Report No. 98-9

Final Report

TESTING OF PILE-TO-PILE CAP MOMENT CONNECTION FOR 30" PRESTRESSED CONCRETE PIPE-PILE

By

Moussa A. Issa, Ph.D., P.E., S.E.
Senior Structures Design Engineer

Structures Design Office
605 Suwannee Street, MS 33
Tallahassee, FL 32399

Structures Research Group
Structures Design Office
Florida Department of Transportation

October 1999

DISCLAIMER

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

TESTING OF PILE-TO-PILE CAP MOMENT CONNECTION FOR 30" PRESTRESSED CONCRETE PIPE-PILE

By
Moussa A. Issa, Ph.D, P.E.
Senior Structures Design Engineer

Florida Department of Transportation
Structures Design Office, MS 33
Tallahassee, Florida 32399

1. General

The regions of the pile where high curvatures occur need to be designed to possess large ductility. Ductility is defined as the ability to undergo large deformations in the post-elastic range without significant reduction in strength. The concept in designing ductile structures capable of withstanding catastrophic forces, such as those resulting from earthquakes and vessel collision, is to dissipate the forces by ductile yielding at plastic hinge regions in the structure.

Piles supporting bridges in soft soil can be subjected to large displacements due to vessel collision or seismic loading. The resulting deflections can result in high curvature values in some regions of the pile. One region of high curvature is at the pile-to-pile cap interface where the bending moment is maximum due to the embedment of the pile into the stiff pile cap. A second region of high curvature occurs within the soil. These critical regions are illustrated in Figure 1 (figure was adopted from reference 6).

To enhance the ultimate bending capacity of the pile to pile-cap connection at the extreme event of vessel collision, the prestressed piles are supplemented with mild reinforcing bars to enhance the bending characteristics, especially at the pile-pile cap interface. This pile detail is very labor intensive and expensive to fabricate. Field cutting the pile to grade is difficult and time consuming as the reinforcing bars must be cut in addition to the usual pile.

Mr. Henry Bollmann, P.E. of the Structures Design Office recommended the pipe-pile to pile-cap moment connection as an alternative, much less expensive way to strengthen 30" square hollow core prestressed concrete piles. Dr. Moussa Issa, P.E. of the Structures Research Center designed and developed the details of the test specimen and supervised its fabrication, setup and testing. A pipe-pile splice detail for 30" square hollow core prestressed concrete piles was successfully tested on February 25, 1999 at the Florida Department of Transportation (FDOT)/Structures Research Center when composite action between the steel pipe and the prestressed concrete pile was established.

The moment connection is achieved by grouting a steel pipe inside the hollow core prestressed concrete pile and embedding this into the pile cap four (4) feet. This type of connection will enhance the moment capacity of standard prestressed concrete piles, particularly for the extreme event of vessel collision. Based on the results of the research and testing, the FDOT Structures Design Office pursued the development of the design, methodology and construction details of the steel pipe-pile to pile-cap moment connection. This report presents a summary of the results for the steel pipe-pile to pile-cap moment connection project.

2. Objectives of Study

The overall objectives of this study were as follows:

- Fabricate the test specimen.
- Install the steel pipe-pile to the pile cap.
- Test the moment capacity of the connection.
- Develop a field installation procedure.
- Develop design recommendations and connection details.

3. Engineers Present

The engineers and staff of the FDOT/Structures Research Center conducted the test in the presence of Florida DOT/Structures Design Office Engineers (Mr. Henry Bollmann, Mr. Jack Evans, Mr. Larry Sessions, Dr. Joe Bhuvasorakul, Ms. Hong Chen, Dr. Dongming White, and Mr. Ned Kawar), visiting professors from Canada (Dr. Omar Challal and Dr. Barrington deV Batchelor), visiting professors from the University of Florida (Dr. Cliff Hays and Dr. John Lybas) and an FHWA representative (Mr. Doug Edwards).

4. Design of the Pile Cap

To simulate actual field conditions, a pile cap was designed with dimensions 16.0 ft long, 6.5 ft wide and 4 ft deep and containing two 30 in. hollow-core piles spaced at 5.5 ft center-to-center. Appendix A shows the reinforcement and dimensions for the pile cap. The pile cap was designed to restrain the piles when subjected to a moment of 2000 kip-ft.

5. Steel Pipe-Pile to Pile Cap Fabrication and Testing

The step-by-step procedure for the fabrication of the steel pipe-pile to pile cap is presented in Appendix B. The illustrations show and describe the entire installation and fabrication procedure, including the test setup.

6. Instrumentation Test Setup and Test Procedure

Appendix C shows the instrumentation and the test setup for the test specimen. A hydraulic jack and two load cells were placed between the two piles at 16 ft from the face of the pile cap (see figures in appendix). This arrangement was selected so the shear and moment coexisting would simulate a typical vessel collision event in Florida waterways. The piles were then loaded incrementally until failure occurred. Failure was defined as slip of the prestressing strands or flexure failure due to yielding of the steel and crushing of the concrete in the compression zone at the face of the pile cap.

Deflection and strain measurements were recorded at specified load increments during the test. The load was applied in increments of 5 kips up to 100 kips. The load was then applied in smaller increments up to failure. During the test, cracks were marked at each load step to follow their development.

7. Design Material Properties

Concrete: Class V (special):
f'_c = 6,000 psi @ 28 days
f'_{ci} = 4,000 psi @ release
Non-Shrink Grout Strength f'_c = 6,000 psi

Prestressing Steel:
½" diameter LRS (special) strands
f_{pu} = 270 ksi.
E = 28E6 psi.

Reinforcement Steel (Grade 60 minimum):
f_y = 60 ksi.
E = 29E6 psi

Steel Pipe: 14" diameter steel pipe
Wall Thickness = ½"
f_y = 36 ksi (Minimum)

Corrugated Steel Pile Void Form:
18" O.D. Corrugated Metal Void (Corrugation Amplitude = ½")
Thickness = 0.052 in.
f_y = 50 ksi

Prestressed Piles:
30" square piles with 18" diameter hollow core
24-½" diameter LRS (special) strands stressed @ 32,307 # each

8. Pile Embedment Length

The results of this test and previous tests indicate that the required embedment length of the piles in the pile cap is much less than the length recommended in the ACI 318-95 Building Code. It is questionable whether the use of a strand development multiplier is necessary in the design of piles, which are embedded in the caps. Pile embedment into a pile cap presents a much different end condition than that encountered by a superstructure prestressed beams supported on bearings. In the case of the embedded pile, shrinkage of the confining concrete of the pile cap creates a clamping force that serves to reduce the development length of the strands. See reference 5 for more details on the effect of pile embedment length on prestressing strands.

9. Nominal Moment Capacities without Axial Load.

The nominal moment capacities for the 30" square hollow core prestressed concrete piles are presented in Table 1. Appendix D presents the computer program (RESPONSE) input and output files as well as the longhand calculations by MATHCAD. The author recommends the following approximate equation to calculate the nominal moment strength of the test piles:

$$M_n = (f'_c/f_{pu})^{\alpha} \cdot t \cdot [\, A_{ps} \cdot f_{pu} + \sum A_s \cdot f_y \,]$$

f'_c = concrete compressive strength, ksi

f_{pu} = ultimate strength of tendons, ksi

f_y = ultimate yield strength of steel, ksi

A_s = area of steel, in^2.

A_{ps}= total area of prestressing steel, in^2.

t = thickness of pile, in.

α = 0.27 for hollow core piles

α = 0.29 for solid piles or hollow core piles filled with concrete

Discussion of the recommended approximate equation:

(nominal moment strength) = (moment arm} ⋅ [force]

$$(M_n) = \{(f'_c/f_{pu})^{\alpha} \cdot t\} \cdot [A_{ps} \cdot f_{pu} + \sum(A_s \cdot f_y)]$$

M_n = nominal moment strength.

