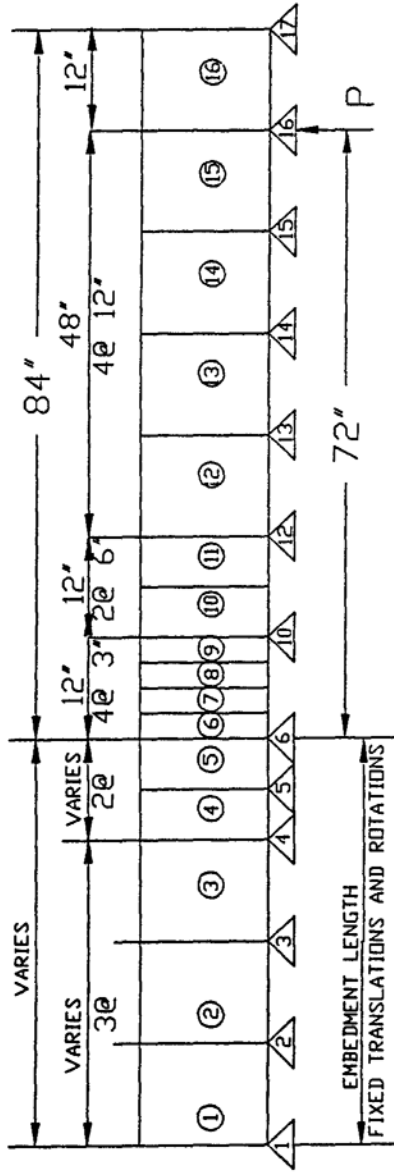
△ NODE NUMBER

○ ELEMENT NUMBER

PILE SECTION

Figure 13- Computer Model for the 8-1/2" Strand Diameter
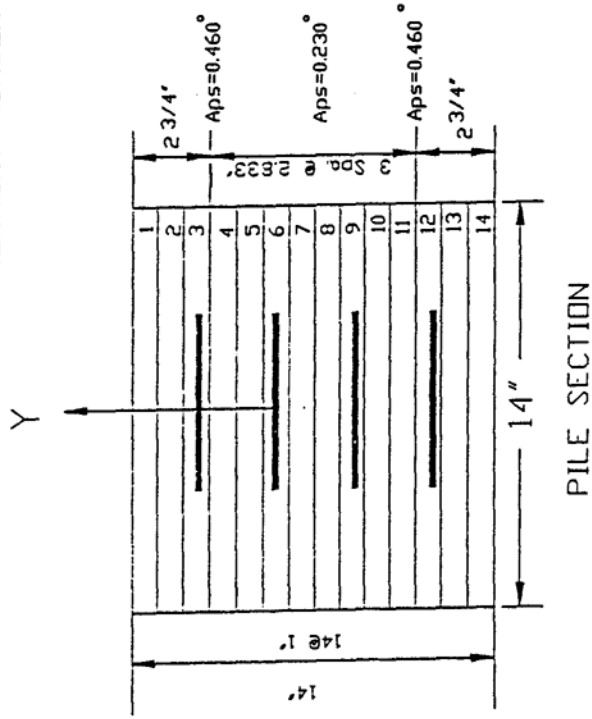
Figure 14— Computer Model for the 12-7/16" Strand Diameter

4.3 Boundary Conditions

The boundary conditions (interface between the model and external support) were, adjusted to satisfy actual boundary conditions. The model was supported vertically and horizontally at points within this embedded section of the pile.

4.4 Material Properties

The values of Young's Modulus used in the model were as follows:

Steel : 28 x 10^6 psi

Concrete : 4.4 x 10^6 psi (at f'$_c$ = 6000psi)

CHAPTER 5

PRESENTATION AND DISCUSSION OF RESULTS

5.1 GENERAL

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 the appendix.

5.2 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 results are summarized in Table 2. The value of the ultimate prestressing steel stress, fps, was found to vary between 242 ksi to 250 ksi. The measured external moment, producing failure was compared to the calculated ultimate load in Table 2.

Typical applied moment-deflection curves for the piles are presented in Figures 15 through 18 for embedment length of 36", 42", 48" and 60" respectively.

These curves show a three stage relation. The first stage represents the precracking (essentially elastic) stage. The second

BEAM NO.	MEASURED ULTIMATE MOMENT (K-ft.)	THEORITICAL ULTIMATE MOMENT (K-ft.)		TEST ACI	f_{ps} AT FAILURE (ksi)	AVERAGE INITIAL BOND STRESS (psi)
		Finite Element	ACI			
A-4	153.0	128.4	130.2	1.175	248.48	675
B-2	150.0	124.2	130.2	1.152	246.74	667
B-7	129.0	127.2	130.2	0.991	248.55	672
B-9	129.0	127.2	130.2	0.991	248.38	672
A-1	135.0	124.2	130.2	1.037	248.70	576
A-5	156.0	124.2	130.2	1.198	248.14	575
A-6	147.0	124.2	130.2	1.129	242.71	563
B-1	130.0	131.4	130.2	1.198	250.70	581
B-3	153.0	127.2	130.2	1.175	246.04	570
B-8	132.6	127.2	130.2	1.018	249.67	579
A-2	126.0	129.6	130.2	0.977	249.23	505
B-4	141.0	133.2	138.2	1.020	248.13	503
B-5	147.0	129.6	130.2	1.129	250.61	508
B-6	138.0	133.2	138.2	1.000	249.76	506
C-3	140.4	128.4	130.2	1.078	248.90	405
C-4	142.2	128.4	130.2	1.092	248.90	405
A-3	144.0	127.2	130.2	1.106	249.64	405
C-1	144.0	124.2	130.2	1.106	249.64	405
C-2	135.0	124.2	130.2	1.037	249.64	405

TABLE 2   TEST RESULTS

PILE B-9

36" EMBEDMENT

APPLIED MOMENT VS DEFLECTION
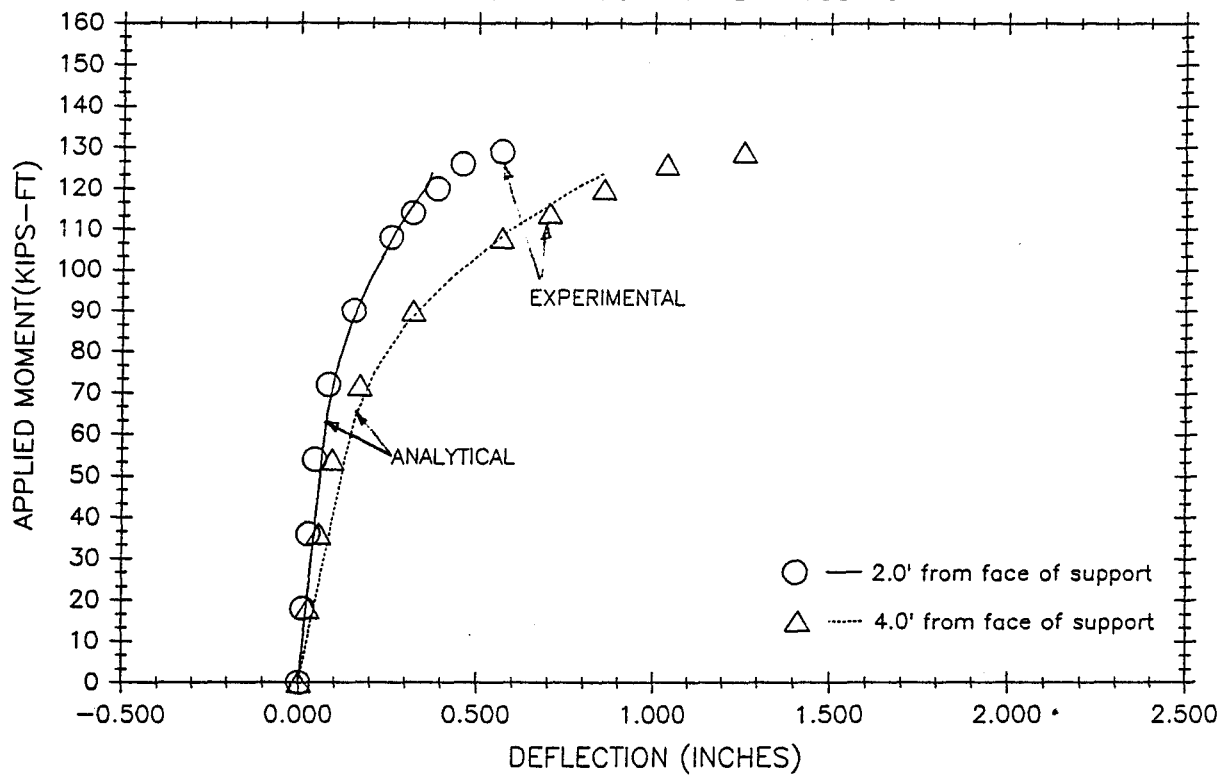
Figure 15- Analytical and Experimental Curves

PILE B—9

36" EMBEDMENT

APPLIED MOMENT VS DEFLECTION
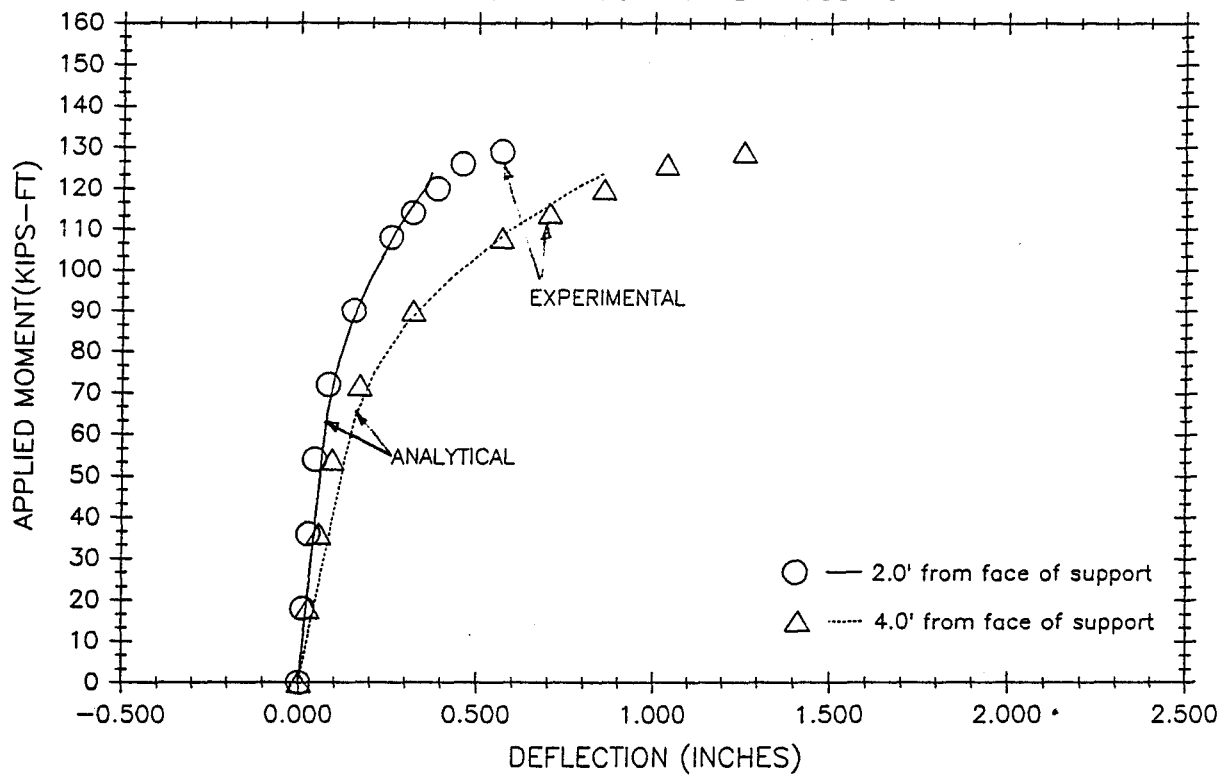
Figure 15— Analytical and Experimental Curves

PILE A-5

42" EMBEDMENT
APPLIED MOMENT VS DEFLECTION
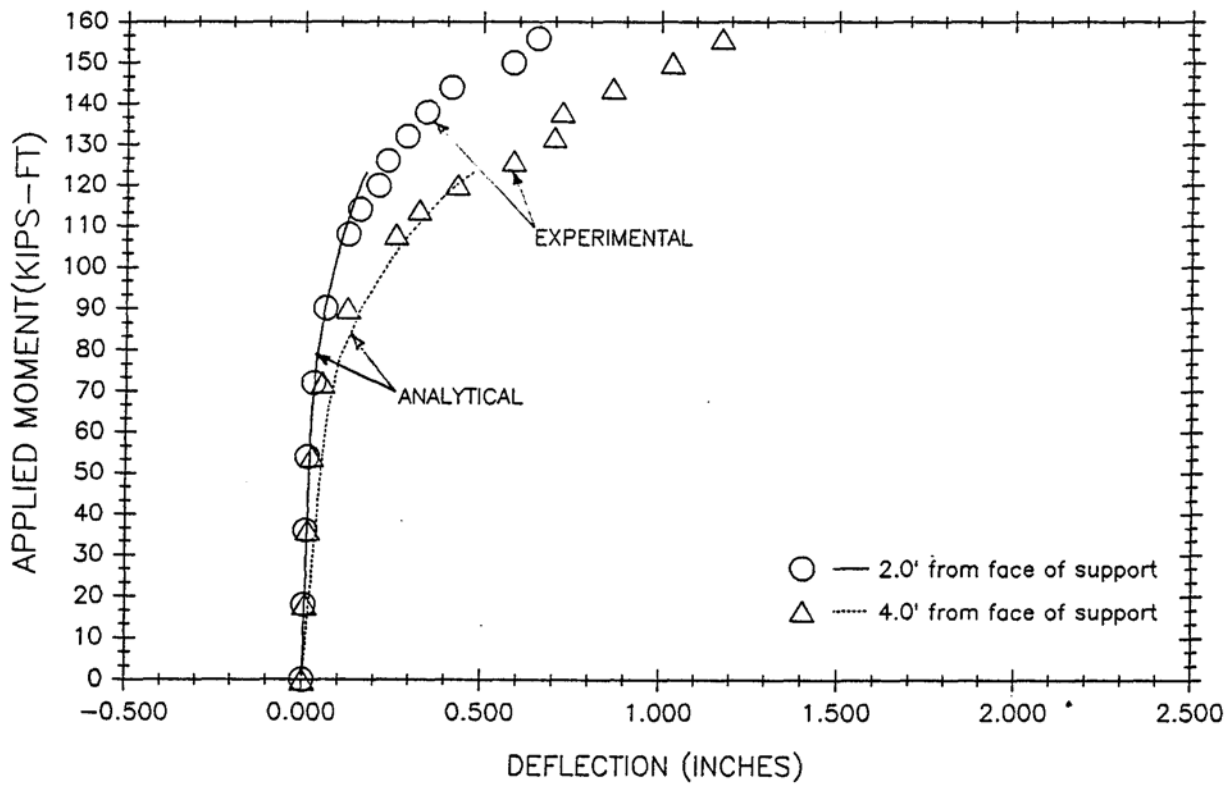
Figure 16- Analytical and Experimental Curves

PILE C−2

60" EMBEDMENT

APPLIED MOMENT VS DEFLECTION
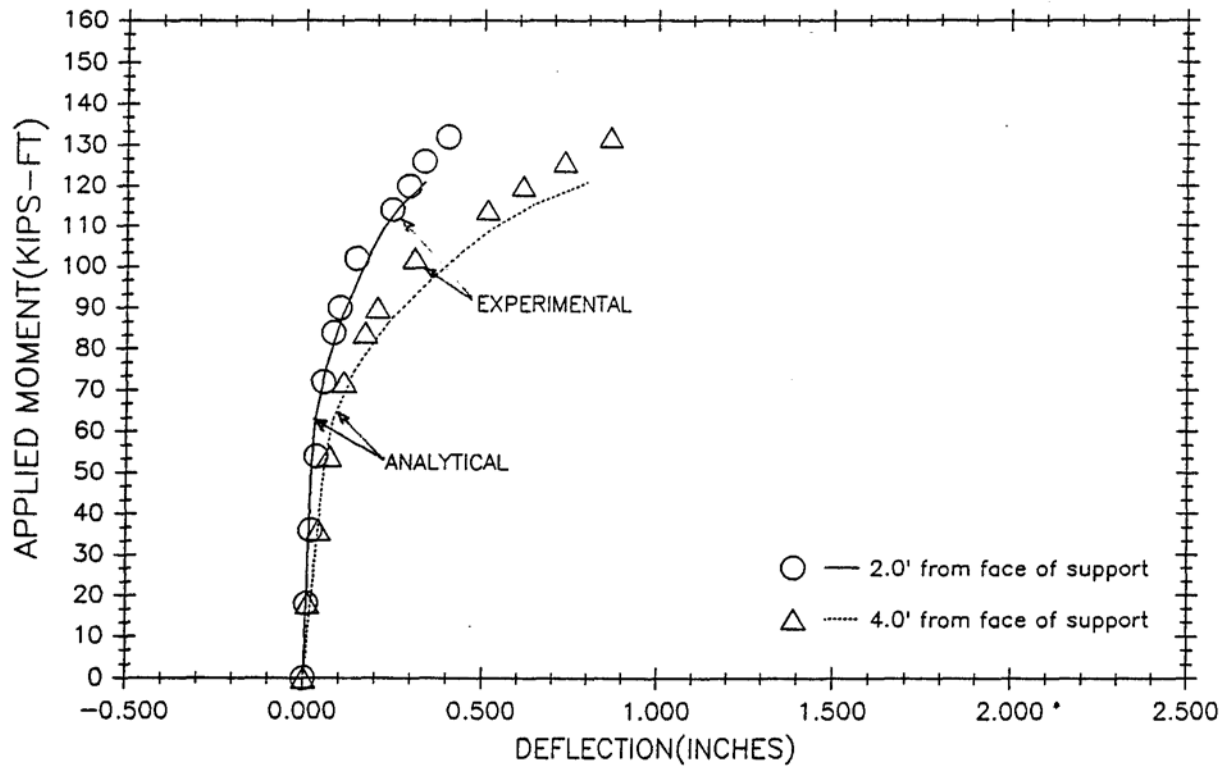
Figure 18- Analytical and Experimental Curves

stage is the post-cracking stage where deflection of the beam increases more rapidly as cracks develop. The third stage shows the behavior just prior to failure.

Typical strains along the embedded length of the pile at different load stages are shown in Figures 19 through 22.

Figure 23 is a typical applied moment-strain curve showing the experimental and the analytical strains at the face of the support.

Generally, good agreement is obtained between the experimental analytical results. The test ultimate load in each pile was somewhat higher than the analytical solution. This is particularly evident from the moment-deflection plots (Figures 15 through 18).

Figures 24 to 26 show the moment-strain relationships along the pile at different locations. These figures show that the ultimate load can be predicted fairly closely by PCFRAME.

Figures 27 and 28 show the variation in stress along the length of the prestressing tendon at various levels of applied load. The expected stress concentration at the face of support is obvious.