$\{(f'_c/f_{pu})^{\alpha} \cdot t\}$ = moment arm, the term $(\mathbf{f'_c/f_{pu}})^{\alpha}$ is to account for different concrete compressive strengths, and the term α is used to differentiate between hollow core and solid piles. (α = 0.27 for hollow core piles, and α = 0.29 for solid piles or hollow core piles filled with concrete.)

$A_{ps} \cdot f_{pu}$ = force, this term is the area of prestressing steel times the ultimate tensile strength of prestressing steel.

$\sum(A_s \cdot f_y)$= force, this term is the summation of all the areas of steel times the specified yield strength of the non-prestressed reinforcement.

The following three examples show how the equation can be applied for different condition. The results are compared with the experimental results and the code equation in Table 1.

<u>Example 1-Prestressed Piles Only (Case #1):</u>

f'_c = 10.13 ksi (Tested Concrete Strength)
f_{pu} = 270 ksi (Prestress Ultimate Stress)
A_{ps}= 4.01 in^2. (Area of prestressing steel)
t = 30 in. (Pile Thickness)

Solution:

$M_n = (10.13/270)^{0.27} * 30 * [(0.167*24)*270]$
$\mathbf{M_n}$ **= 13,380.1 k-in. (1,115.0 k-ft)**

Example 2-Prestress Pile + Void Form (Case #2):

f'_c = 10.13 ksi (Tested Concrete Strength)
f_{pu} = 270 ksi (Prestress Ultimate Stress)
A_{ps}= 4.01 in^2. (Area of prestressing steel)

18" corrugated metal void:
f_{y2} = 50 ksi (Steel Yield Strength)
A_{s2} = 2.94 in^2. (Area of Steel)
t = 30 in. (Pile Thickness)

Solution:
M_n = (10.13/270)$^{0.27}$ *30*[(0.167*24)*270 + 2.85*50]
M_n = 15,142.0 k-in. (1,261.8 k-ft)

Example 3-Prestress Pile + Void Form + Steel Pipe (Case #3):

f'_c = 10.13 ksi (Tested Concrete Strength)
f_{pu} = 270 ksi (Prestress Ultimate Stress)
A_{ps}= 4.01 in^2. (Area of prestressing steel)

14" steel pipe:
f_{y1} = 36 ksi (Steel Yield Strength)
A_{s1} = 21.21 in^2. (Area of Steel)

18" corrugated metal void:
f_{y2} = 50 ksi (Steel Yield Strength)
A_{s2} = 2.94 in^2. (Area of Steel)
t = 30 in. (Pile Thickness)

Solution:
M_n = (10.13/270)$^{0.29}$ *30*[(0.167*24)*270 + 21.21*36 + 2.85*50]
M_n = 23,025.7 k-in. (1,918.8 k-ft)

Table 1

Description	*Computer Program RESPONSE (k-ft)	**Calculated Moment (k-ft)	***Computer Program FLPIER (k-ft)	Approx. Moment by Issa (k-ft)	% Error Calculated To Approx (%)
Case #1: 30" square PC Pile with 18" diam. Hollow Core.	1,109	1,117	1,190	1,115	0.20 %
Case #2: 30" square PC Pile with 18" O.D. Corrugated Metal Void, Form.	1,249	1,262	1,300	1,262	0.03 %
Case #3: 30" square PC Pile with 18" O.D. Corrugated Metal Void and 14" diam steel Pipe filled with concrete and grouted into the Pile.	1,854	1,904	1,860	1,919	0.80 %

Note 1: The nominal test moment was **1921 k-ft.**

*The theoretical moment was based on strain compatibility using a computer program called RESPONSE.

**The calculated moment was obtained by long hand (neglect the effect of compression steel).

***The calculated moment was based on strain compatibility analysis, FLPIER.

Note 2: The Moment increase was based on case No. 1.
% increase = (1921)/{(1117+1109)/2} = **72.6%**

10. Discussion of Results

The experimental stress strain plots of the prestressed concrete pile and the pile cap are presented in Figures 2a and 2b, respectively. The results indicated an average compressive stress of 10,129 psi for the pile and 8,990 psi for the pile cap. These compressive strength values are much higher than the design values of 6,000 psi.

Figures 3 and 4 show the measured moment versus lateral deflection at the loading point, and the deflection along the pile for specified moments. The maximum deflection was 4.161 in. at the maximum test moment. **There was no slip in the strands at the maximum test moment (moment = 1921 k-ft).**

Figures 3 through 11 show typical results of strains at different locations on the pile and the pile cap.
The cracking and failure of the test specimen concentrated in the critical region of the pile and not in the pile cap (see Figure 12). Flexure failure occurred long after the initial cracking moment of the pile was reached. A flexural failure was characterized by considerable flexural cracking, yielding of steel and, finally, crushing in the concrete in the compression zone at the point of maximum moment.

The calculated nominal capacity of the pile without the 14" diameter steel pipe and the 18" diameter corrugated metal void was 1,117 k-ft. The computer program FLPIER, calculated nominal capacity of the pile with the 14" diameter steel pipe and the 18" diameter corrugated metal void was 1,920 k-ft. The nominal moment test capacity observed in the test was 1921 k-ft. The nominal moment by using the recommended approximate equation by the author (Issa) was 1919 k-ft. Therefore, the test moment connection was 1% higher than the calculated moment and less than 1% by the approximate equation. In conclusion, an increase of 72.6% was achieved in the moment connection of the pipe-pile to pile cap by using the recommended detail in Figure 13. Appendix E presents the remainder of the experimental plots and results.

11. Conclusions and recommendations

The following conclusions can be drawn from this study:

- The calculated nominal moments of the pile sections match the test moments. The nominal test moment capacity (M_{test} = 1,921 k-ft.) in the test was 1% higher than the calculated moment ($M_{calculated}$= 1,904 k-ft). The calculated nominal moment by FLPIER was 1,860 k-ft.

- The non-prestressed longitudinal steel (steel pipe) is required to enhance and increase the moment connection strength in the pile end and the pile cap. A 72.6% increase in the pipe-pile to pile cap moment connection was attained by using the test detail. (see figure 13)

- The formation of a plastic hinge between the pile and the pile cap interface was very visible from the ductile failure of the pile.

- The pile embedment length in a footing or a pile cap has a significant, positive influence on the average bond stress. An embedment length of 48 in. was enough to develop the full flexural capacity of the section without any slippage of the strands.

- Shrinkage of the confining concrete in the pile cap creates a clamping force that reduces the prestressing strands development length. An average confining strain of 112 microstrain (stress = 510 psi) occurred at a depth of 24 in. from the face of the pile cap.

- The experimental average bond stress of 510 psi is much higher than the assumed value of 250 psi by the ACI Code.

- The current ACI/AASHTO code equation for strand development length is very conservative for prestressed concrete piles embedded in concrete.

- The provided transverse reinforcement spacing of 4 in. in the pile at the pile cap face (4 ft from the pile end) was adequate to ensure a ductile failure with the higher capacity moment connection.

- Composite action between the concrete and the steel was developed in this section. The ductile mode of failure and the results prove this conclusion.

- The nominal moments of the recommended equation by the author were within 1% of the nominal test results and the calculated values by the code (see Table 1).

The following recommendations can be drawn from this research project:

- An embedment length of 48 in. is adequate to develop the flexural strength of such piles without slippage of the strands.

- Use 12 ft long, 14" diameter steel pipe (thickness = 1/2") with minimum yield strength of 36 ksi.

- Figure 13 presents the recommended pipe-pile to pile cap moment connection detail.