The extent and patterns of cracking in concrete, as predicted by the nonlinear analysis and as observed in the pile test at failure, are shown in Figures 29 through 31. The agreement between the observed and predicted crack patterns is excellent. The in

PILING TEST 5
STRAIN GAGES 1 – 6

EMBEDMENT LENGTH = 42"
PILE A–5
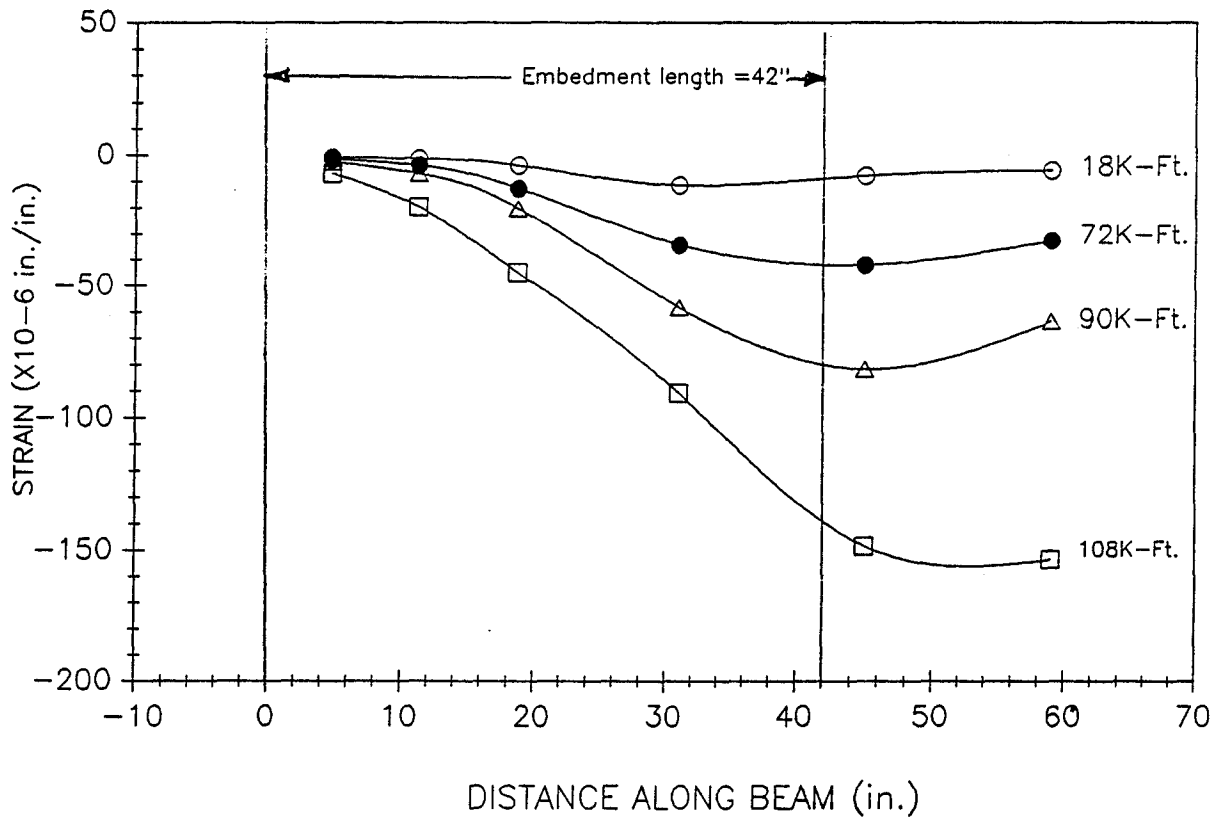
FIGURE 19 – Top Strain VS. Distance along pile embedment

PILE TEST 14
STRAIN GAGES 1−11(TOP)
EMBEDMENT LENGTH=42"

PILE B−8
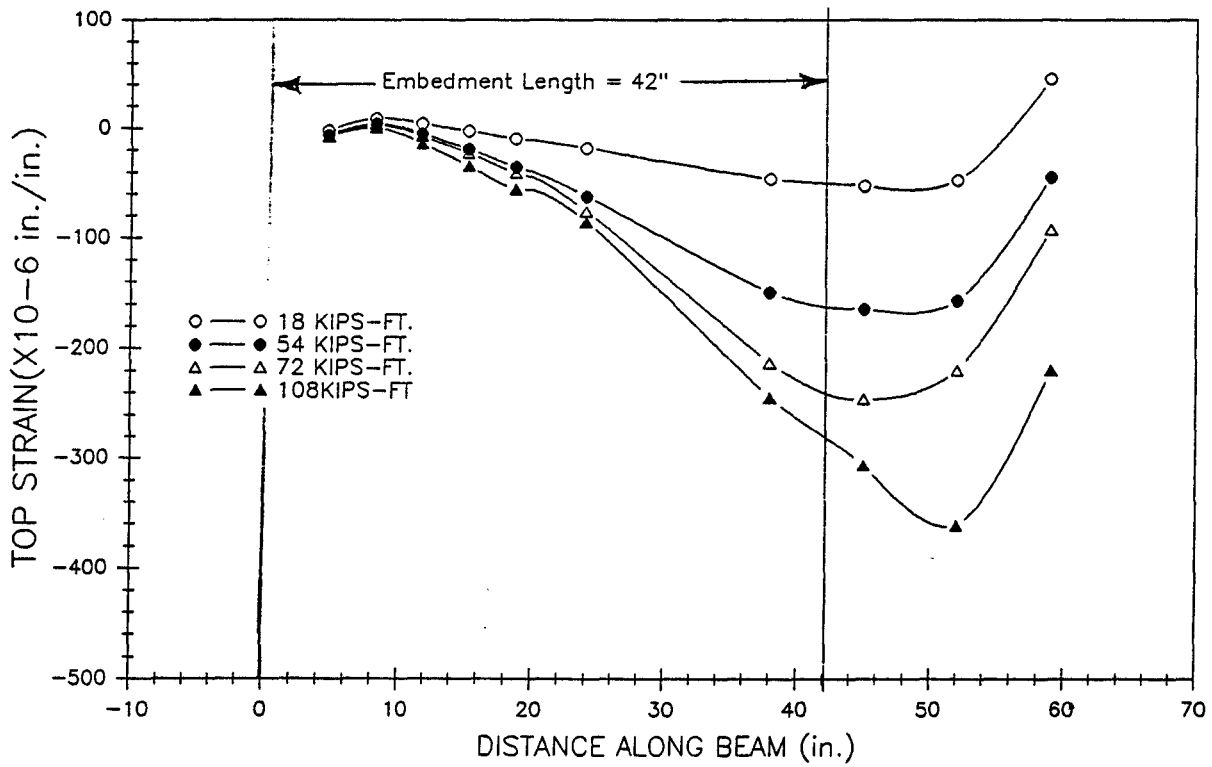
Figure 20 −Top Strain VS. Distance along Pile Embedment

PILE A−5

BOTTOM STRAIN (TENSION)
EMBEDMENT LENGTH = 42"

EXPERIMENTAL RESULTS
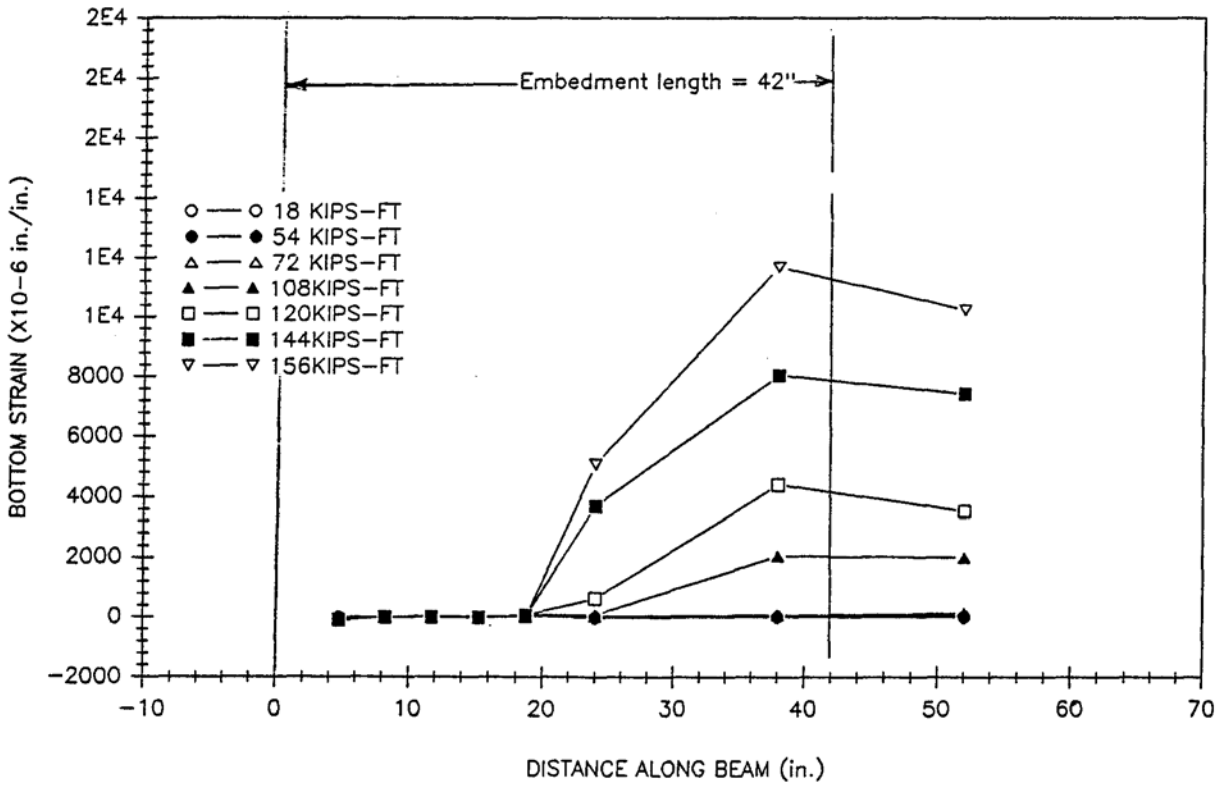
Figure 21 −Bottom Strain Along the Pile Embedment

PILE B-8

BOTTOM STRAIN (TENSION)
EMBEDMENT LENGTH = 42"
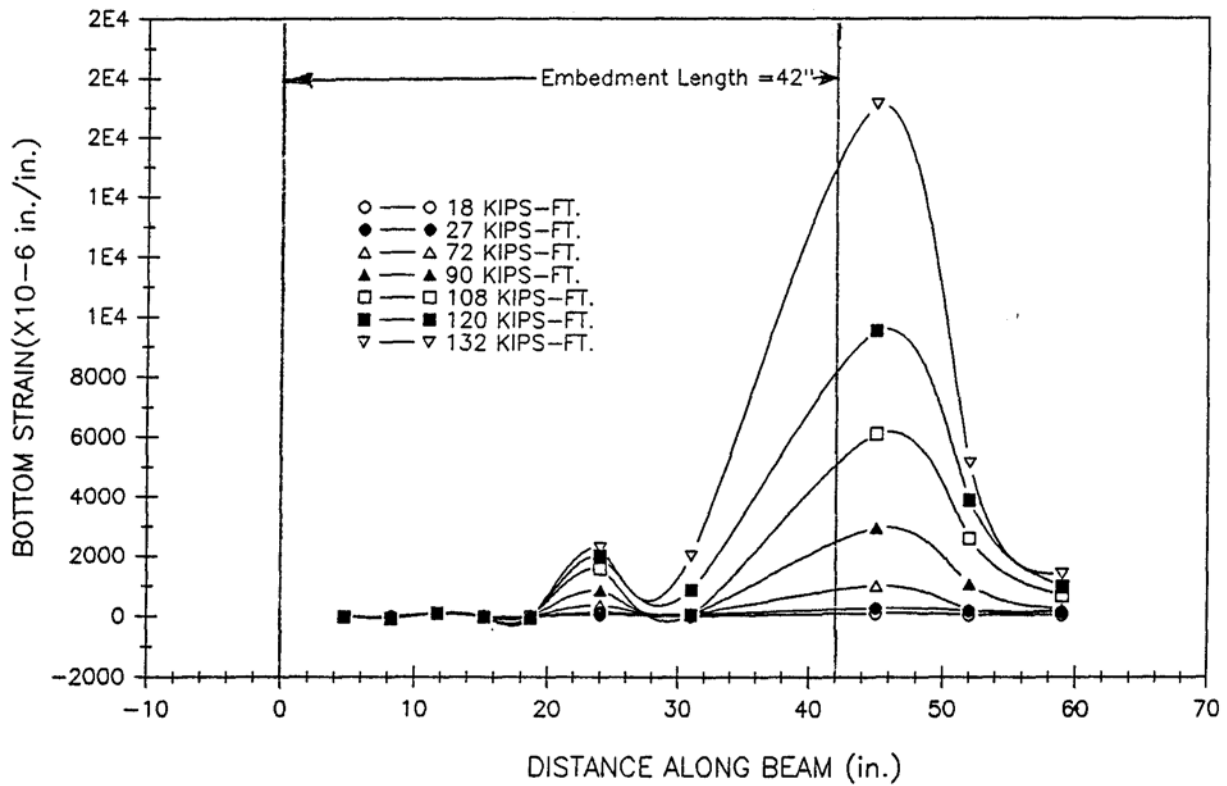
Figure 22 —Bottom Strain Along the Pile Embedment

PILE A—5

42" EMBEDMENT

APPLIED MOMENT VS STRAIN
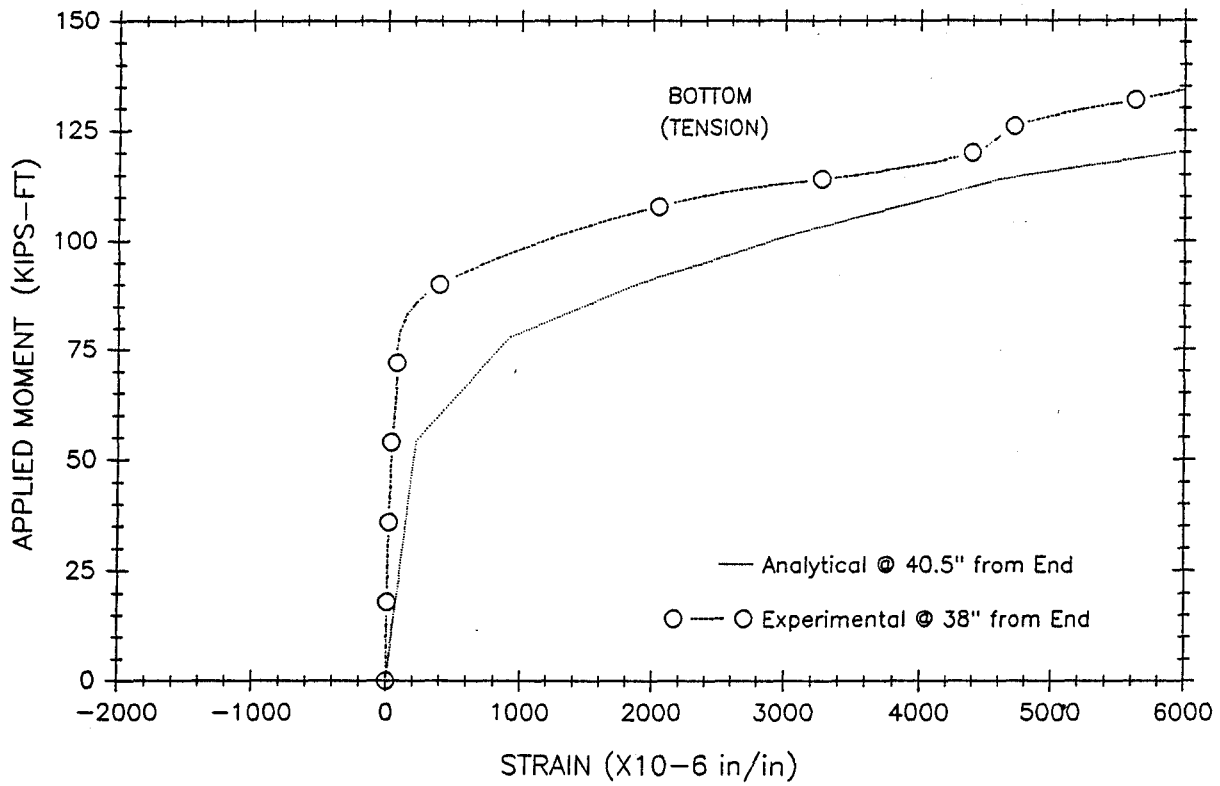
FIGURE 23 – Analytical and Experimental Curves

59

PILE A—5
EMBEDMENT LENGTH =42"

APPLIED MOMENT VS. STRAIN
COMPUTER ANALYSIS RESULTS
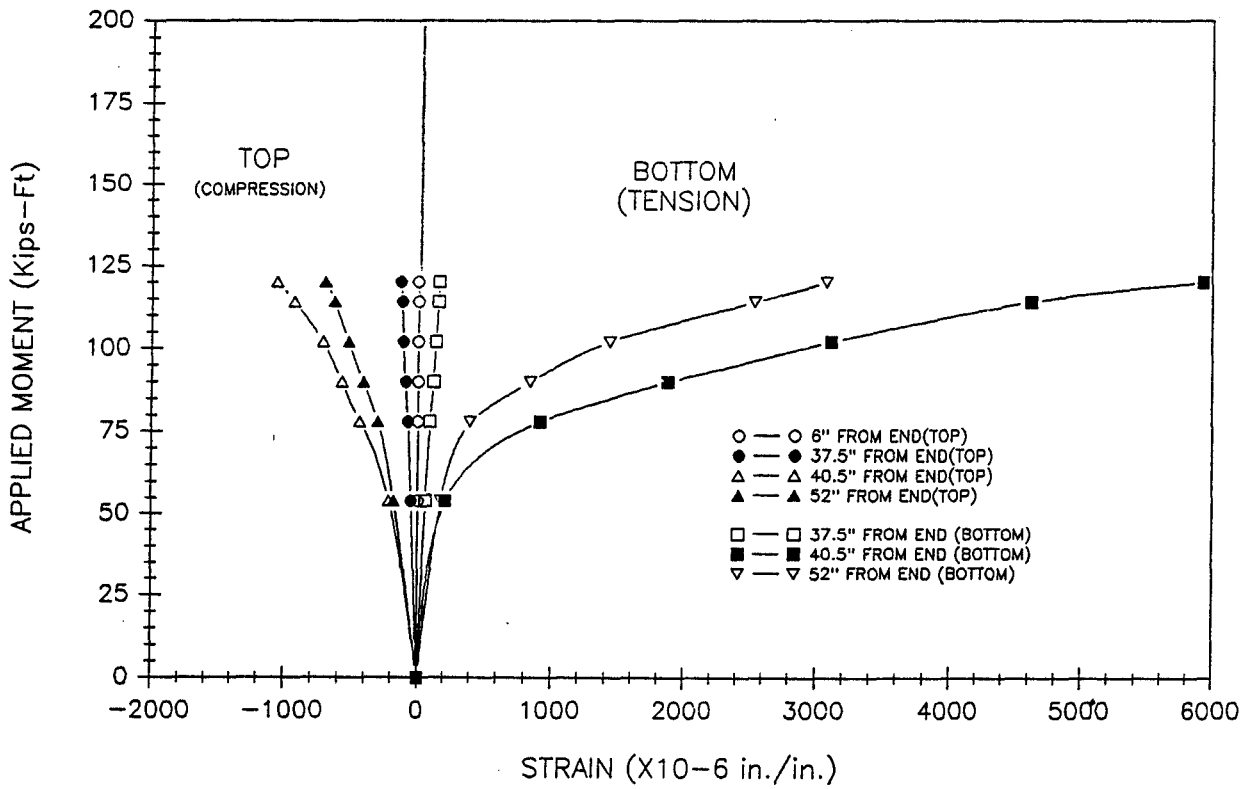
Figure 24 — Analytical Strain VS. Applied Moment

PILE B-8
EMBEDMENT LENGTH =42"

APPLIED MOMENT VS. STRAIN
COMPUTER ANALYSIS RESULTS
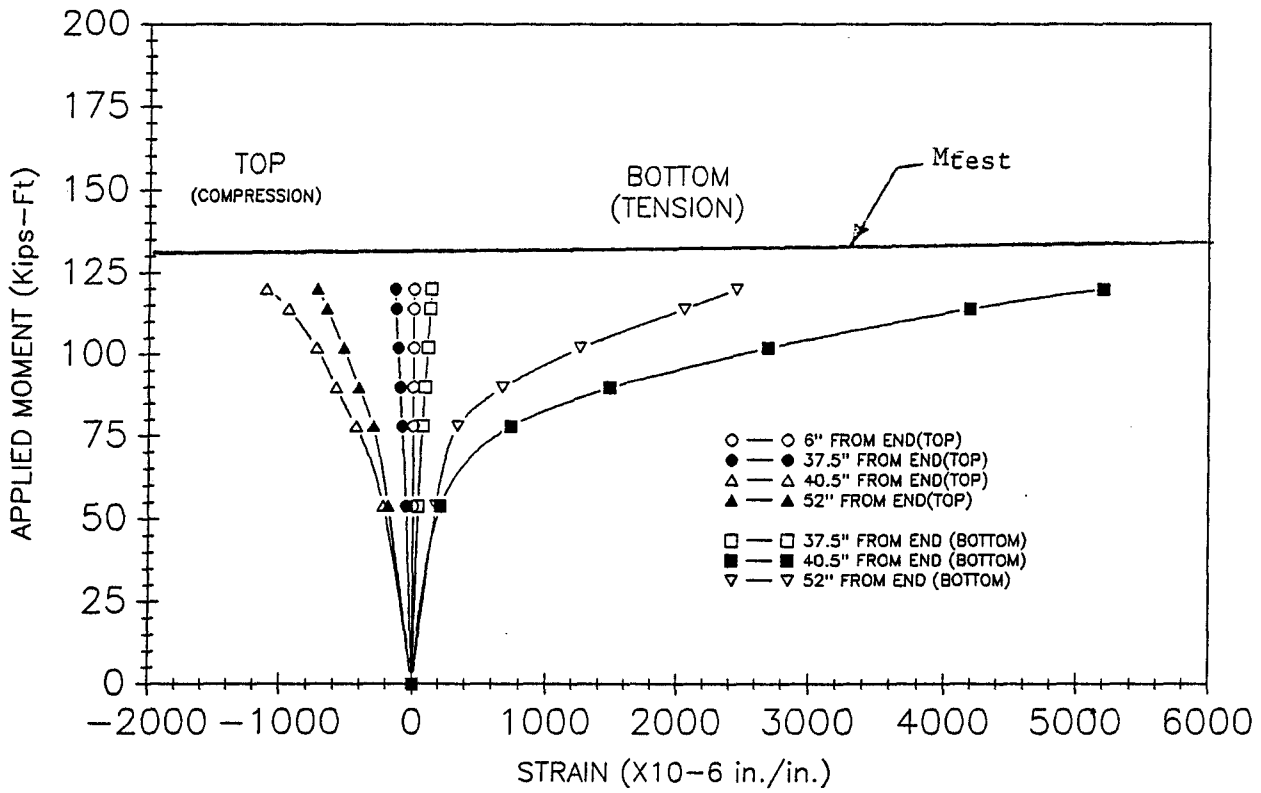
Figure 25 — Analytical Strain VS. Applied Moment

PILE A—5
EMBEDMENT LENGTH =42"

APPLIED MOMENT VS. STRAIN
EXPERIMENTAL RESULTS
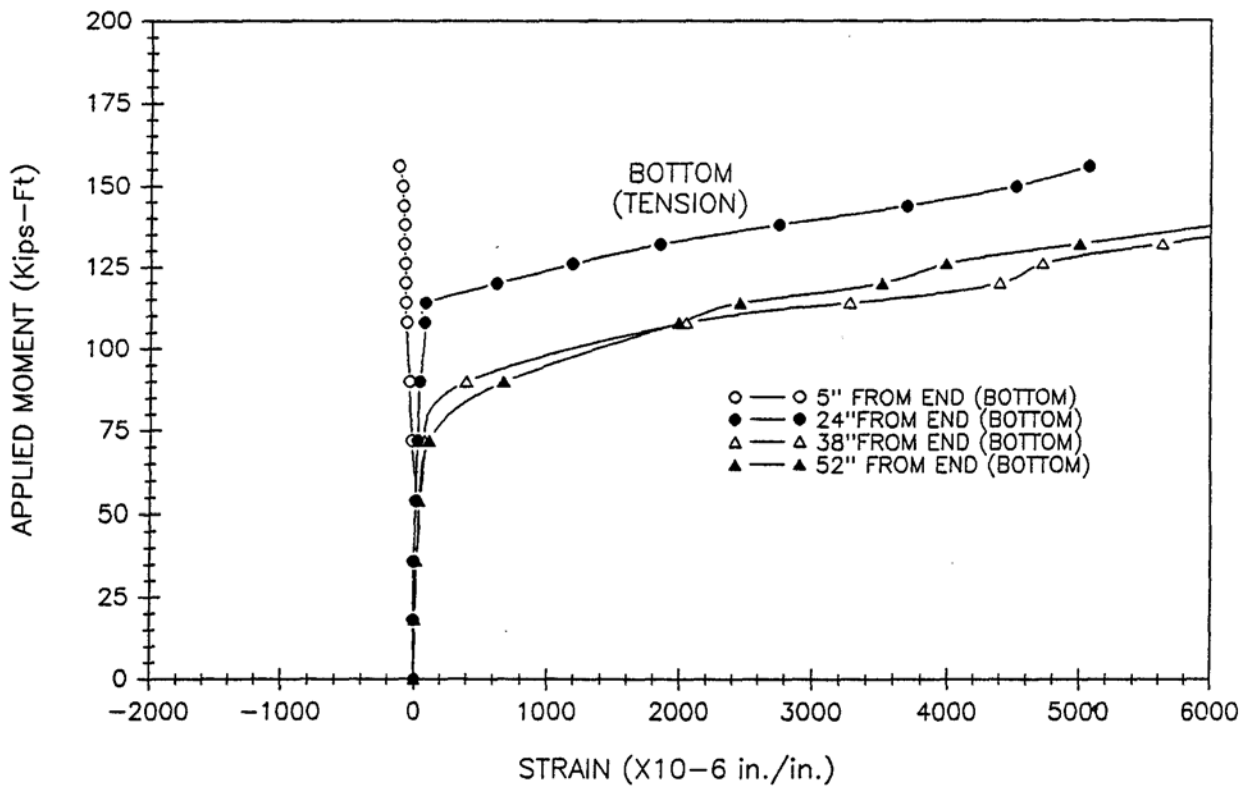
Figure 26 —Experimental Strain VS. Applied Moment

PILE A-2
BOTTOM STEEL STRESSES
EMBEDMENT LENGTH = 48"
COMPUTER ANALYSIS RESULTS

STRESSES ALONG THE PILE LENGTH
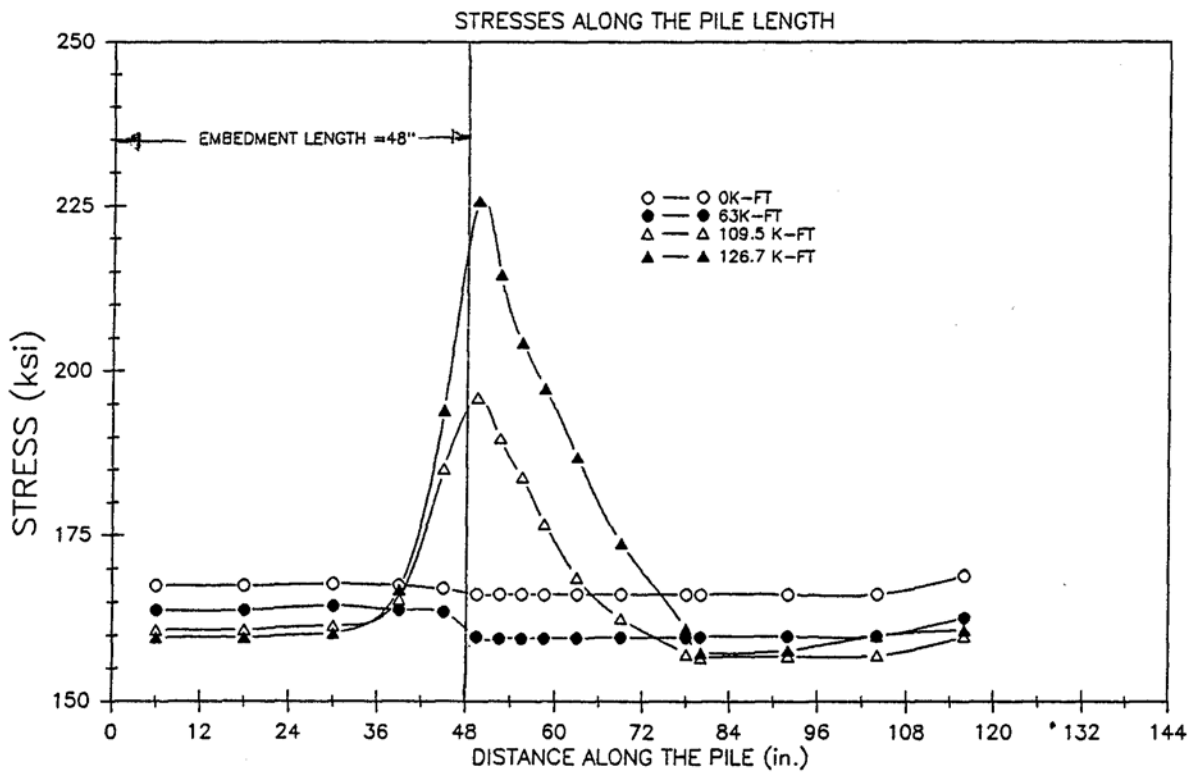
Figure 27 —strand stresses along the Pile at different moments

PILE A-4
BOTTOM STEEL STRESSES
EMBEDMENT LENGTH =36 "
COMPUTER ANALYSIS RESULTS

STRESSES ALONG THE PILE LENGTH
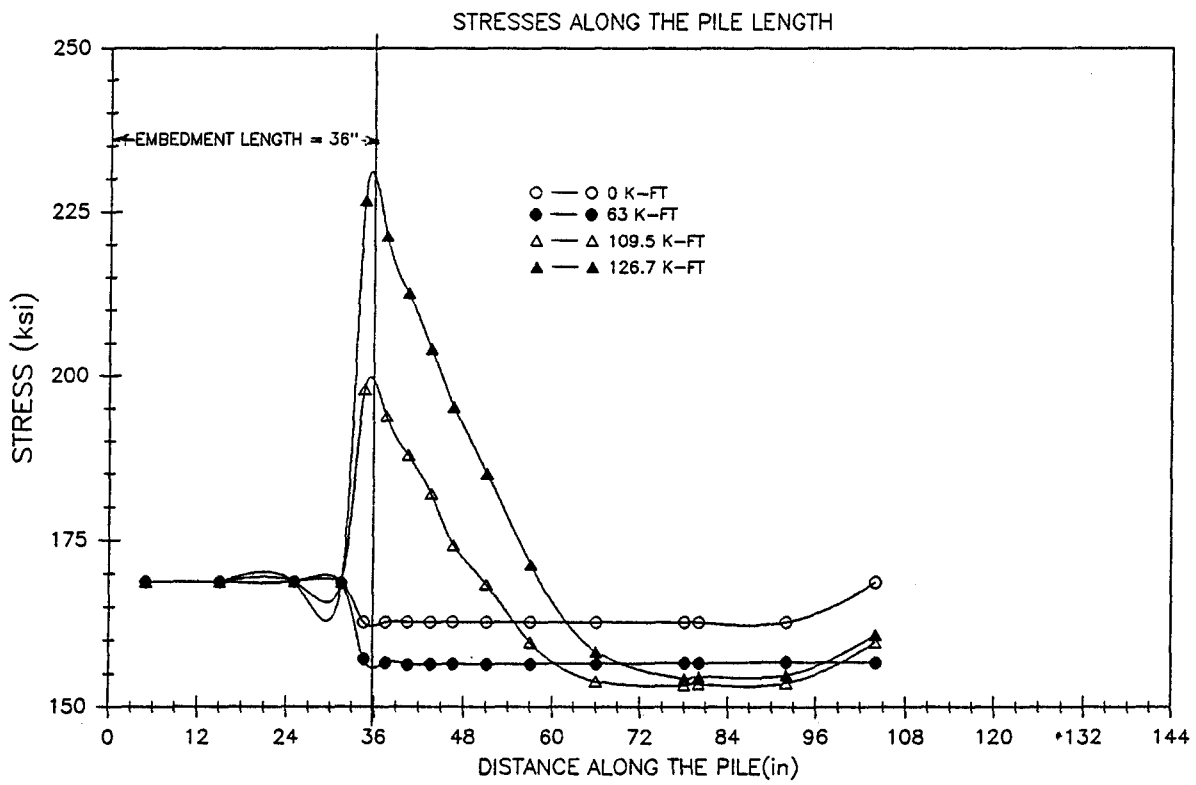
Figure 28-strand stresses along the pile at different moments

64

PILE C-1

Applied Moment = 86.5 k-ft

Applied Moment = 121 k-ft

- ▨ IN TENSION
- ■ CRACKED
- ☐ IN COMPRESSION
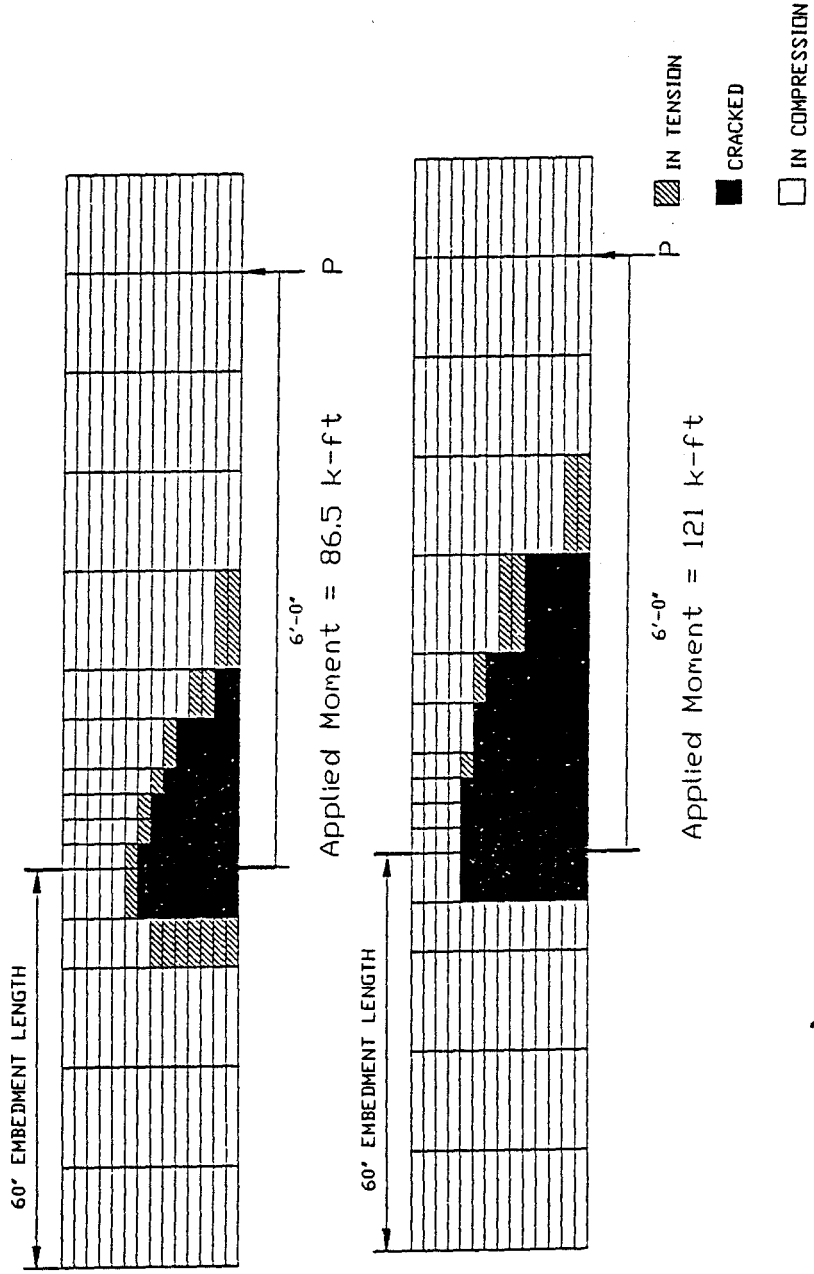
FIGURE 29-Predicted Extent of Cracking at different Applied Moments