- An average bond stress of 510 psi can be used within the embedment length region. It is recommended to wash the prestressed concrete pile surface in the pile cap region for a better bond. A roughened surface may be used to enhance the tension capacity of the pile if needed.

- Fill the 12 ft long, 14" diameter steel pipe with concrete before it is placed inside the prestressed concrete pile. (optional-contractor can use other methods)

- Use standard design practice to calculate the ultimate design moment $(M_u)=\Phi M_n$, with Φ varies from 0.70 to 0.90.

- Use the nominal test moment for piles designed for vessel collision. $\mathbf{M_{recommended} = 1921}$ **k-ft.**

- The nominal moment of piles can be calculated by the author's recommended equation :

$$M_n = (f'_c/f_{pu})^{\alpha} \cdot t \cdot [\, A_{ps} \cdot f_{pu} + \sum A_s \cdot f_y \,]$$

These recommendations are only valid for prestressed concrete piles that are embedded in reinforced concrete pile caps with a minimum distance of 4 feet.

12. Acknowledgments

This project was made possible through the support and cooperation of many individuals. The following members of the Structures Research Center assisted in this project: M. Shahawy, T. Beitelman, A. El-Saad, F. Cobb, A. Fishburn, G. Johnston, S. Eudy, S. Cai, S. Hurston, J. Lewis, T. Brown, J. Witherspoon and S. Beddy. The author thanks them all. Special thanks are due to H. Bollmann of the Structures Design Office for his idea and valuable input in this research project.

13. References

1. ACI Committee 543. "Recommendations for Design, Manufacture, and Installation of Concrete Piles", ACI 543R-74, ACI Manual of Concrete Practice 1981, Part 4, ACI, Detroit, 1981, pp. 543R-1 to 543R-39.

2. Gerwick, Ben C., Jr. Construction of Prestressed Concrete Structures, Text book, John Wiley and Sons, Inc., New York 1971.

3. Gerwick, Ben C., Jr. Prestressed Concrete Piles, PCI Journal, October 1968.

4. Collins, M., Mitchell D. Prestressed Concrete Structures, Text book, Prentice Hall, Inc., NJ 1991.

5. Shahawy, M. and Issa, M. "Effect of Pile Embedment of the Development Length of Prestressing Strands", PCI Journal, Prestressed Concrete Institute, November-December 1992, pp. 44-59.

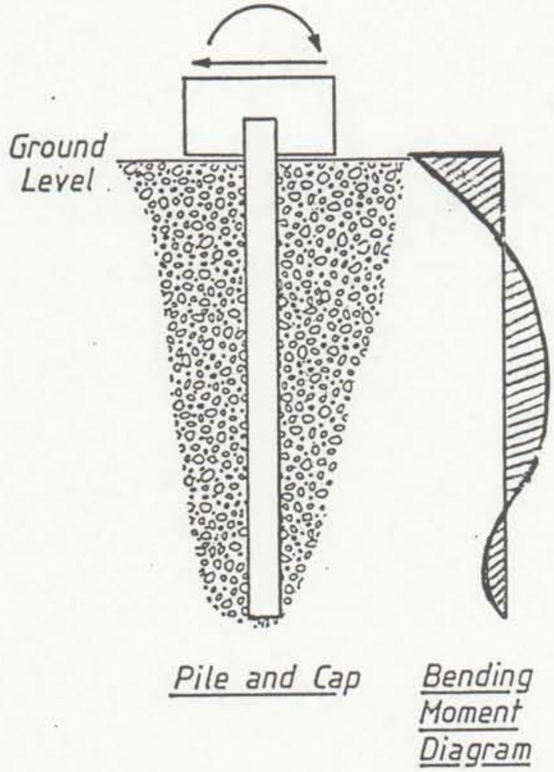
6. Joen, P. and Park, R. "Simulated Seismic Load Tests on Prestressed Concrete Piles and Pile-Pile Cap Connection", PCI Journal, Prestressed Concrete Institute, November-December 1990, pp. 42-61.

7. Cook, Jack R., Severe Foundation Problem Solved Using Long Precast Prestressed Concrete Piles, PCI Journal, January/February 1974.

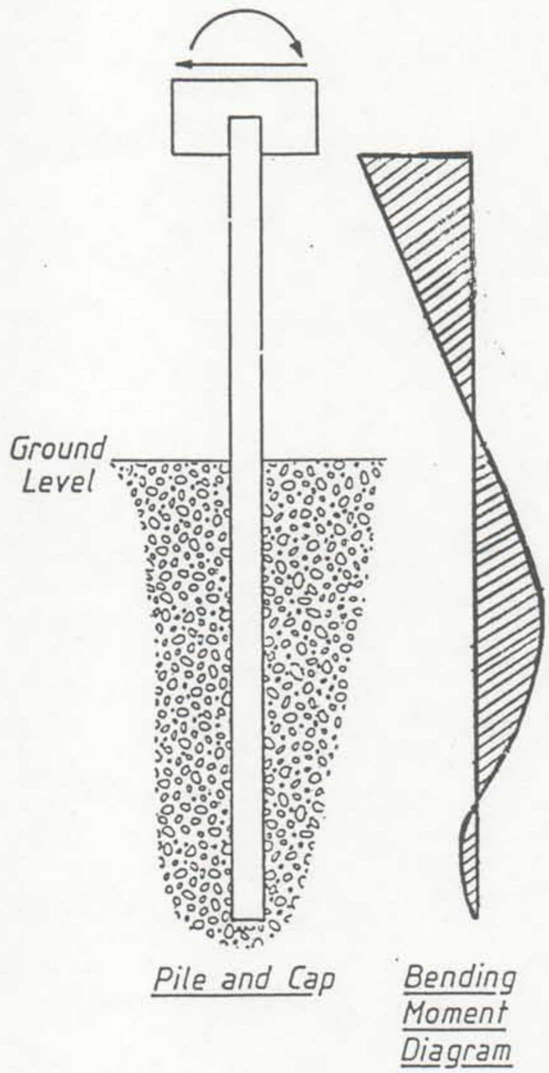
8. FDOT Construction Office. Special Provisions to the Standard Specifications-A4550512, Concrete Piling - Extensions and Buildups, Subarticle 455-5.12, pp. 1-50.

9. FDOT Structures Design Office. Plan Sheet Index No. 600, 12", 14", 18", 20", 24" and 30" Prestressed Concrete Piles, Florida DOT, May 1991.

10. FDOT Structures Design Office - District IV. Plan Sheet AB-5, Roosevelt Bridge - 30" Prestressed Special Concrete Pile Detail, Project number 89010-3541/3548, July 1993.



(a) Pile Totally Embedded in Soil



(b) Pile Partially Embedded in Soil

Figure 1-Bending diagrams of long piles due to horizontal loading.
(adopted from reference 6)

Figure 2a
Pile Moment Connection
Compressive Strength
Pile Specimen No.2

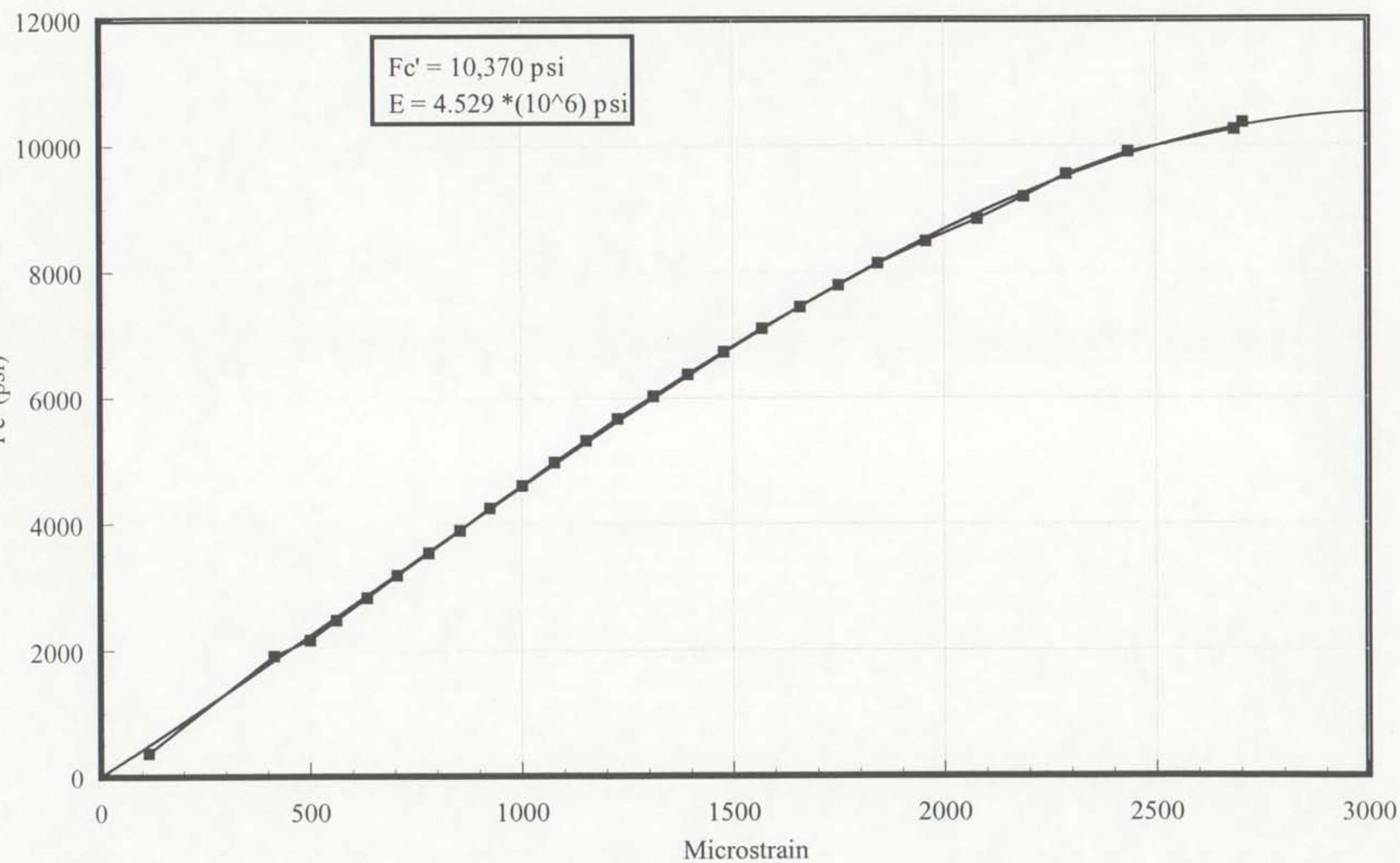


Figure 2b
Pile Momnet Connection
Compressive Strength
Pile Cap Specimen No. 2

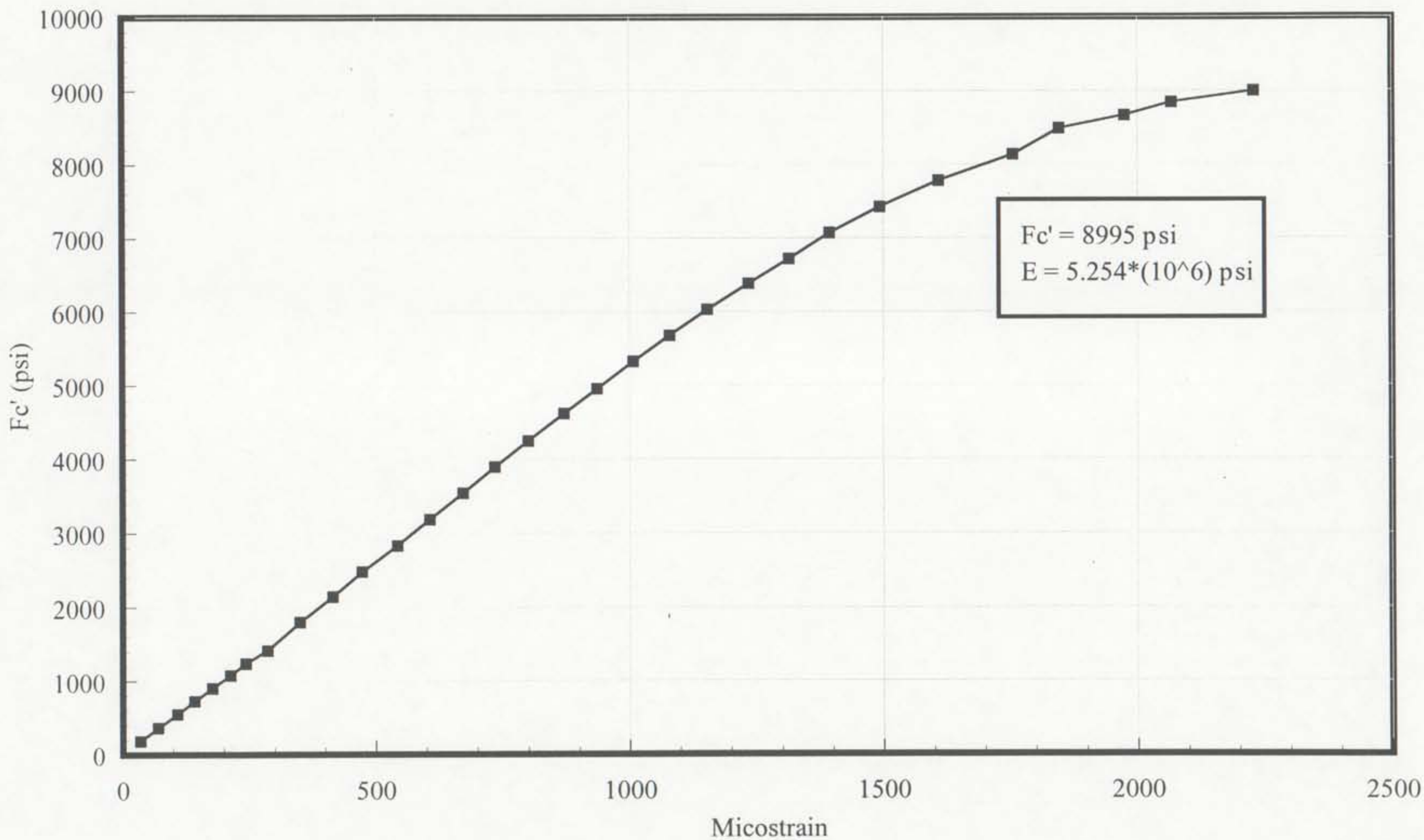


Figure 3

Moment vs Deflection Along East Pile

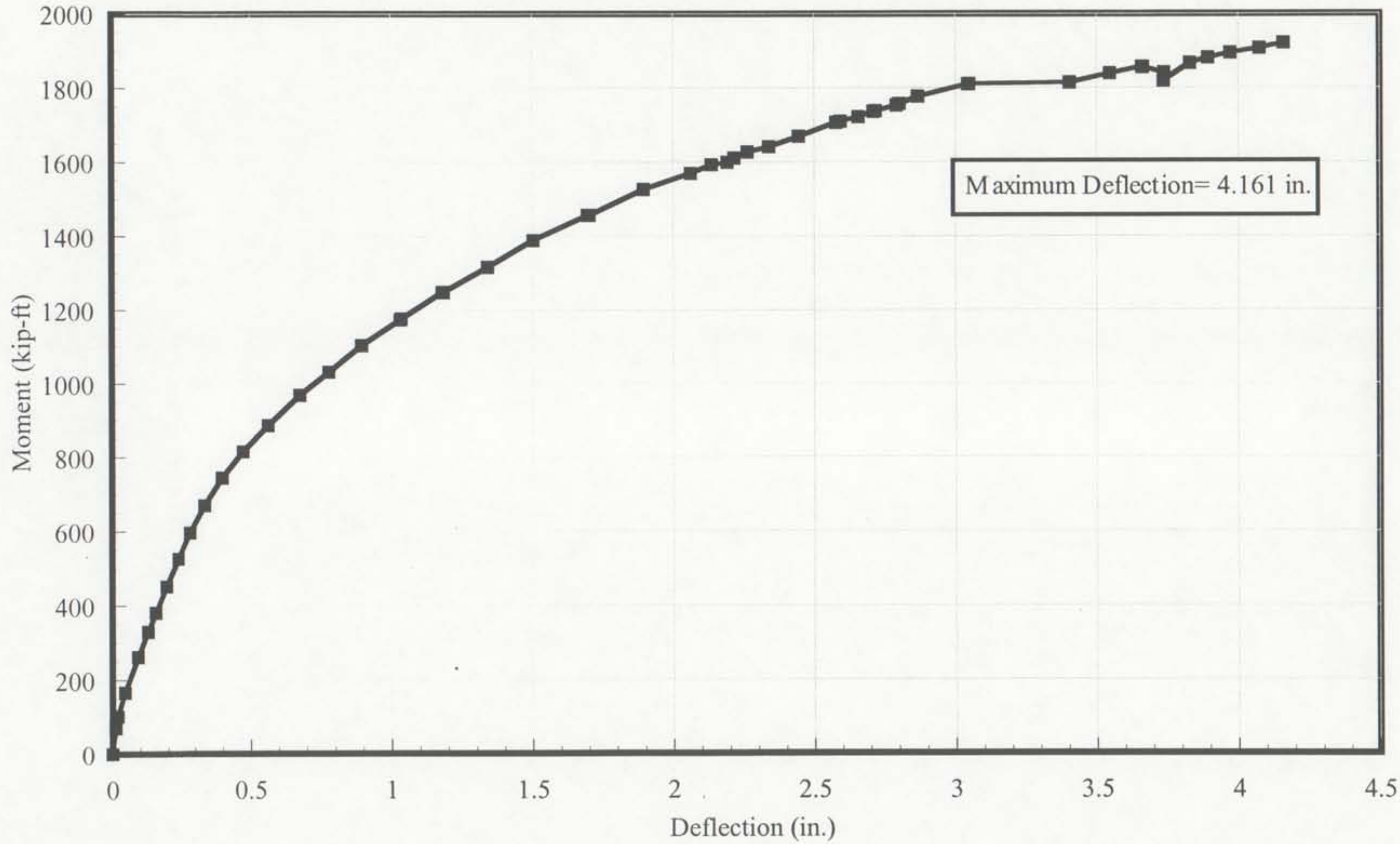


Figure 4