WEST VIEW

EAST VIEW

PILE TEST NO. C-11
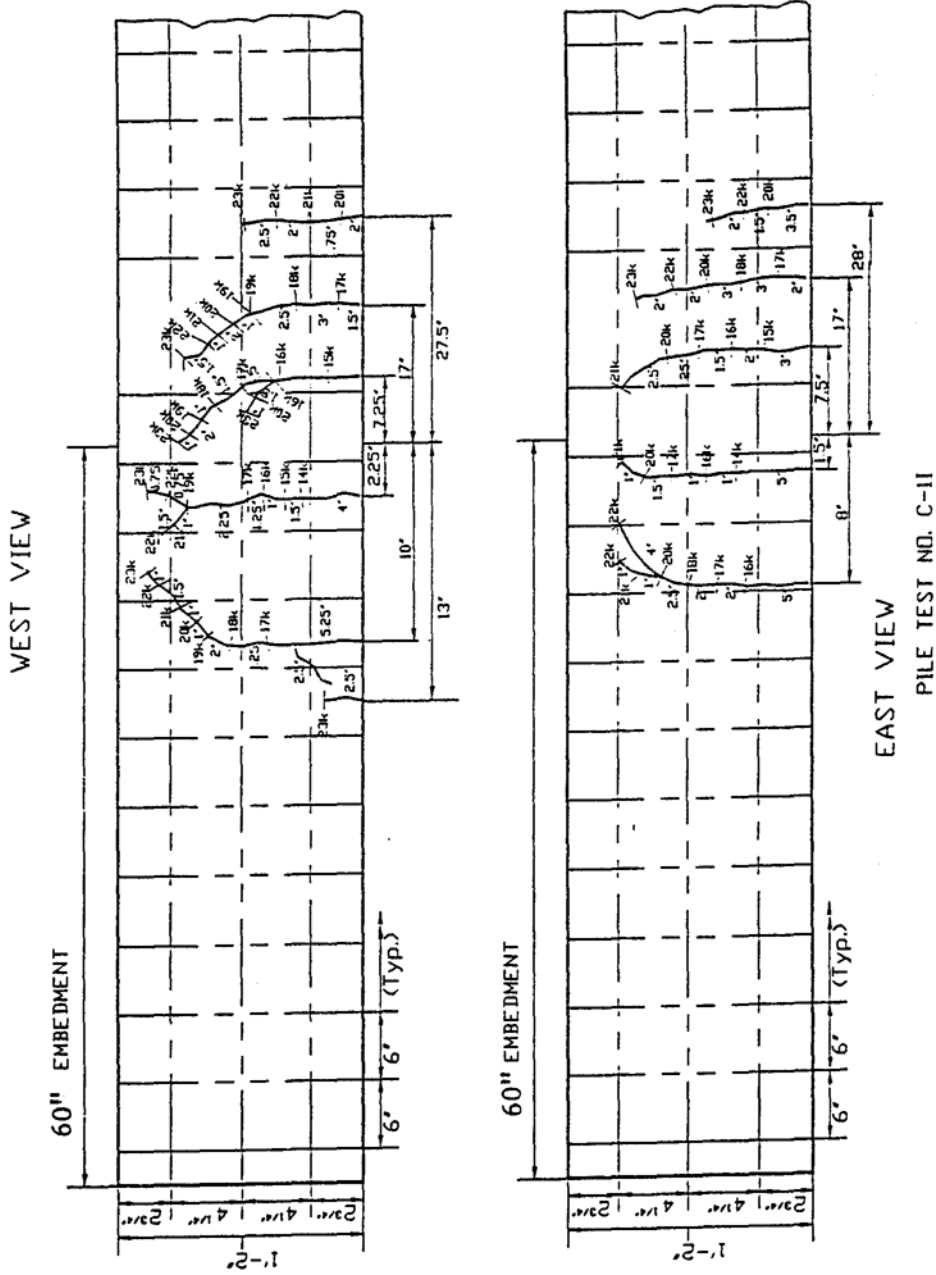
Figure 30— Observed Cracking at Different Load Levels
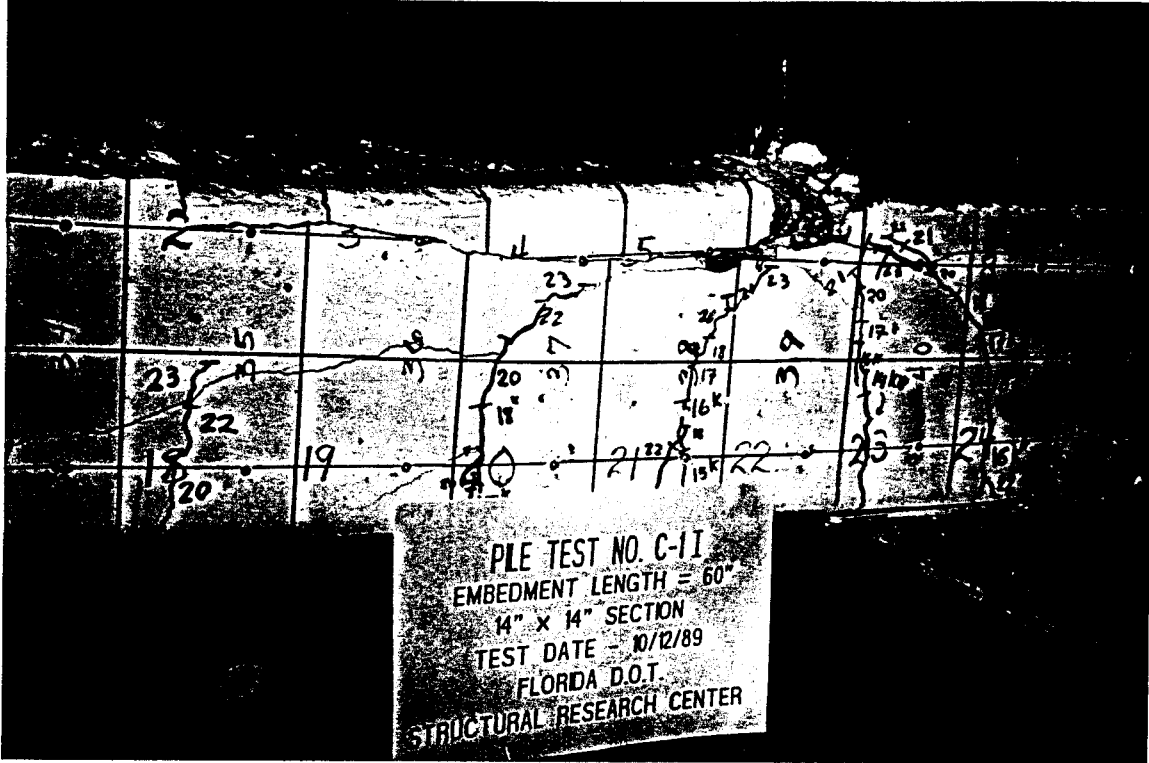
FIGURE 31
OBSERVED CRACKING FAILURE

crease in stress in the prestressing tendon and the crack depth predicted by computer analysis (PCFRAME) can be seen in these

5.3 Effect of Concrete Strength

The results in Table 2 illustrate the observed relationship of initial bond strength (force required to cause first slip of the strand) to concrete strength. It can be seen that concrete strength has only a minor effect on initial bond strength. This is in general agreement with the findings of Salmons and McCrate[16].

5.4 Effect of Strand Diameter

The effect of strand diameter on bond capacity of strands subject to flexure can be studied by considering the results of tests of specimens in groups 2 and 3. A point of reference maybe established by considering the average values of initial strength developed by all specimens in these groups. Figure 32 illustrates the relationship of initial bond stress to embedment length for a 1/2 inch nominal diameter strands.

The relationship shown in this figure supports the generally accepted assumption that initial bond stress is proportional to embedment length.

5.5 Effect of Shear Confinement

The spiral shear reinforcement varied along the length of the pile as shown in Figure 3. The test specimens from the pile end

section were provided with more shear reinforcement than those from the interior section (see Figure 4). Figure 33 shows that the shear confinement at the end section generally increased the moment capacity of the piles.(see Table 1)

5.6 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 a given section, is illustrated in figure 34, in which M_{test} is the measured moment at ultimate strength, M_{bond} is the measured moment at the general bond failure, M_{calc} is the calculated ultimate flexural moment.

Piles having an embedment length of 48" or more failed in flexure by crushing of the concrete after yielding of the steel. As the embedment length decreased, failure occurred at progressively lower moments due to slippage of the strands. Only one specimen having an embedment length of 48 inches or higher-showed any strand slip at ultimate moment, which seems to justify and embedment length of approximately 50 inches for 1/2" and 7/16" diameter strands.

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
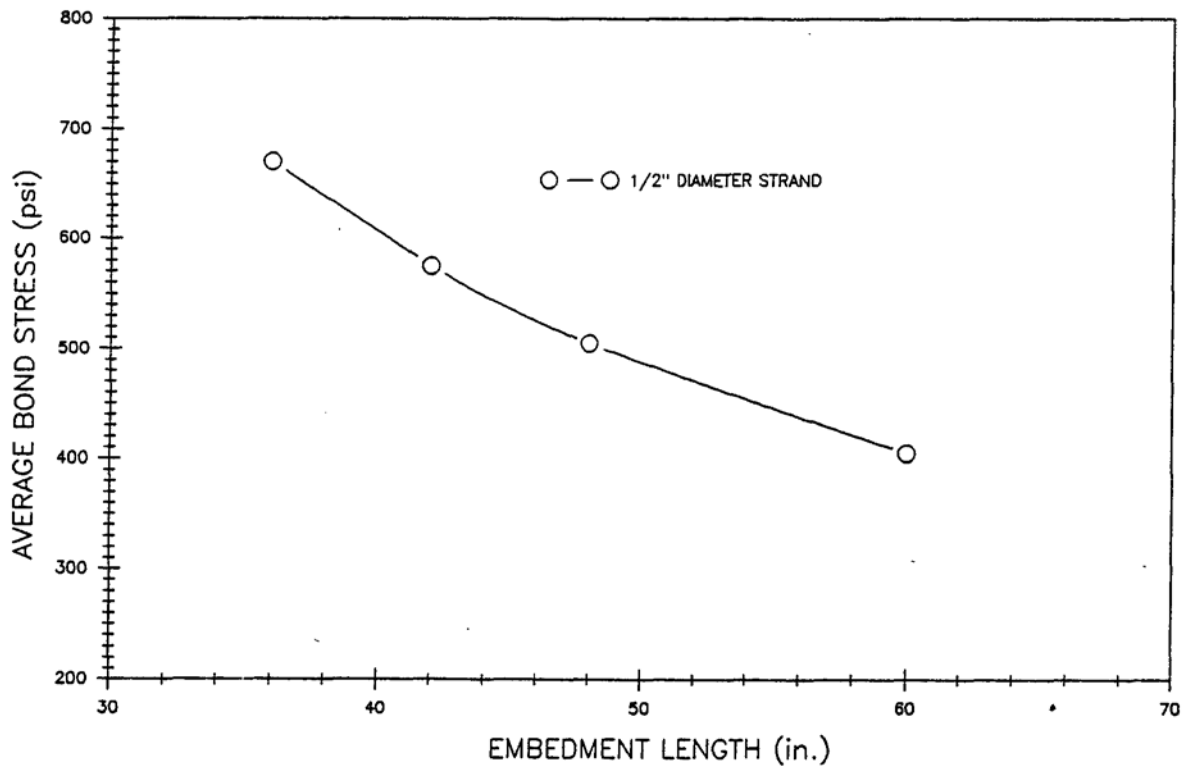
Figure 32- Average Initial Bond Stress VS. Embedment Length
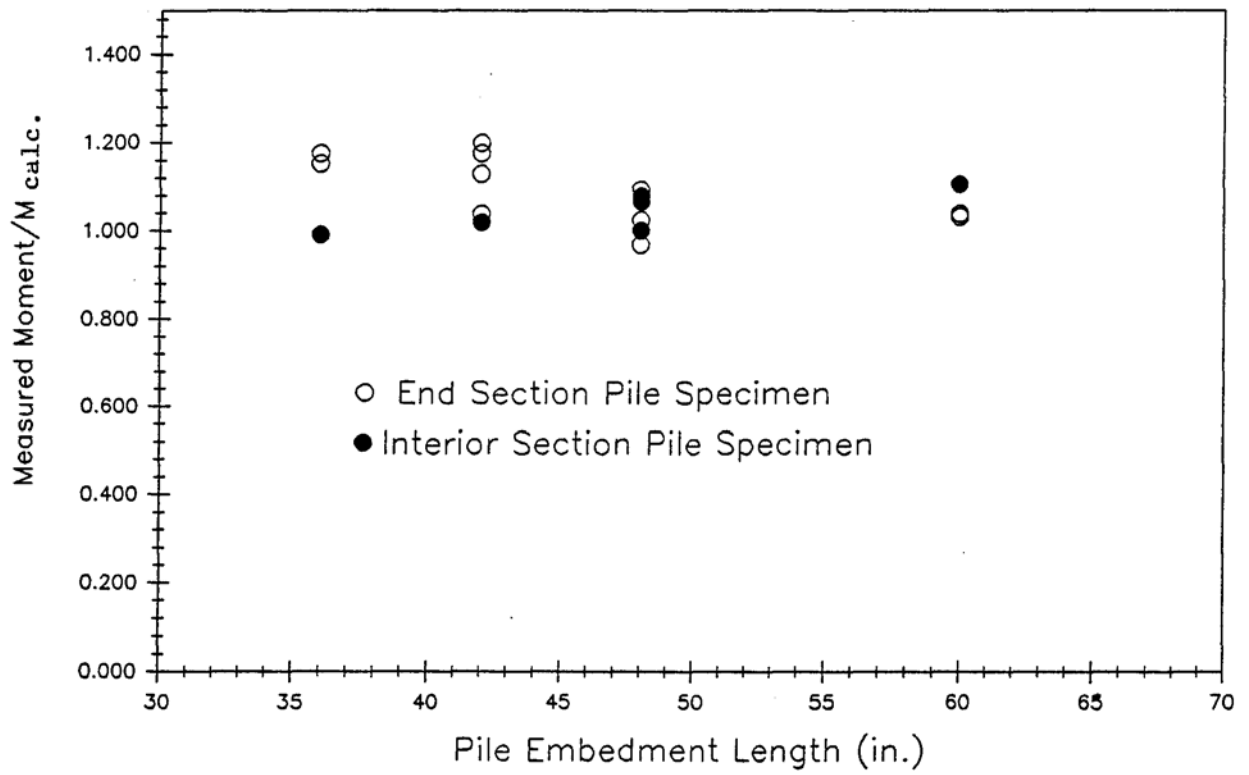
Figure 33 —Comparison Between Interior and End Pile Specimens
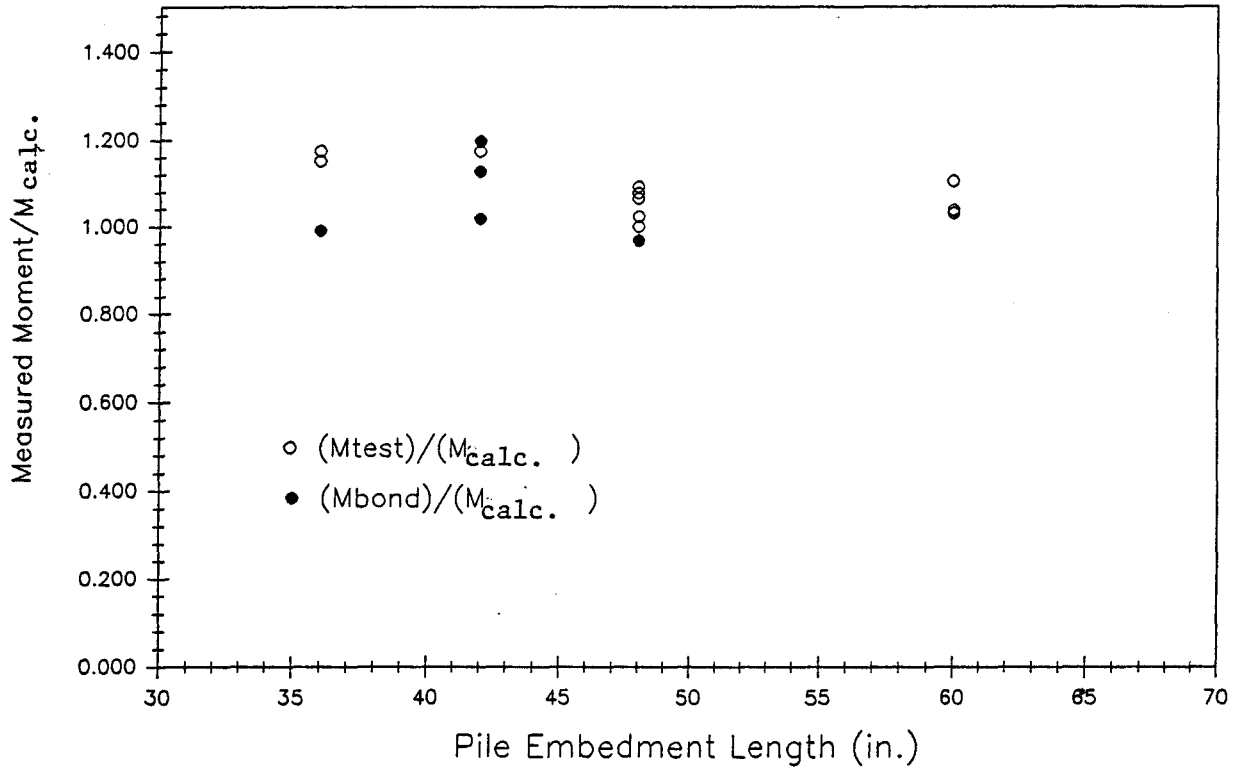
Figure 34 −Pile Performance at Various Embedment Lengths

interlocking effect between the strand surface and the surrounding concrete. In the case of piles with short embedment lengths (36"), a small increase of load was seen between these stages. Figure 35 shows plots of end slippage of the bottom strands vs applied moment.

5.7 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 and the ultimate moment sustained were both of interest in this study. A comparison between the experimental and the analytical ultimate flexural strength is presented in Table 2.

The differences in the modes of failure 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 sketch of the crack pattern of a static test that resulted in a bond failure is shown in Figure 36. 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. No appreciable strand end slip was measured during

a flexural failure. Figure 37 shows a sketch of the crack pattern in a flexural failure. Sketches of the typical crack pattern for the remaining piles are presented in Appendix B.