Deflections at Distributed Moments Along East Pile

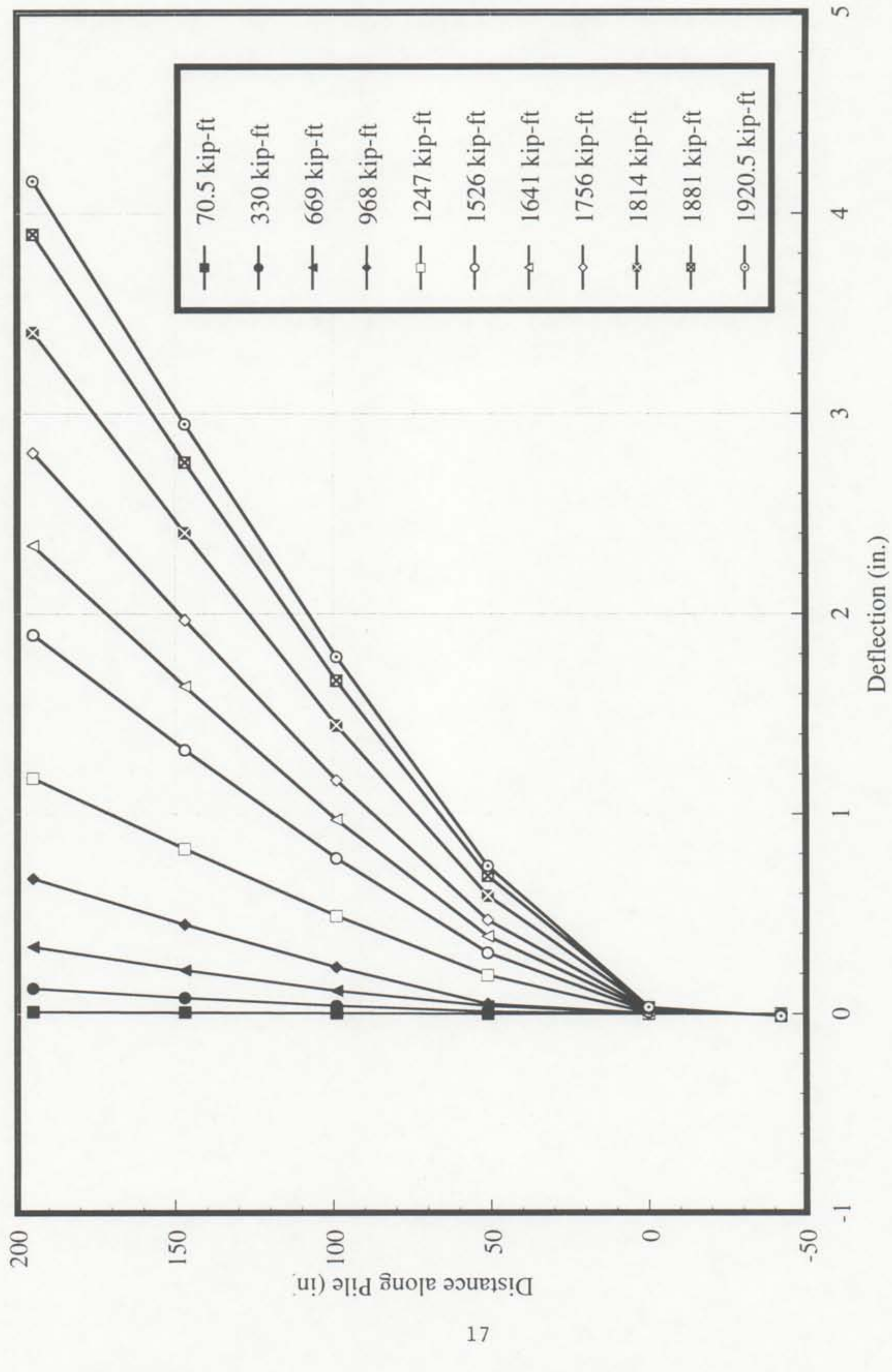


Figure 5

Moment vs Strain East Pile Compression Face Surface Gages

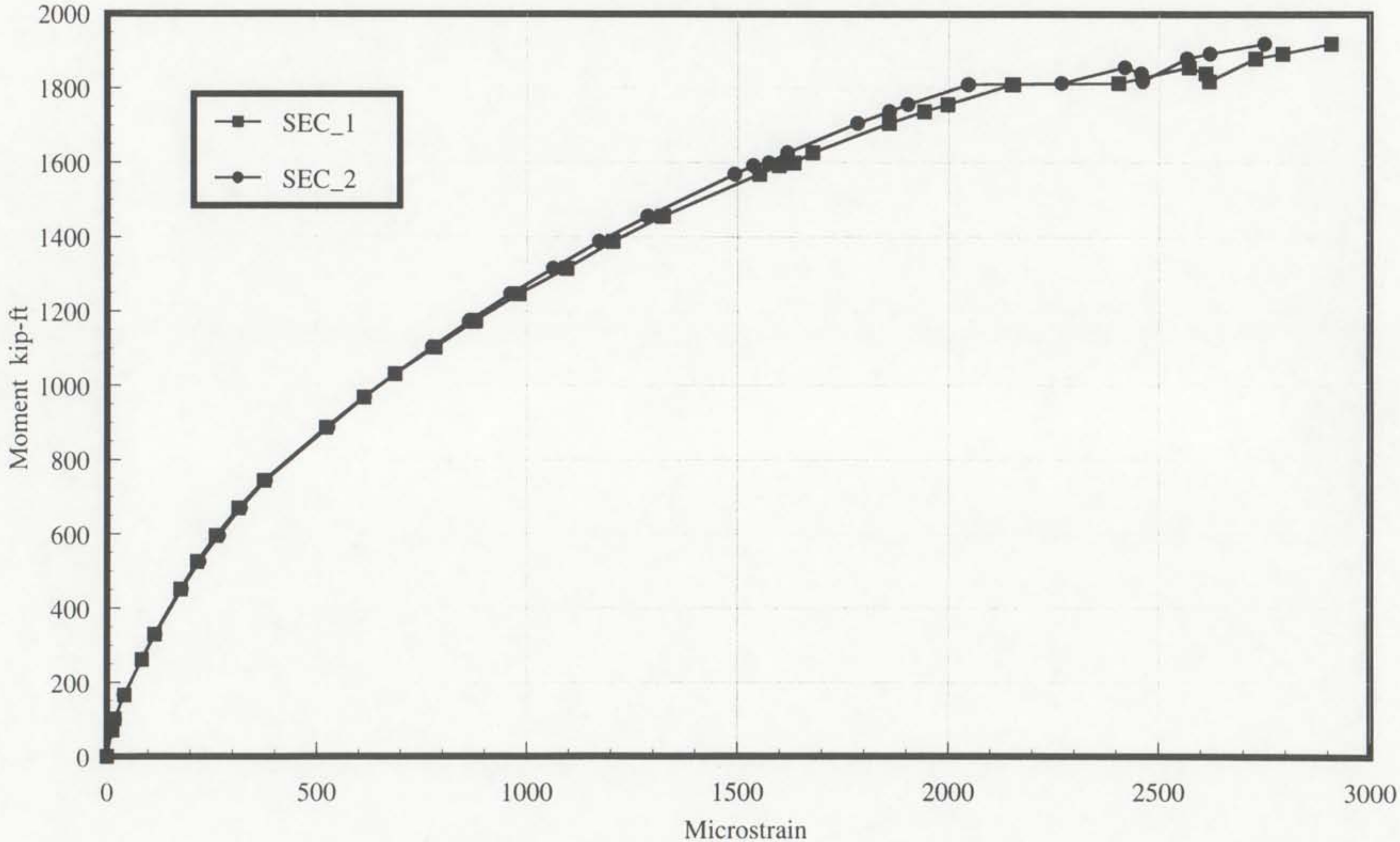


Figure 6

Moment vs Strain West Pile Embedded Gage ELW_11

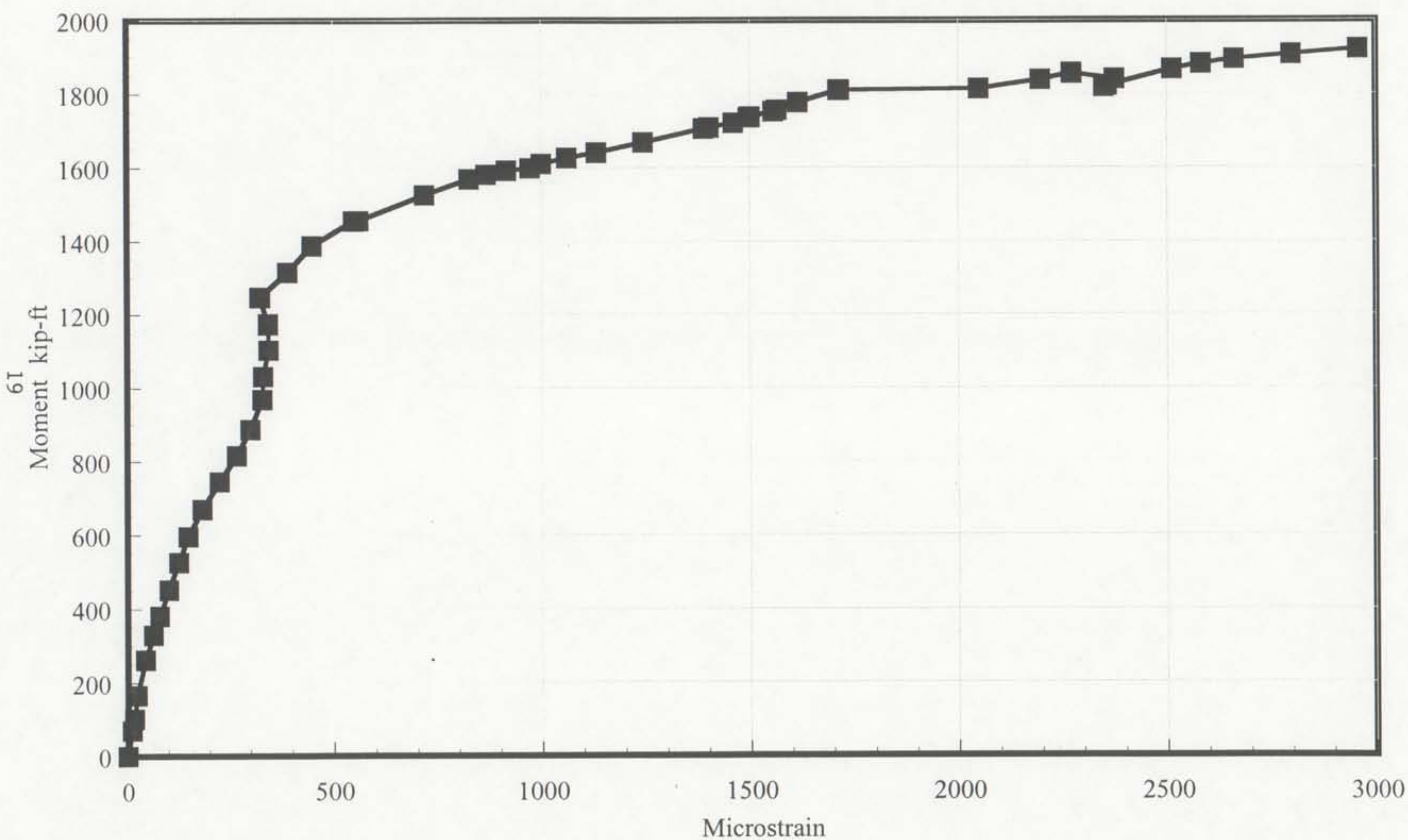


Figure 7

Moment ve Strain Pile Cap Embedded Gage (CAP_8) over West Pile

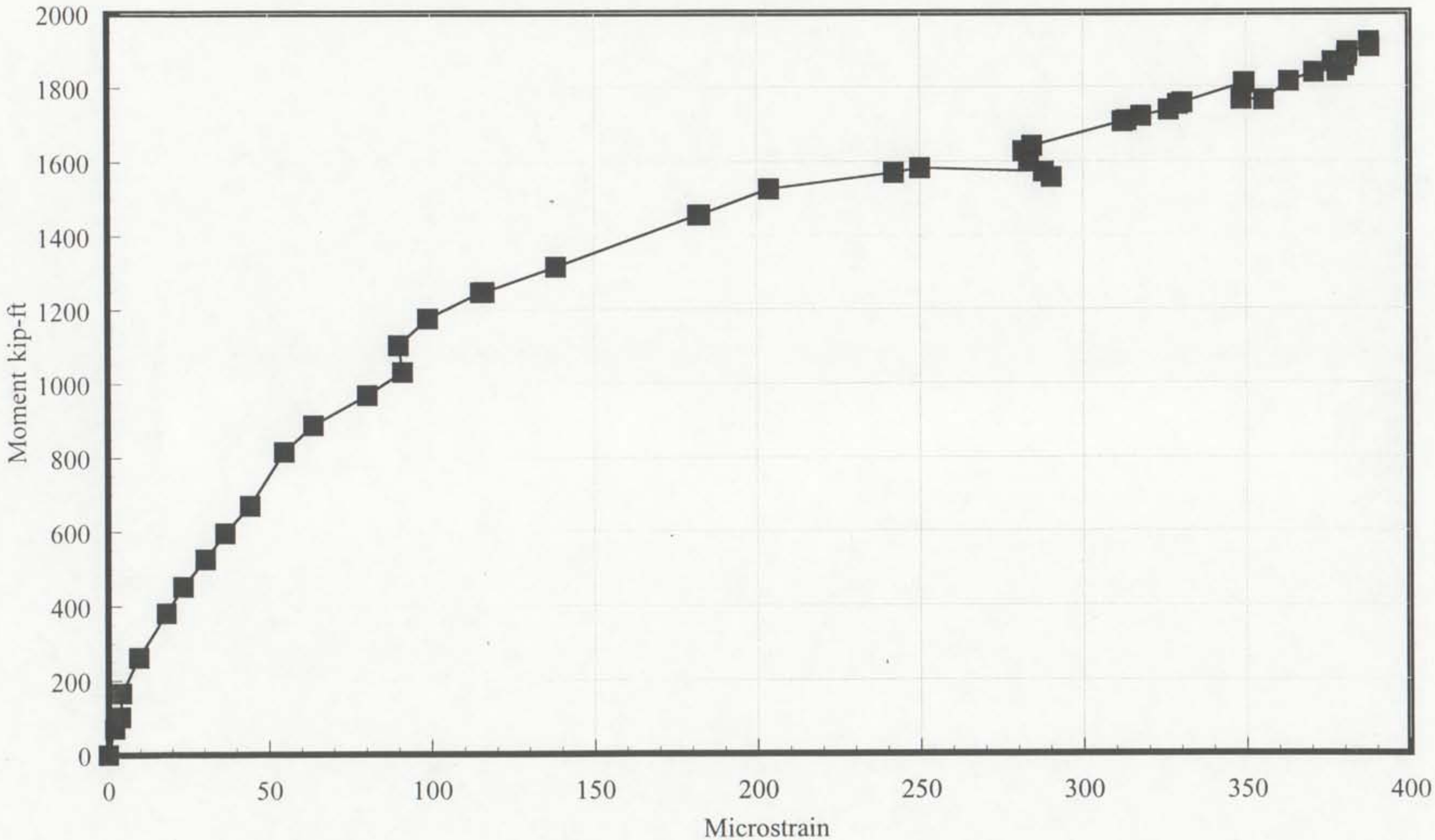


Figure 8

Moment vs Strain Pile Cap Crack Gage (CRC_4) over East Pile

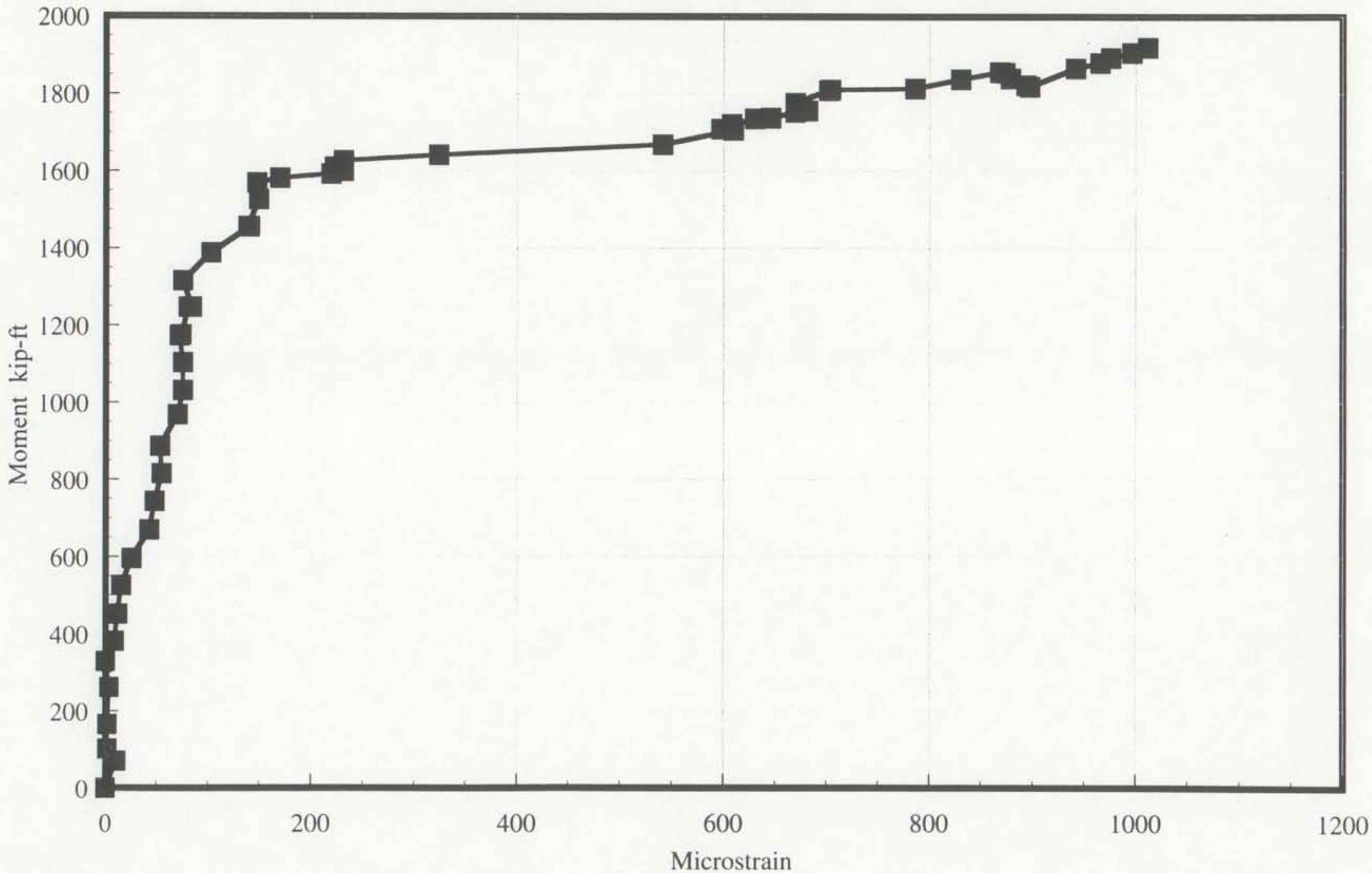