PILE B-9

EMBEDMENT LENGTH = 36"
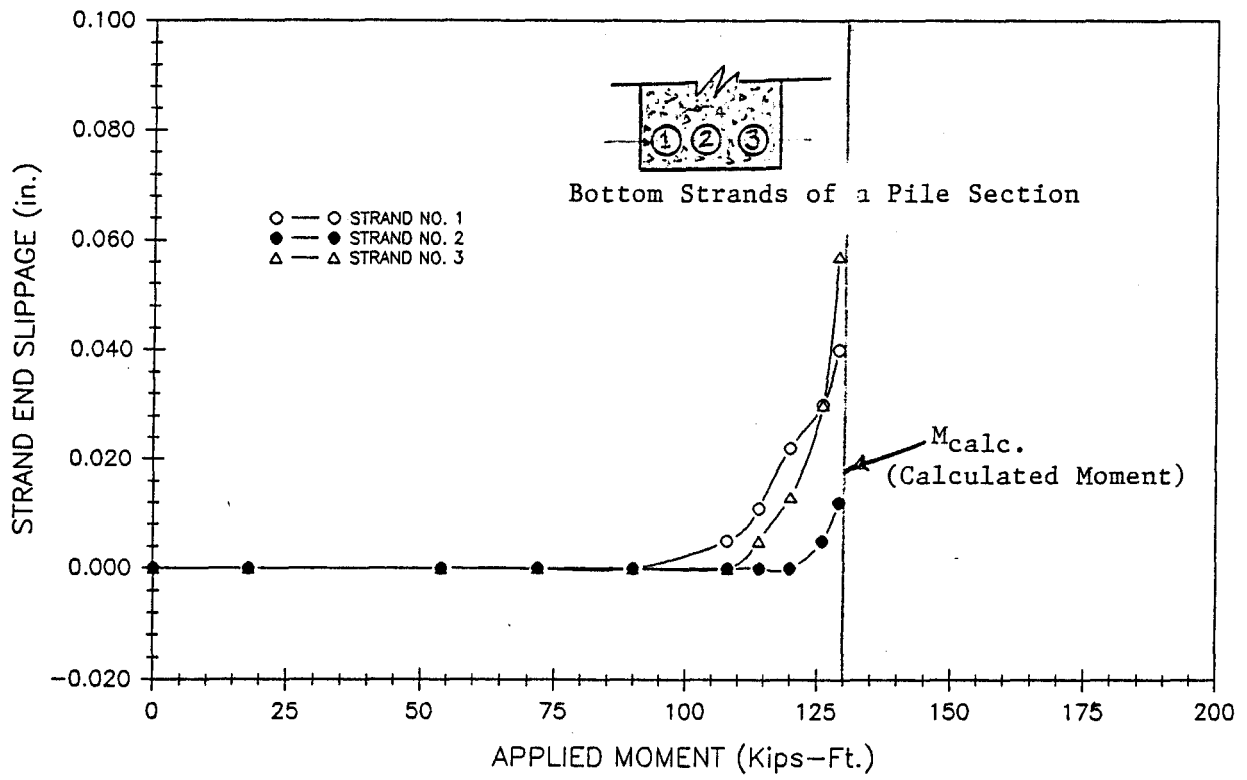
Figure 35 —Applied moment VS. Strand End Slippage
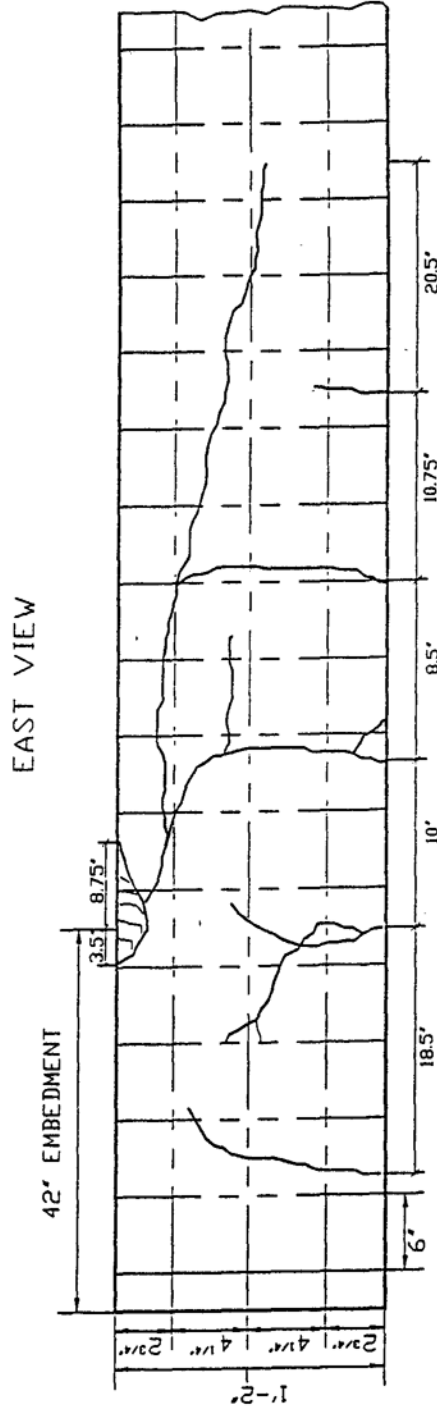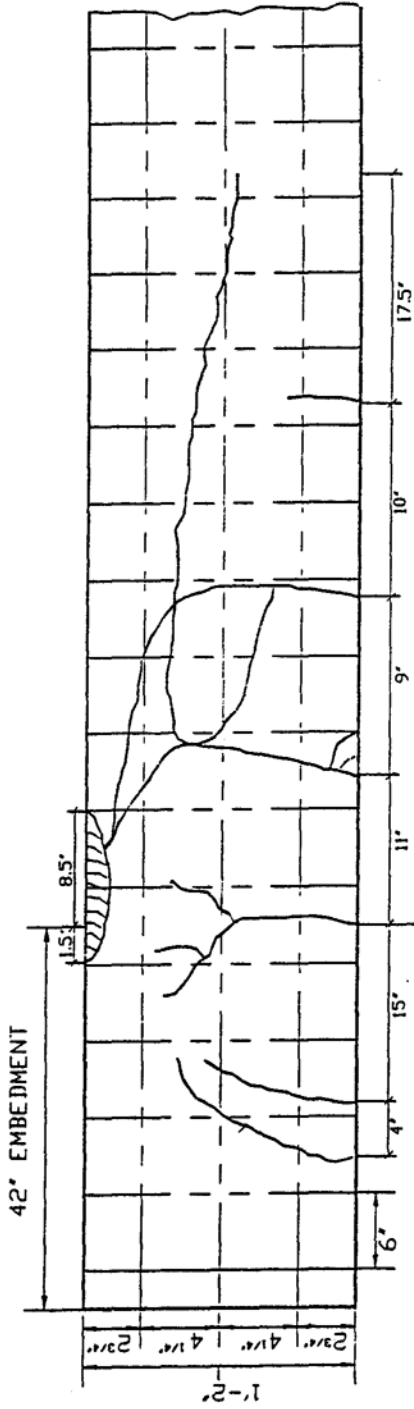
Figure 36—Cracking Pattern of Bond Failure
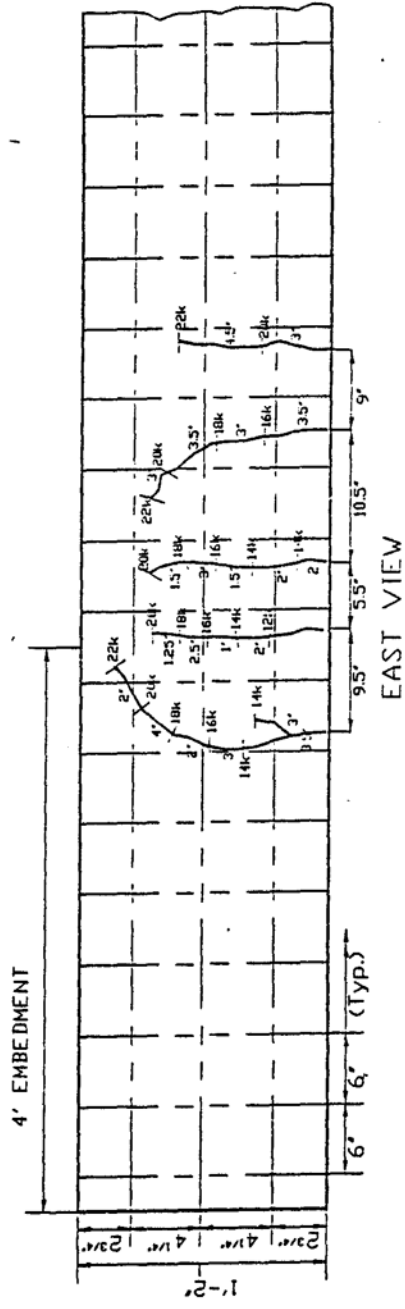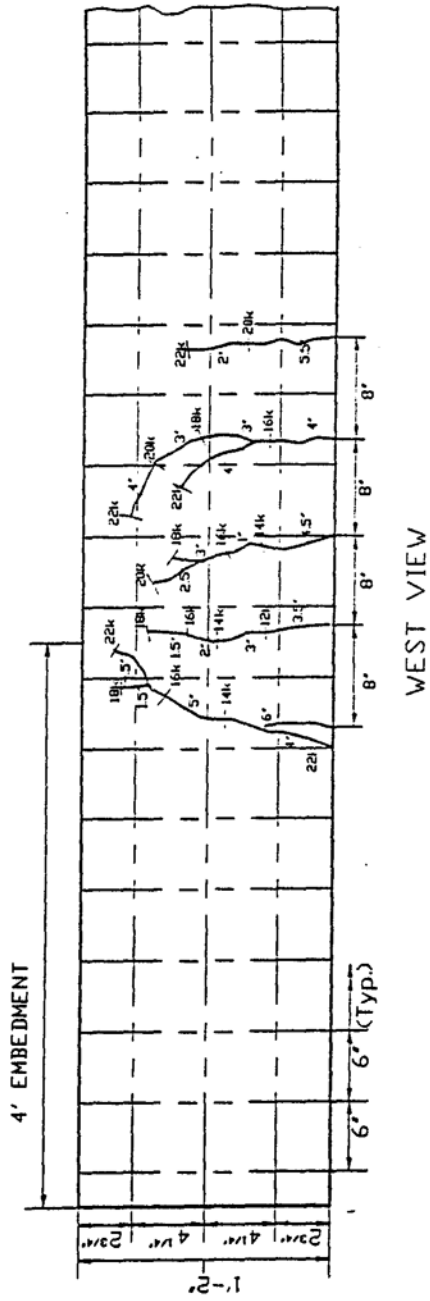
Figure 37–Cracking Pattern of Flexural Failure

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

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 and ACI Codes were examined. Also examined, were the effects of concrete strength, the general bond slip and the maximum average bond stress at failure.

It was found that the embedment length for 1/2" strand has a marked influence on the value of the average bond stress at which general bond slip occurs; this is clearly demonstrated in Figure 32.

The tests were designed to simulate the clamping action to which a pile embedded in a massive footing would be subjected. t is shown that an embedment length of 50 inches 100 d_b for 1/2"(diameter shown), is adequate to develop the flexural strength of such a pile without slippage of the strands. Therefore, there appears to be no justification for the application of a multiplier to the embedment length transfer in the current ACI and AASHTO Codes. Of course, this conclusion appears only to piles that are subjected to clamping action at their ends and not necessarily to

beams.

It must be borne in mind that 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. On the other hand, the beam is subjected to many repetitive load cycles and must therefore be designed to prevent a short fatigue life.

REFERENCES

1. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-83)", American Concrete Institute, Detroit, Michigan, 1983, 111pp.

2. Anderson, A.R. and Anderson,, R.G. "An Assurance Criterion for Flexural Bond in Pretensioned Hollow Core Units", J. Am. Conc. Inst. August 1976 (Proc. Vol. 73), pp. 457-464.

3. Arthur H. Nilson, "Design of Prestressed Concrete", Second Edition, John Willey & sons, New York, 1987.

4. Base, G.D. "An Investigation of Transmission Length in Pretensioned Concrete", Research Report No. 5, Cement and Concrete Association, London, 1958.

5. Ghosh, S.K. and Fintel M., "Development length of Prestressing Strands Including Debonded Strands", Journal of Prestressed Concrete Institute Vol. 31, No. 5 (Sept/Oct 1986)

6. Hanson, N.W. "Influence of Surface Roughness of PrestressingStrand on Bond Performance", Journal of the Prestressed Concrete Institute, 14, No. 1, 32-45 (Feb 1969)

7. Hanson, N.W. and Kaar, P.H. "Flexural Bond Tests of Pretensioned Prestressed Beams", J. Am. Conc. Institute, January 1959, pp. 783-802. Also, Development Department Bulletin D28, Portland Cement Association, Skokie, Illinois.

8. Hoyer E. and Friedrich, E. "Beitrag Zur Frage der Hafspannung in Eisenbetonbauteilen", Beton and Eisen, Berlin, 1939 (Vol. 38, No. 6), pp. 107-110. Also K. Billig, Prestressed Concrete, Van Nostrand Co., New York, 1953.

9. James R. Libby, "Modern Prestressed Concrete", Third Edition, Van Nostrand Reinhold Cold Co., New York, 1984

10. Janney, J.R. "Nature of Bond in Pre-Tensioned Prestressed Concrete", J. Am. Conc. Inst., May 1954 (Proc. Vol. 50), pp. 717-736. Also E. Hognestad and J.R. Janney, "The Ultimate Strength of Pre-Tensioned Prestressed Concrete Failing in Bond", Magazine on Concrete Research, June 1954.

11. Janney, J.R., Hognestad, E., and Mchenry, D., "Ultimate Flexural Strength of Prestressed and Conventionally Reinforced Concrete Beams", ACI Journal, Proceedings V. 52, No. 6, February 1956, pp. 601-620. See also PCA Development Department Bulletin D7.

12. Kaar, p.H., LaFraugh, R.W., and Mass, M.A., "Influence of Concrete Strength on Strand Transfer length", PCI Journal, V. 8, No. 5, October 1963, pp. 47-67. Also PCA Development Department Bulletin D71.

13. Kaar, P.H., and Magura, D.D., "Effect of Strand Blanketing on Performance of Pretensioned Girders", PCI Journal, V. 10, No.

6, December 1965, pp. 20-34. Also PCA Development
Department Bulletin D67.

14. Marshall, W.T. and Mattock, A.H. "Control of Horizontal Cracking in the Ends of Pretensioned Prestressed Concrete Girders", J. Prestressed Conc. Inst. Vol.7, No. 5, October 1962, pp. 56-74.

15. Martin, L.D., and Scott, N.L., "Development of Prestressing Strand in Pretensioned Members", ACI Journal, Proceedings V. 73, No. 8, August 1976, pp. 453-456.

16. Salmons, John R., and McCrate, Timothy E., "Bond Characteristics of Untensioned Prestressing Strand", PCI Journal, V. 22, No. 1, January-February 1977, pp. 52-65.

17. Stacker, M.F., and Sozen, M.A., "Investigation of Prestressed Reinforced Concrete for Highway Bridges, Part V: Bond Characteristics of Prestressing Strand, Bulletin no. 503, Engineering Experiment Station, University of illinois,_ Urbana, Illinois, 1970.

18. Zia, P. and Mostafa, T. "Development Length of Prestressing Strands", J. Prestressed Conc. Inst. vol. 22, No. 5, September/October 1977, pp. 54-65.

APPENDIX A

APPLIED MOMENT VS. DEFLECTION CURVES

PILE A−4

36" EMBEDMENT

APPLIED MOMENT VS DEFLECTION
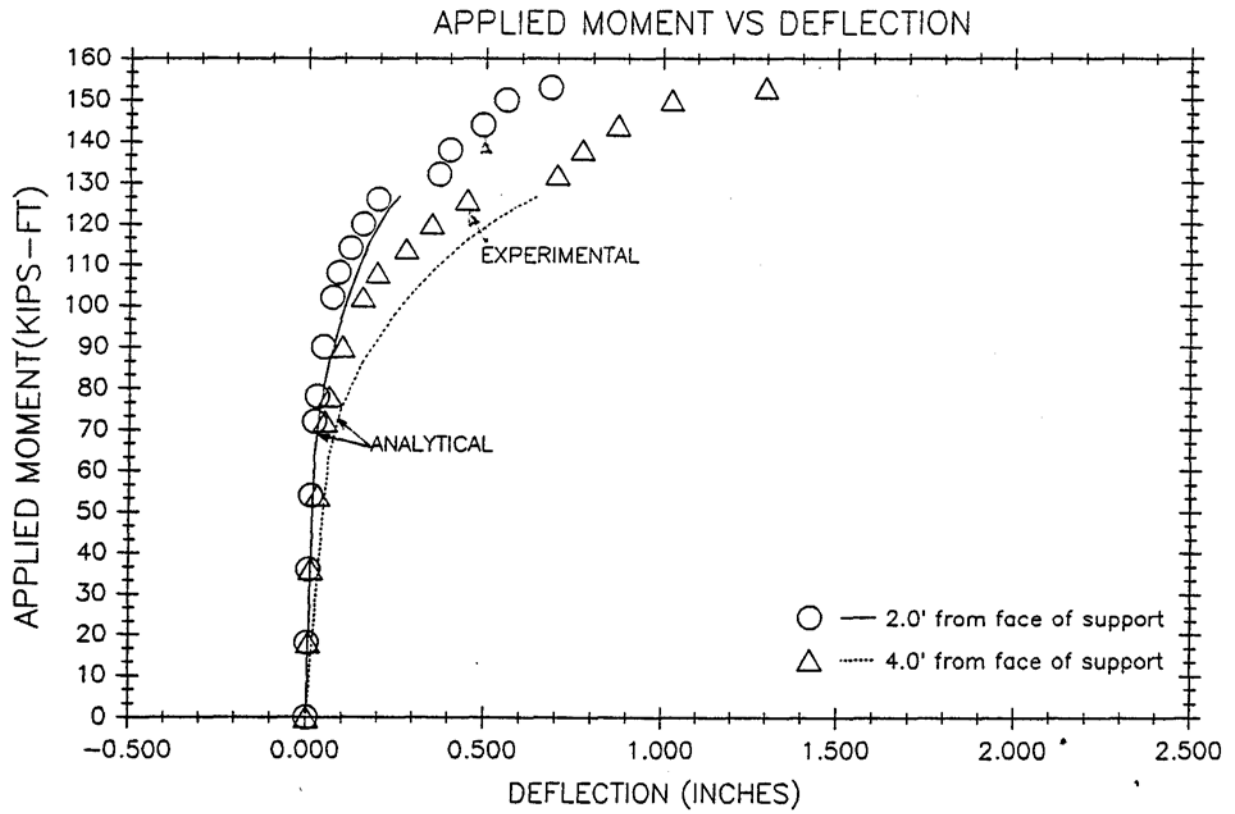
Figure A.1- Analytical and Experimental Curves

A−1

PILE B-2

36" EMBEDMENT

APPLIED MOMENT VS DEFLECTION
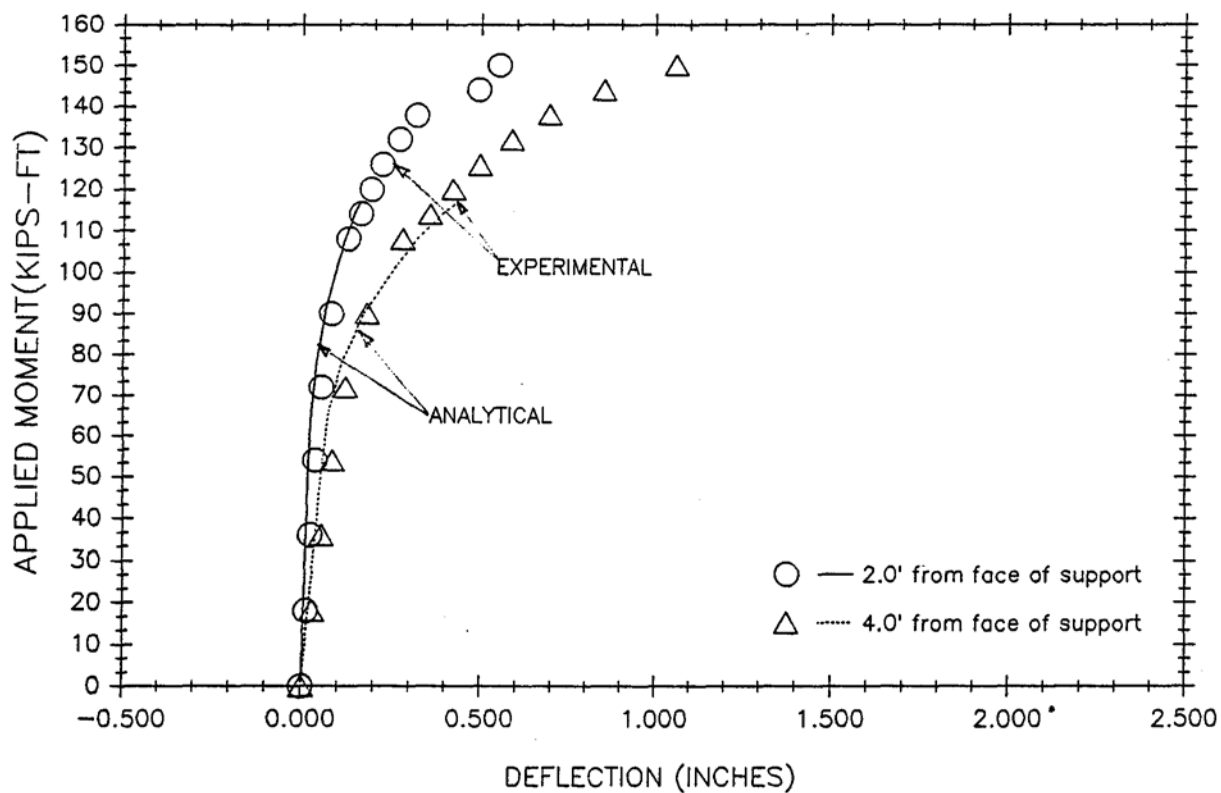
Figure A.2-Analytical and Experimental Curves

PILE B—7

36" EMBEDMENT
APPLIED MOMENT VS DEFLECTION
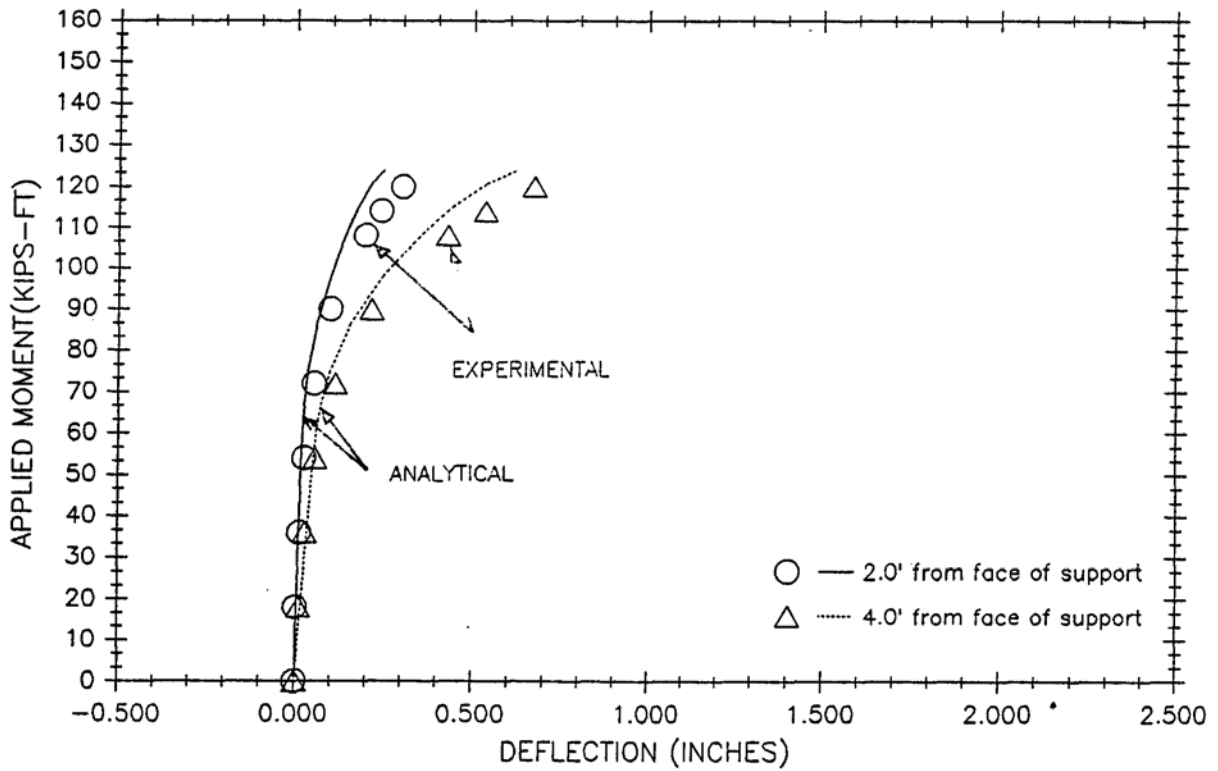
Figure A.3–Analytical and experimental Curves
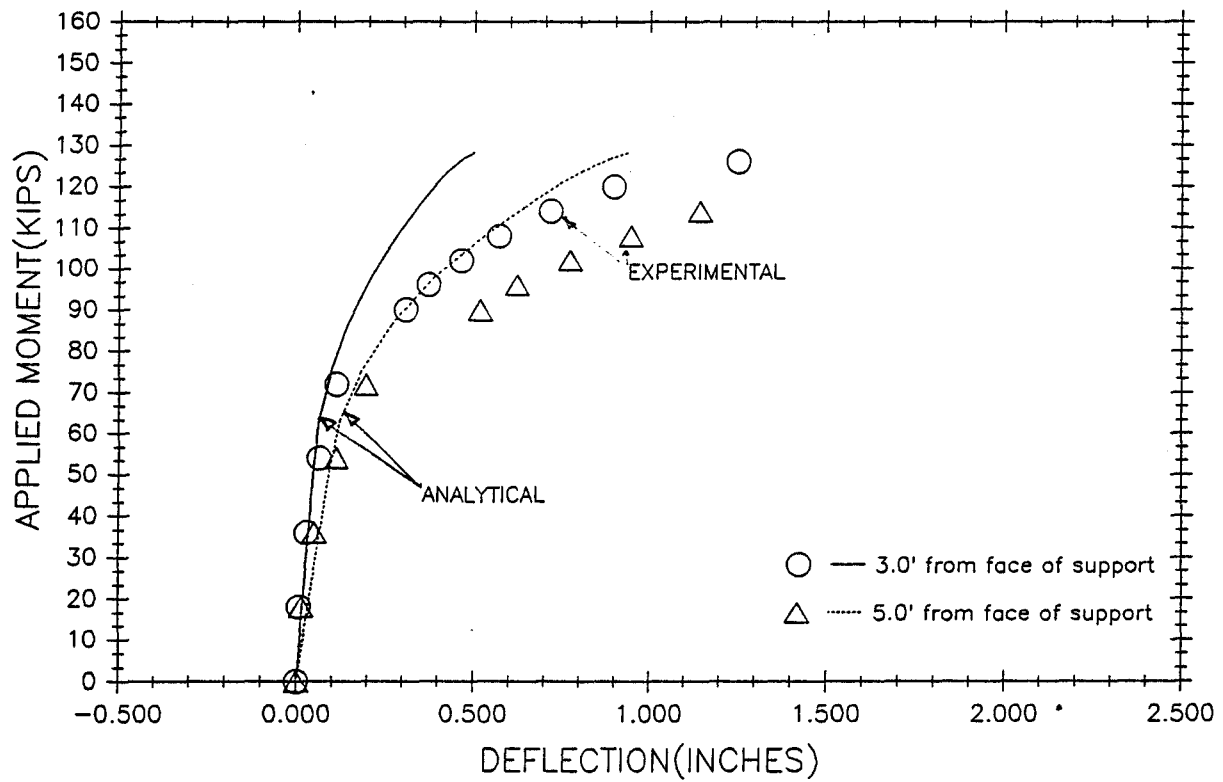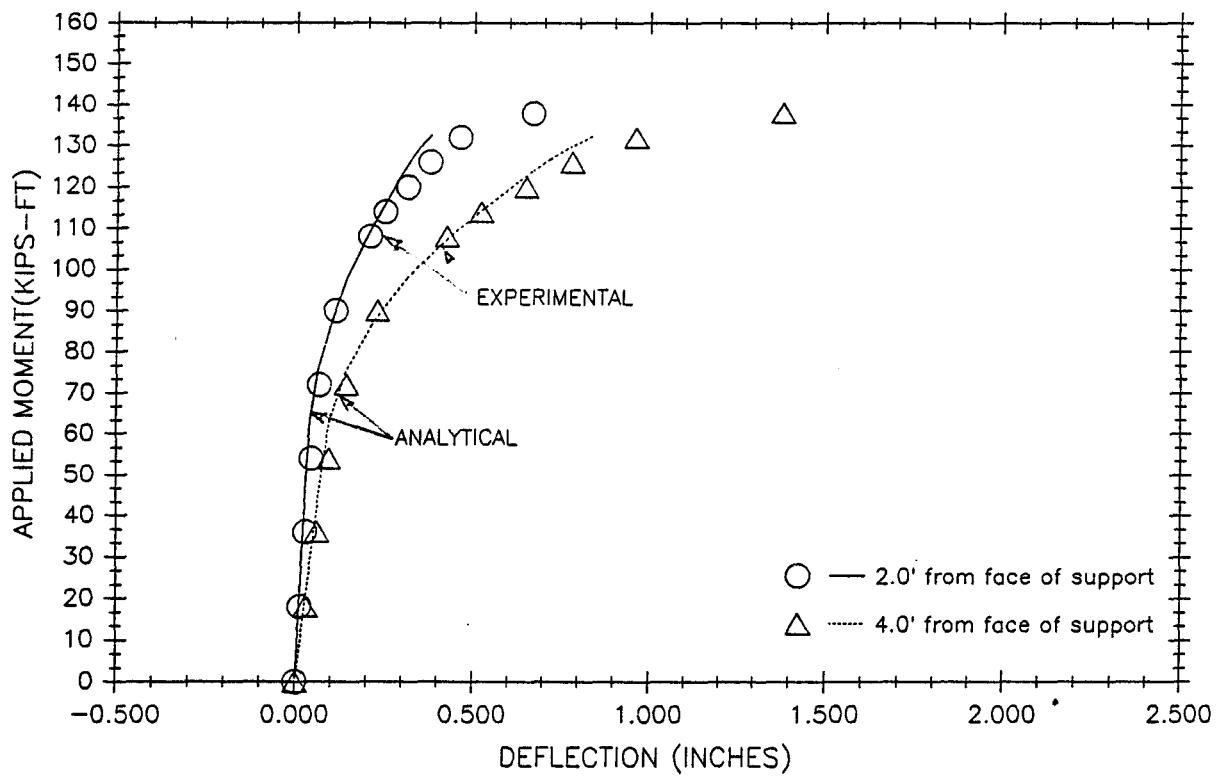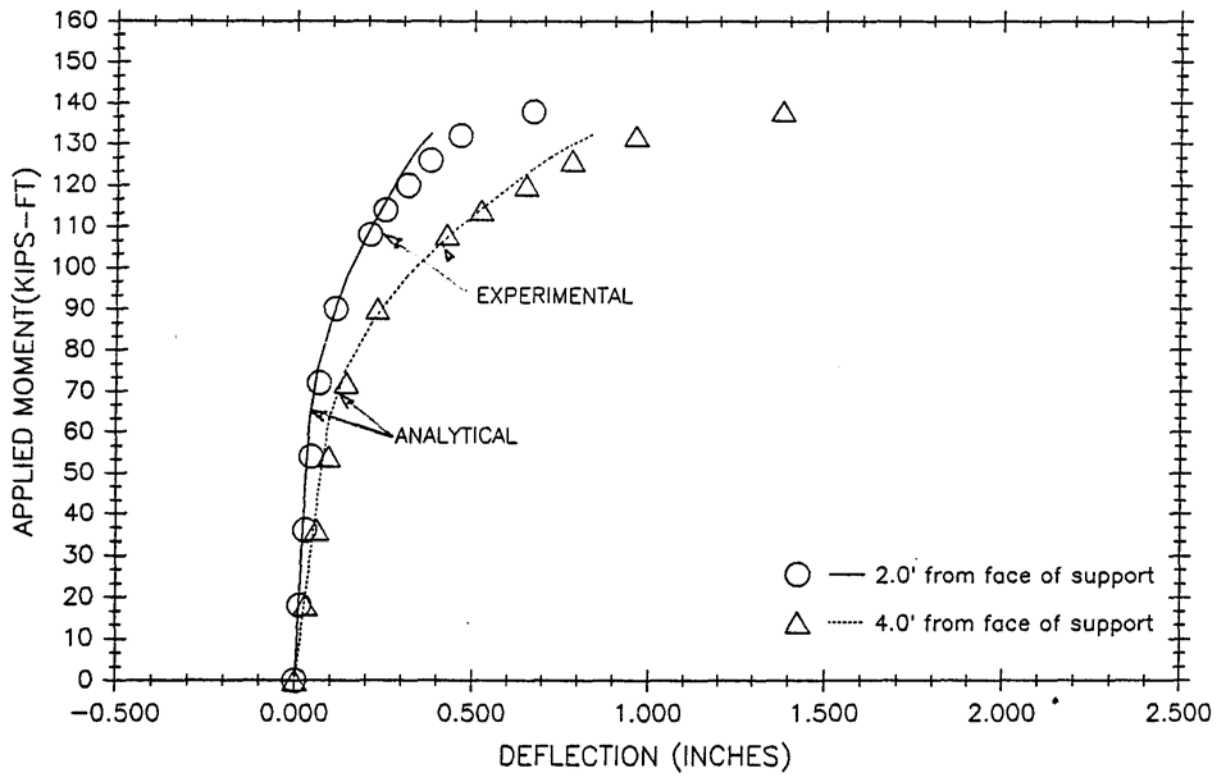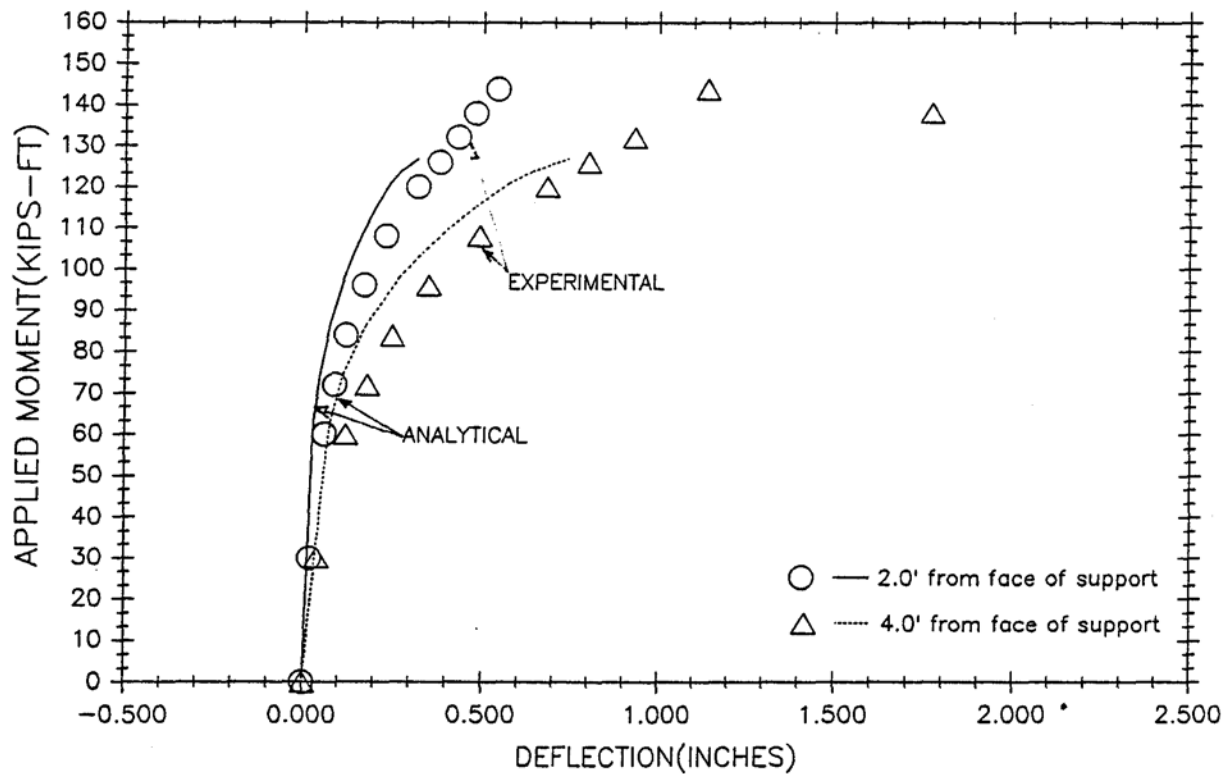
PILE A-1

42" EMBEDMENT

APPLIED MOMENT VS DEFLECTION
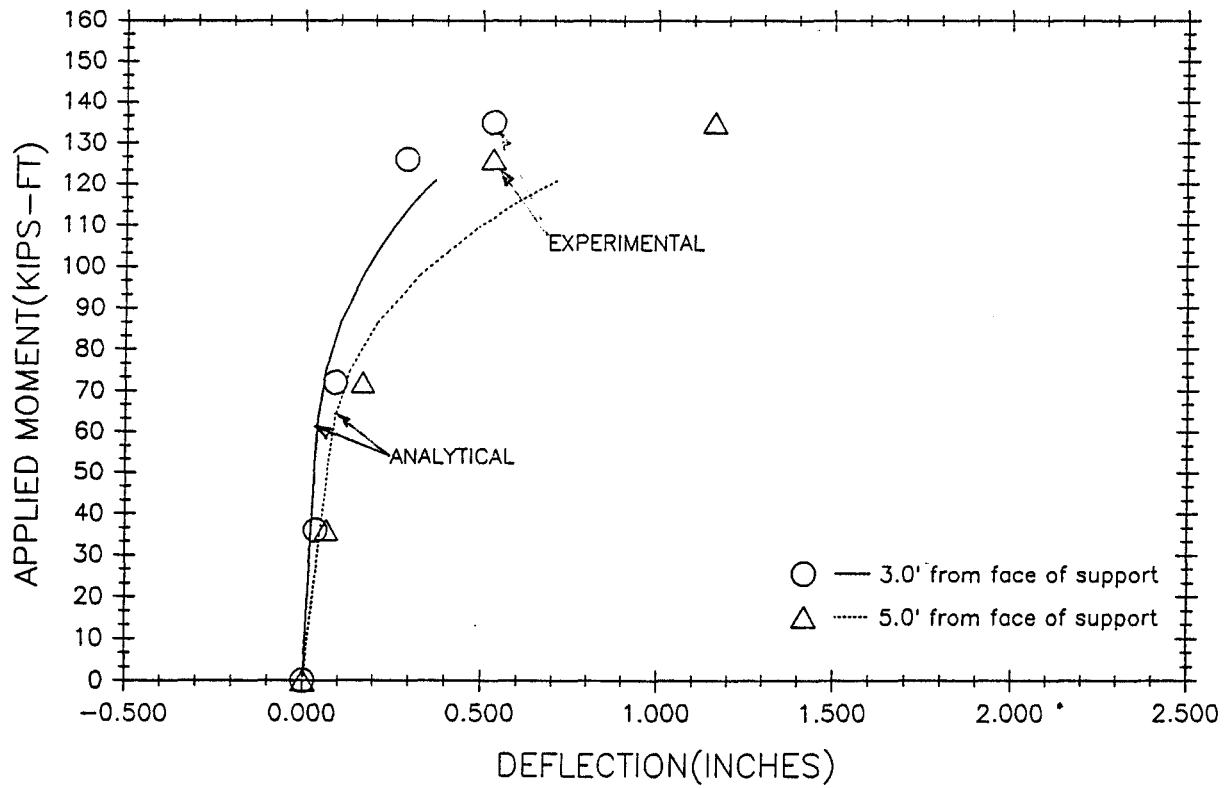
Figure A.4- Analytical and Experimental Curves

PILE B−1

42" EMBEDMENT
APPLIED MOMENT VS DEFLECTION
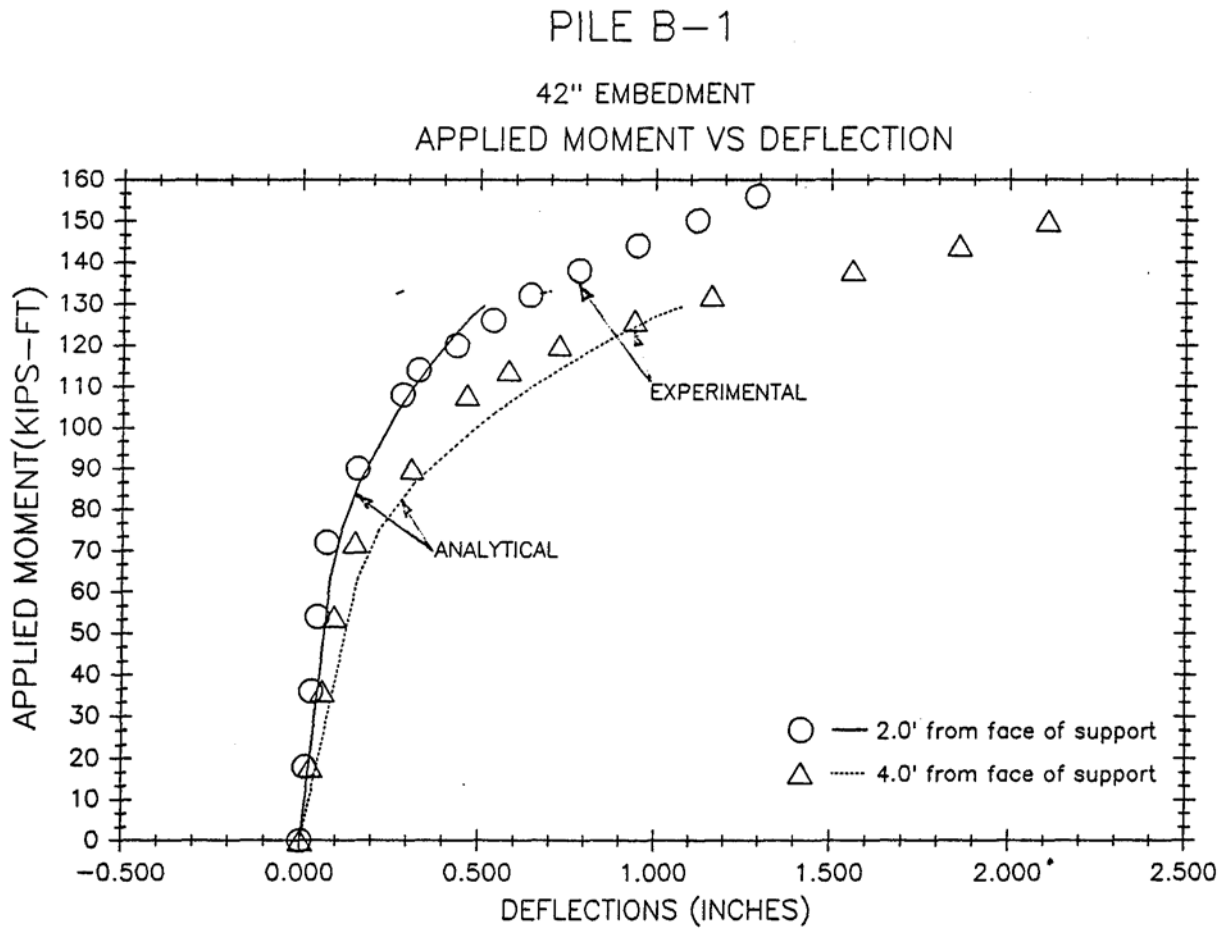
Figure A.5 Analytical and Experimental Curves

PILE B-3

42" EMBEDMENT

APPLIED MOMENT VS DEFLECTION
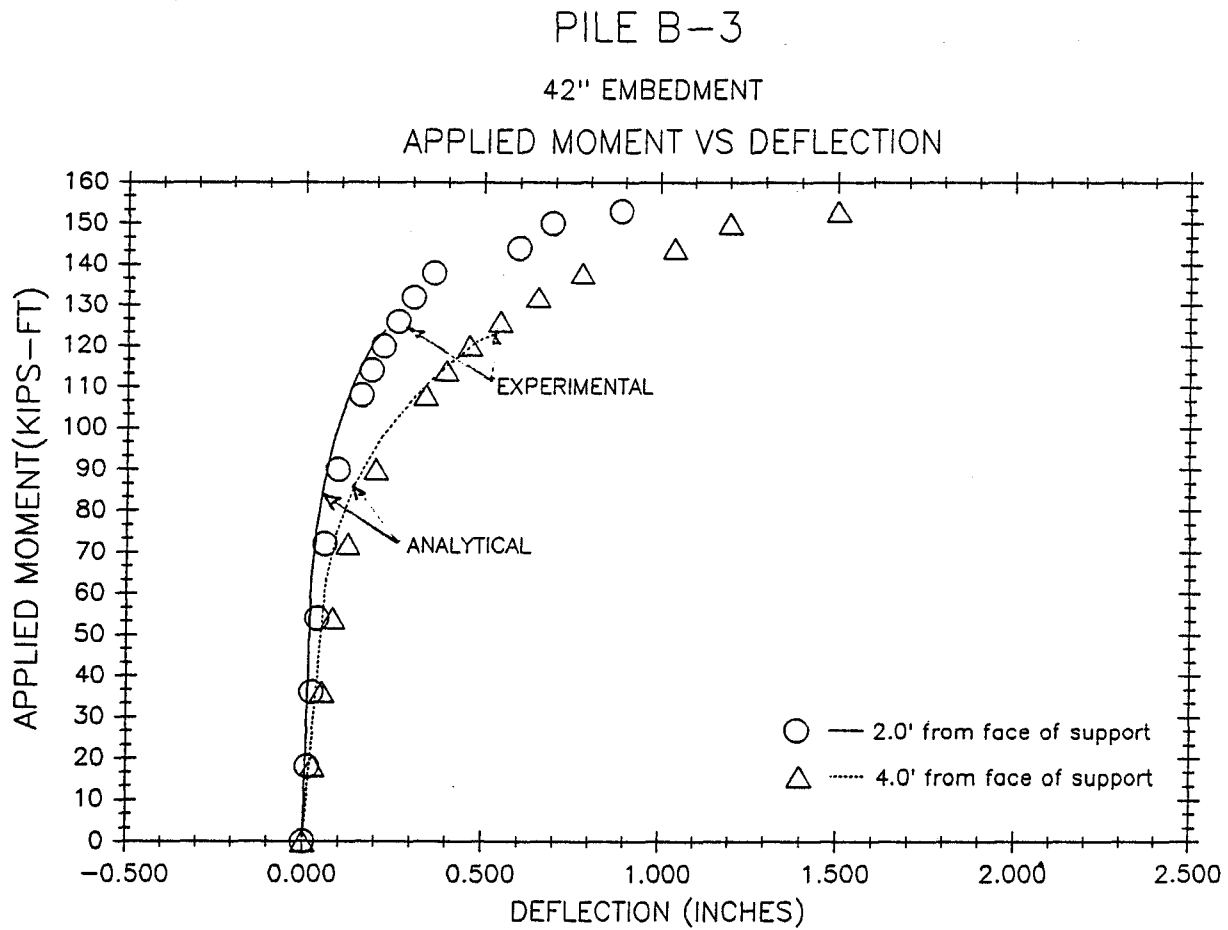
Figure A.6 Analytical and Experimental Curves

PILE A−6

42" EMBEDMENT

APPLIED MOMENT VS DEFLECTION
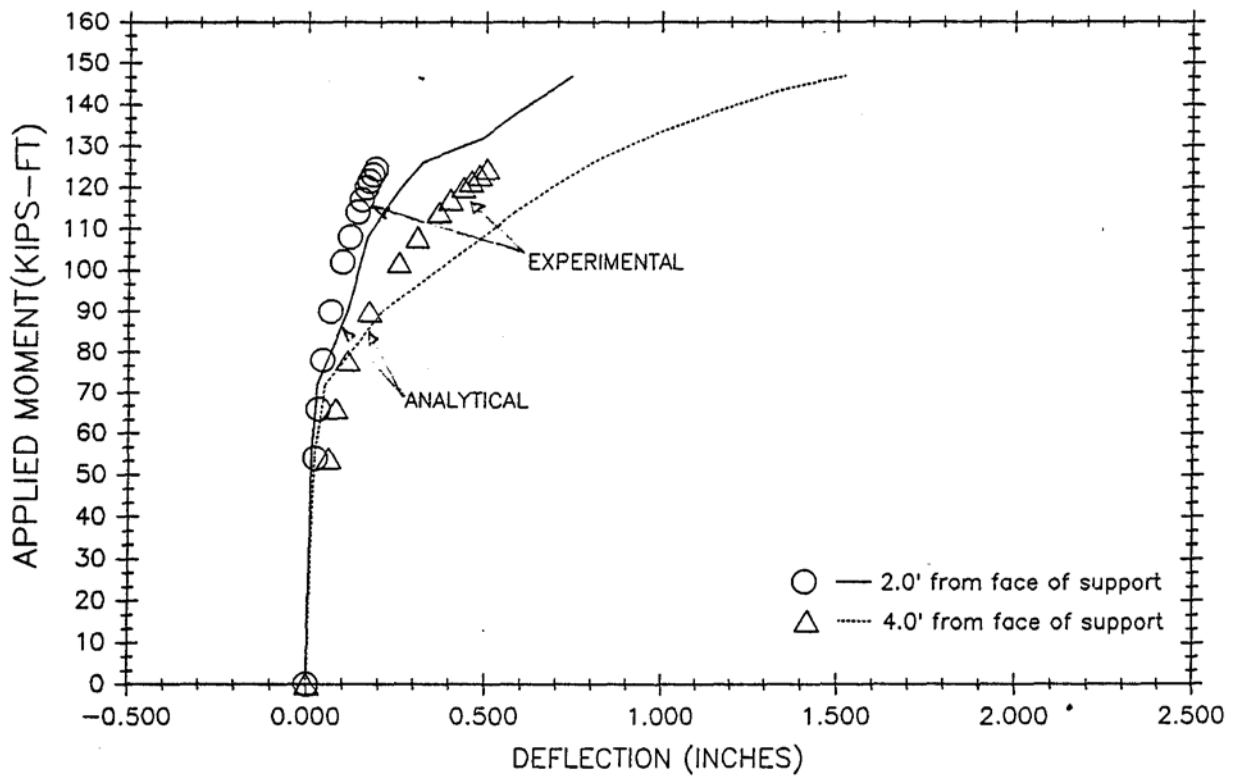
Figure A.7−Analytical and Experimental Curves

PILE B-8

42" EMBEDMENT
APPLIED MOMENT VS DEFLECTION
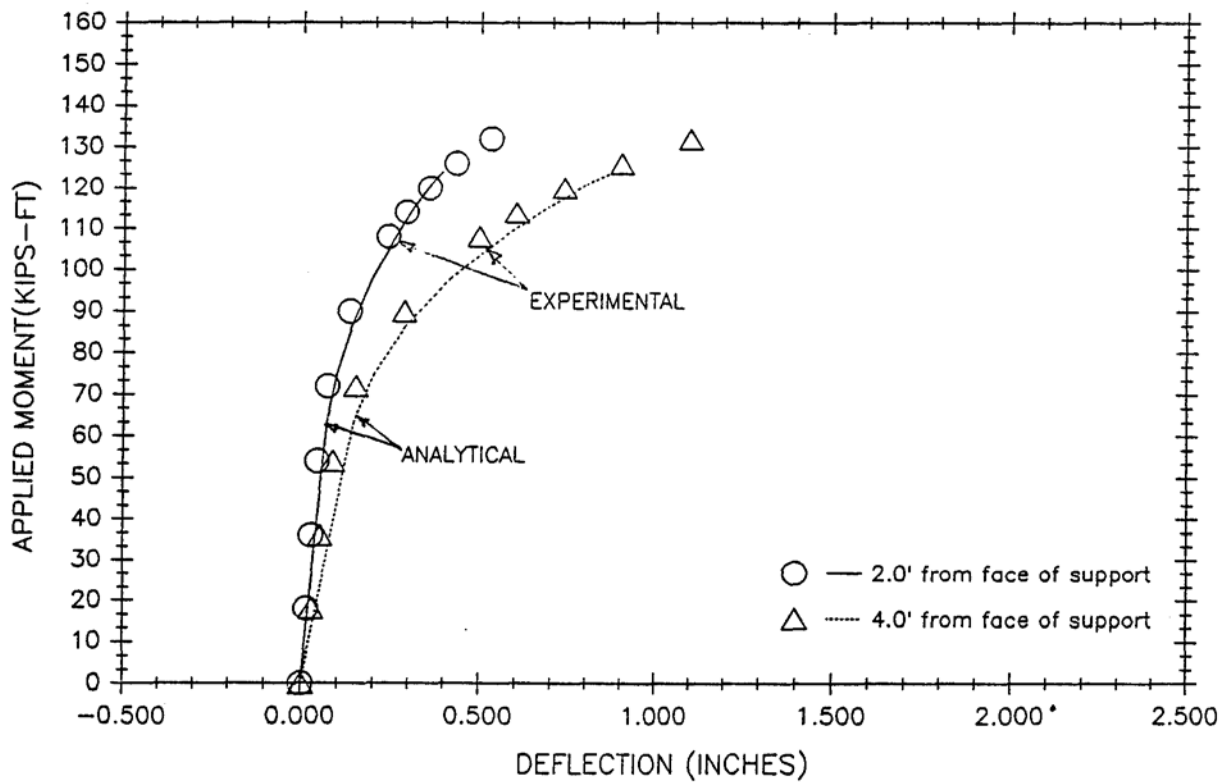
Figure A.8- Analytical and Experimental Curves

PILE B-5

48" EMBEDMENT

APPLIED MOMENT VS DEFLECTION
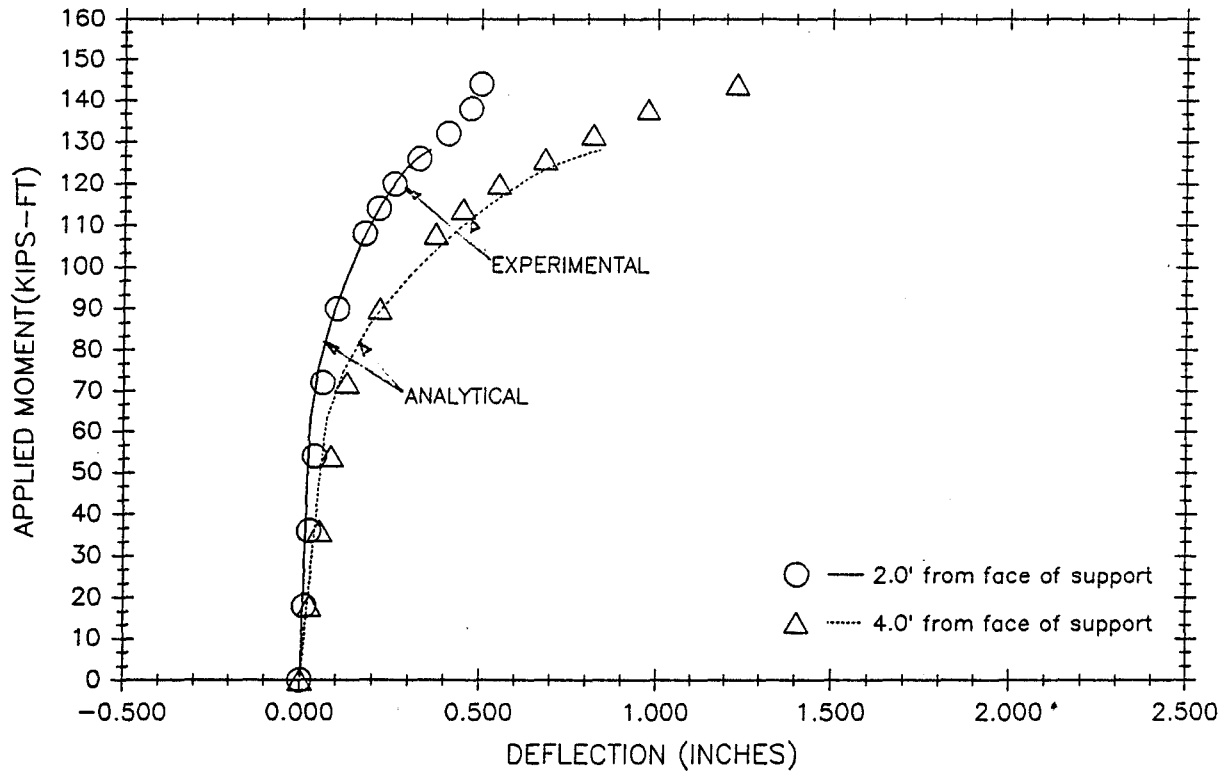
Figure A.9 Analytical and Experimental Curces

PILE A−2

48" EMBEDMENT

APPLIED MOMENT VS DEFLECTION

Figure A.10 Analytical and Experimental Curves

PILE B-6

48" EMBEDMENT

APPLIED MOMENT VS DEFLECTION

Figure A.11 Analytical and Experimental Curves

A-11

PILE B-6

48" EMBEDMENT

APPLIED MOMENT VS DEFLECTION

Figure A.11 Analytical and Experimental Curves

PILE C−31

48" EMBEDMENT

APPLIED MOMENT VS DEFLECTION

Figure A.13 Analytical and Experimental Curves

PILE C—1I

60" EMBEDMENT
APPLIED MOMENT VS DEFLECTION
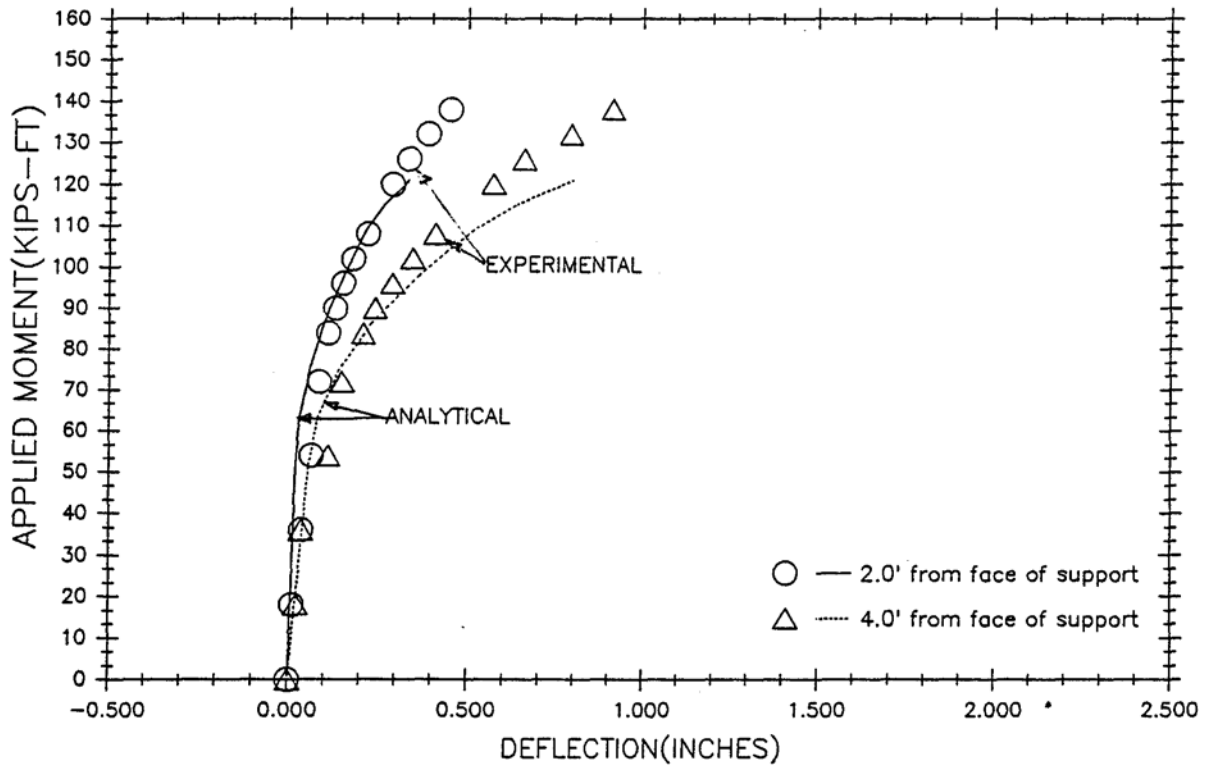
Figure A.14 Analytical and Experimental Curves
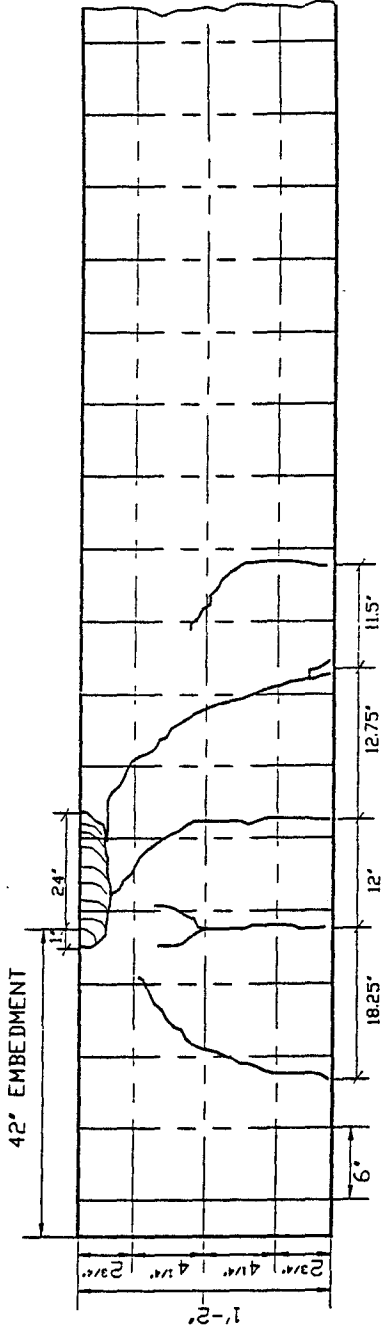
APPENDIX B

OBSERVED CRACKING - FAILURE DATA
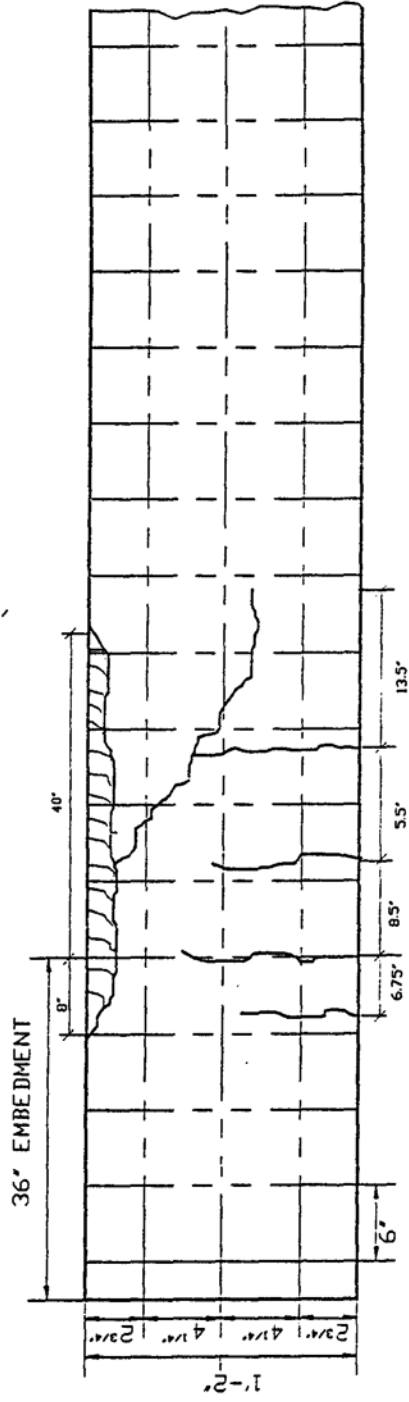
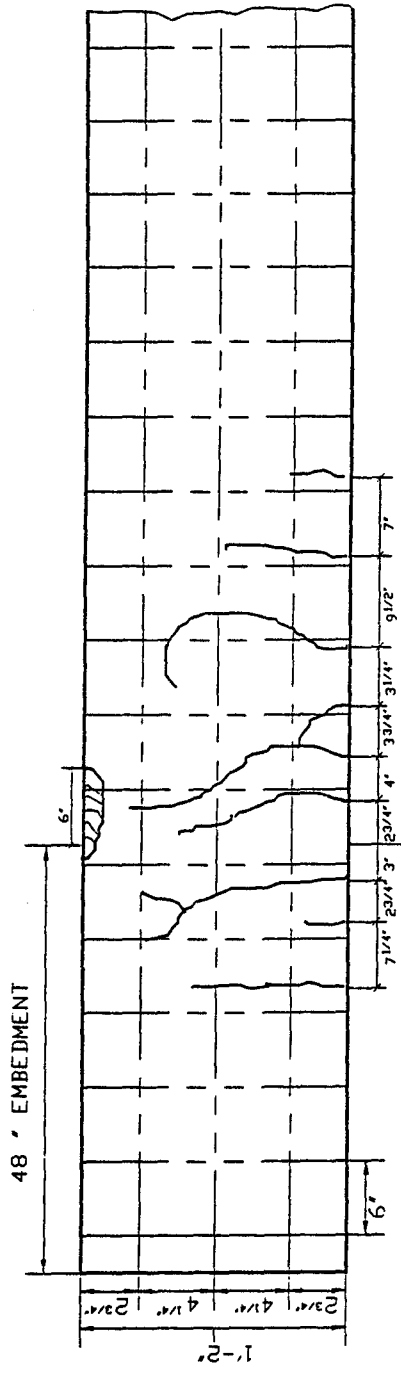EAST VIEW
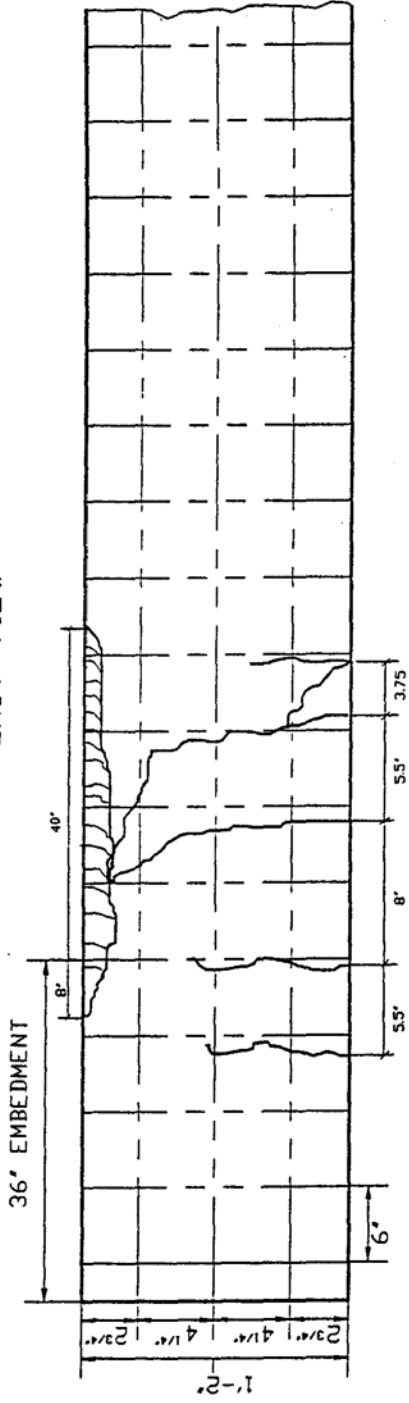
WEST VIEW

PILE TEST NO. B-2
CRACKING DATA

FIGURE B.1
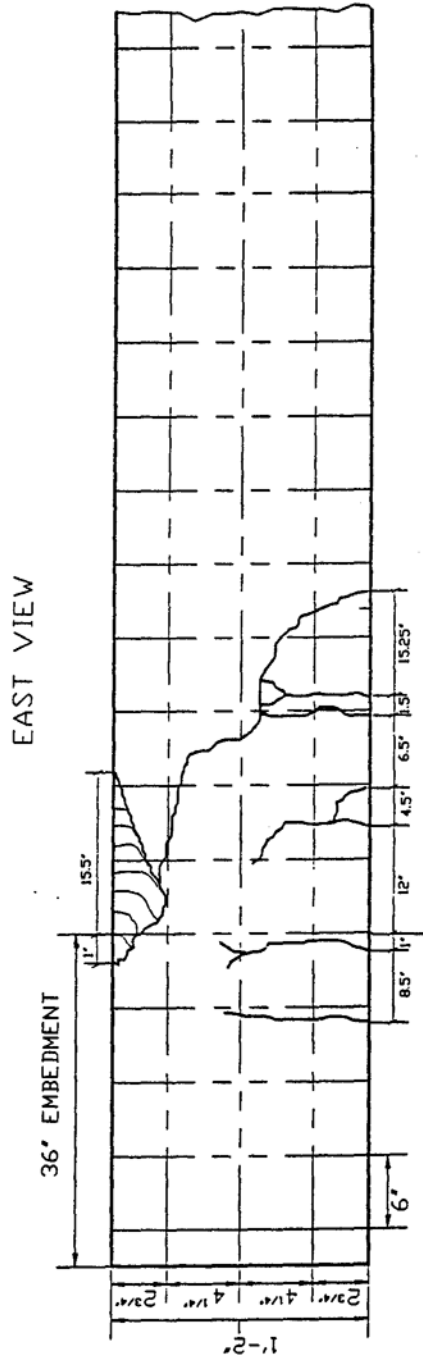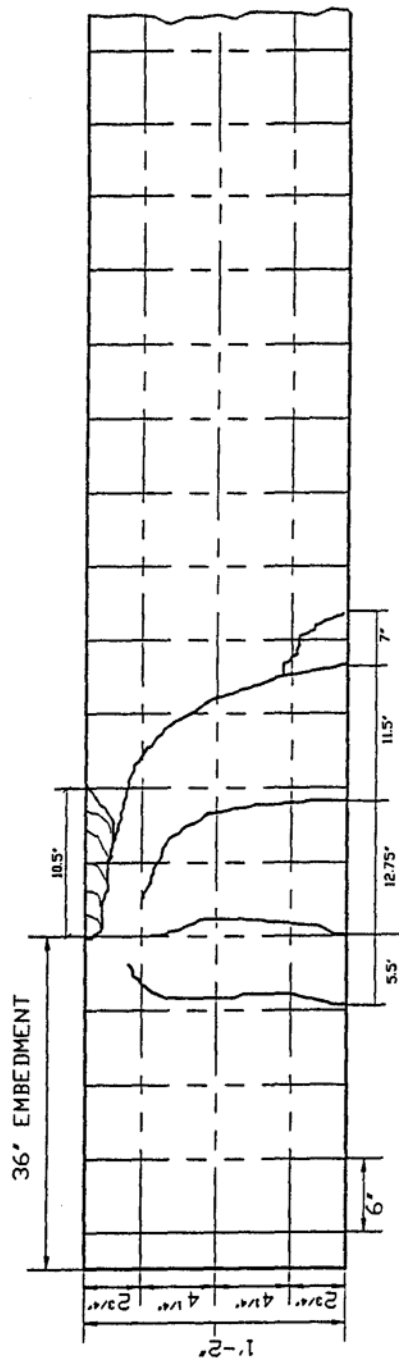
EAST VIEW

WEST VIEW

PILE TEST NO. B-7
CRACKING DATA

FIGURE B.2

EAST VIEW

3' EMBEDMENT

1'-2"
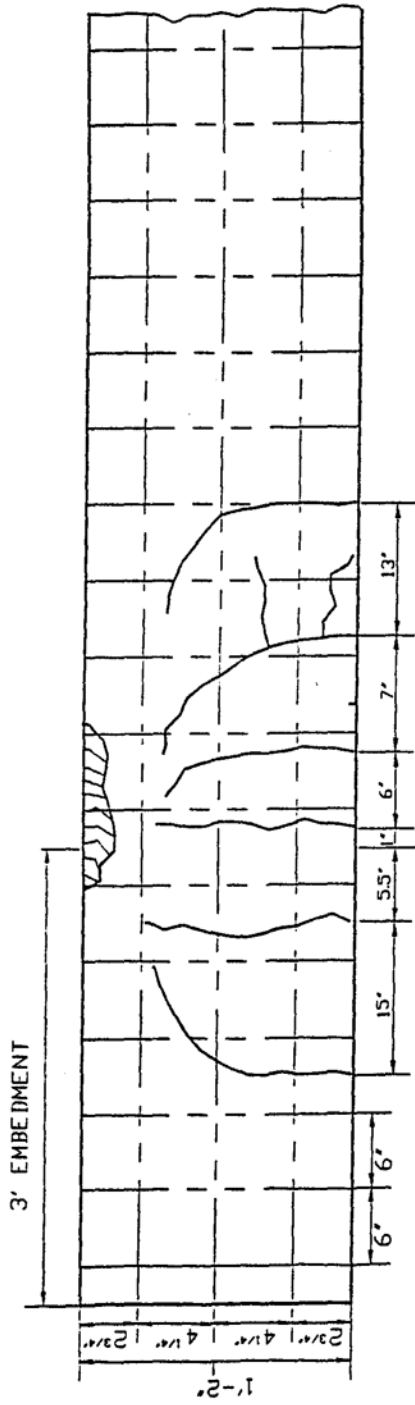
PILE TEST NO. A-4
CRACKING DATA

FIGURE B.3
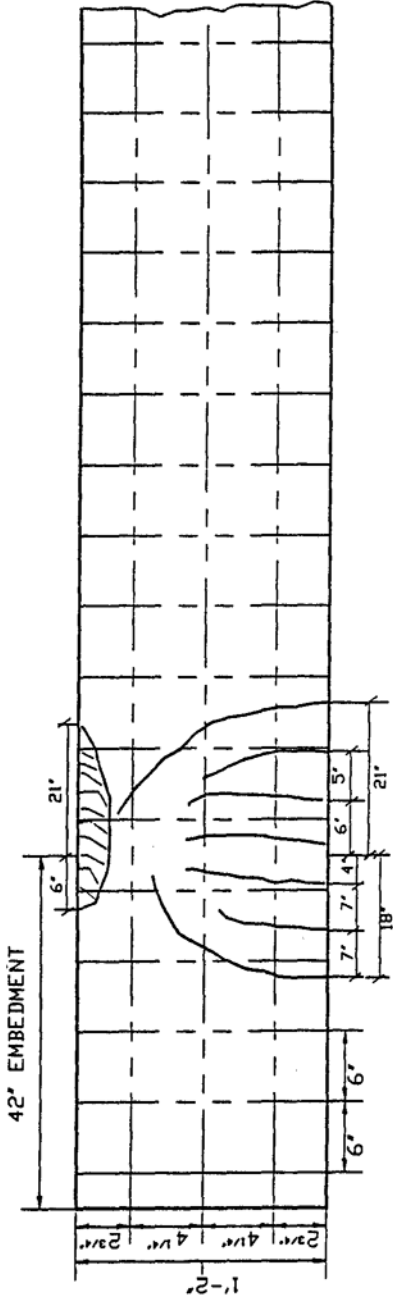
WEST VIEW

PILE TEST NO. A-5
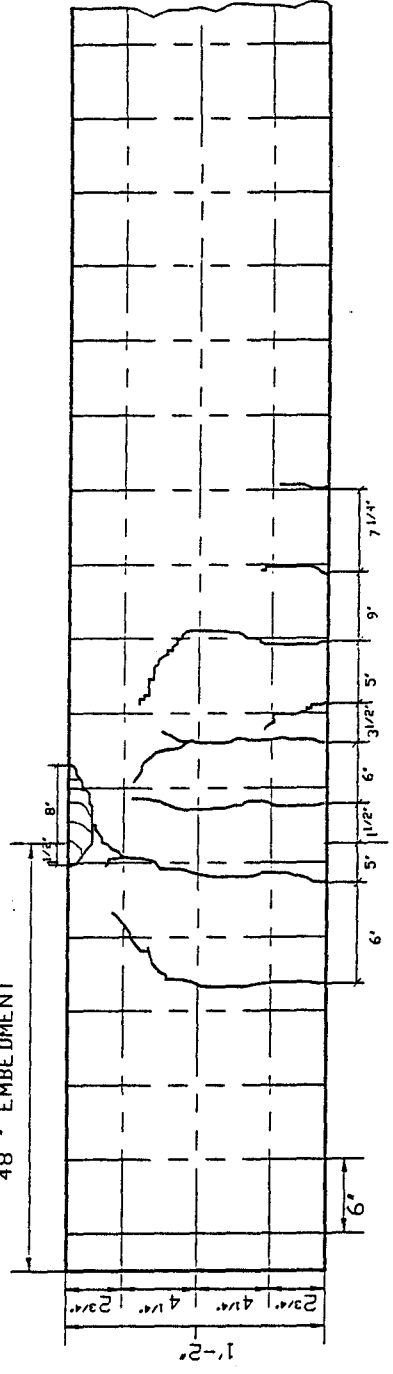CRACKING DATA

FIGURE B.4
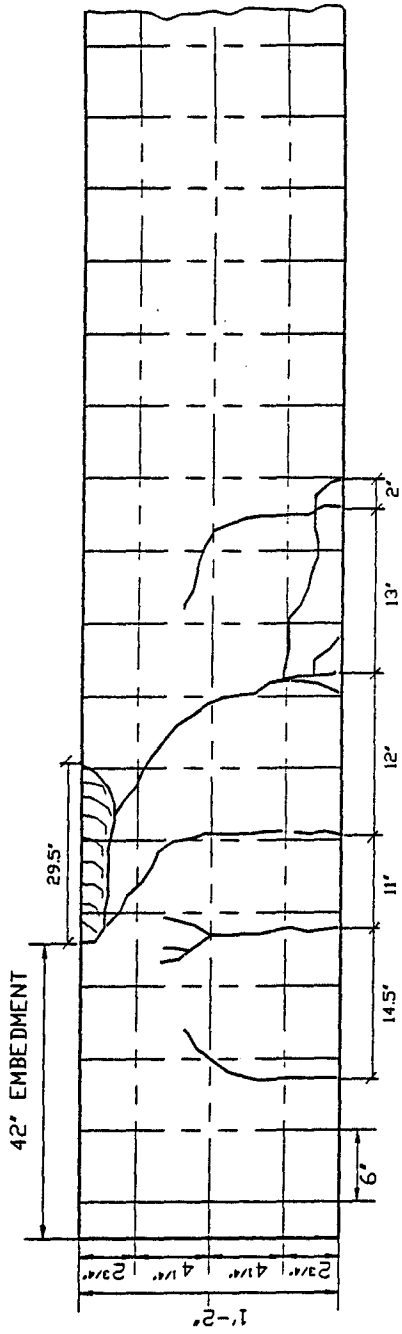
EAST VIEW

WEST VIEW

PILE TEST NO. B-9

CRACKING DATA

FIGURE B.5

EAST VIEW

WEST VIEW

PILE TEST NO. B-6

CRACKING DATA

FIGURE B.6