Figure 9

Moment vs Strain Pile Cap Embedded Gage (CAP_4) over East Pile

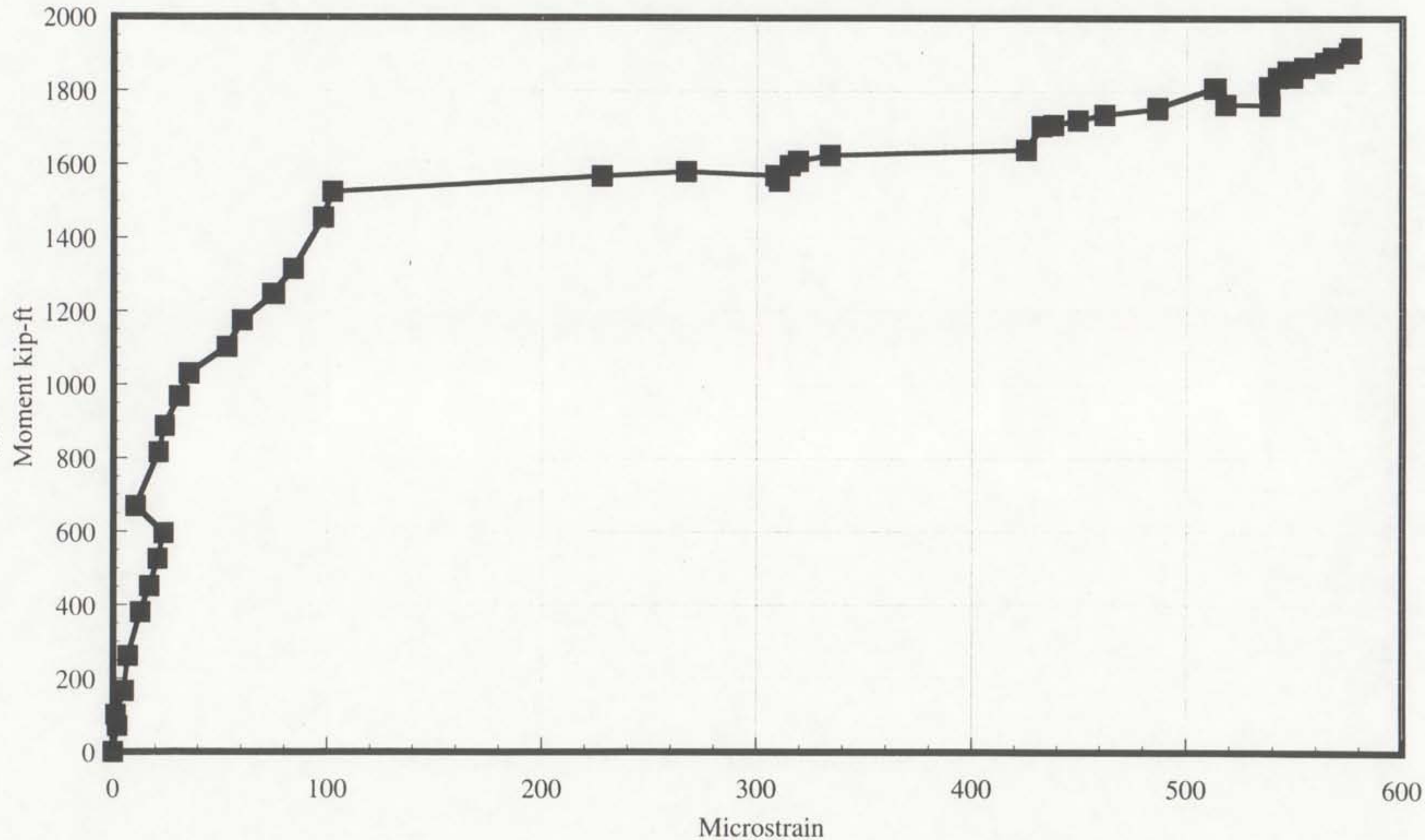


Figure 10

Strains at Distributed Moments Along East Pile Tension Face Embedded Gages

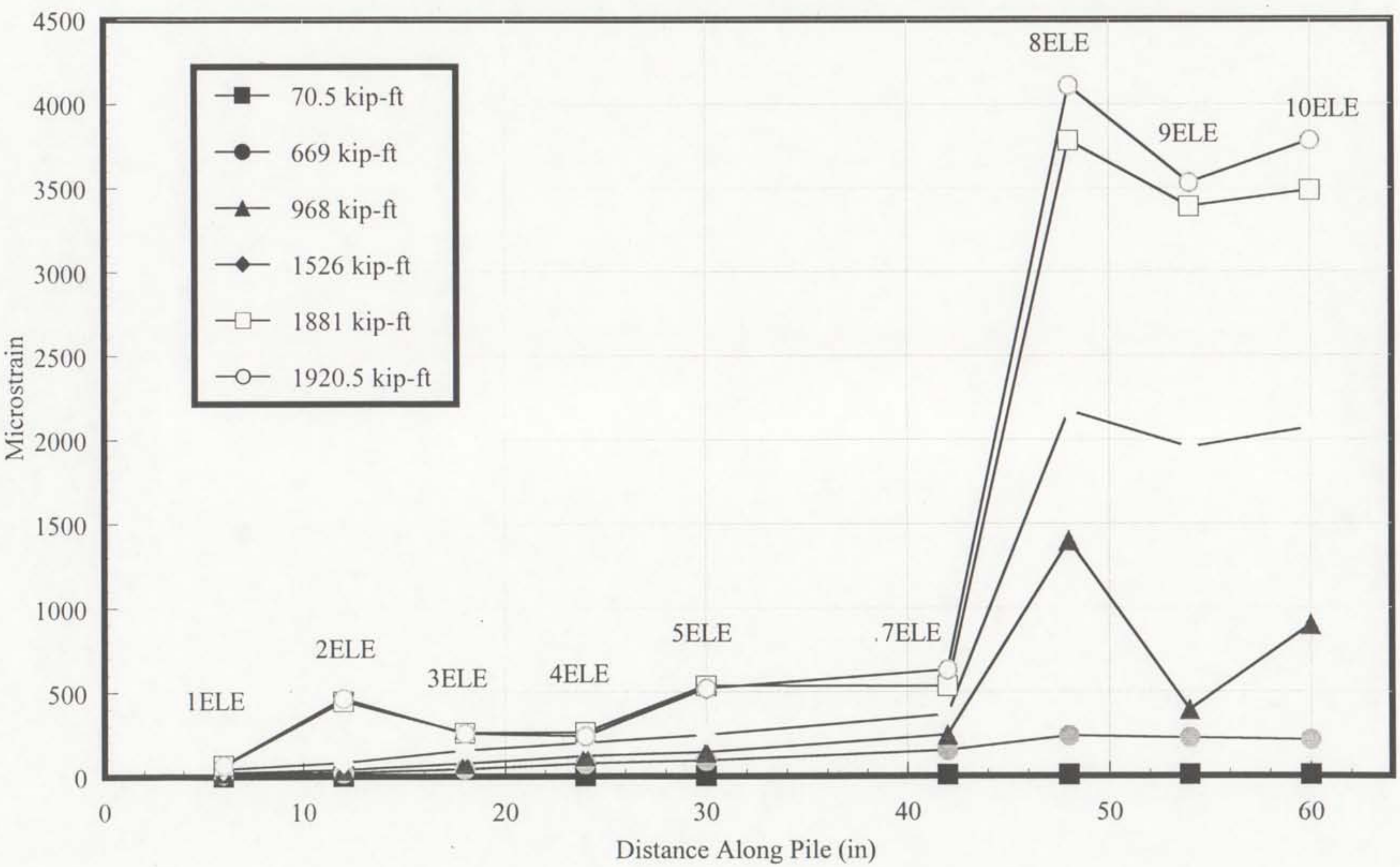
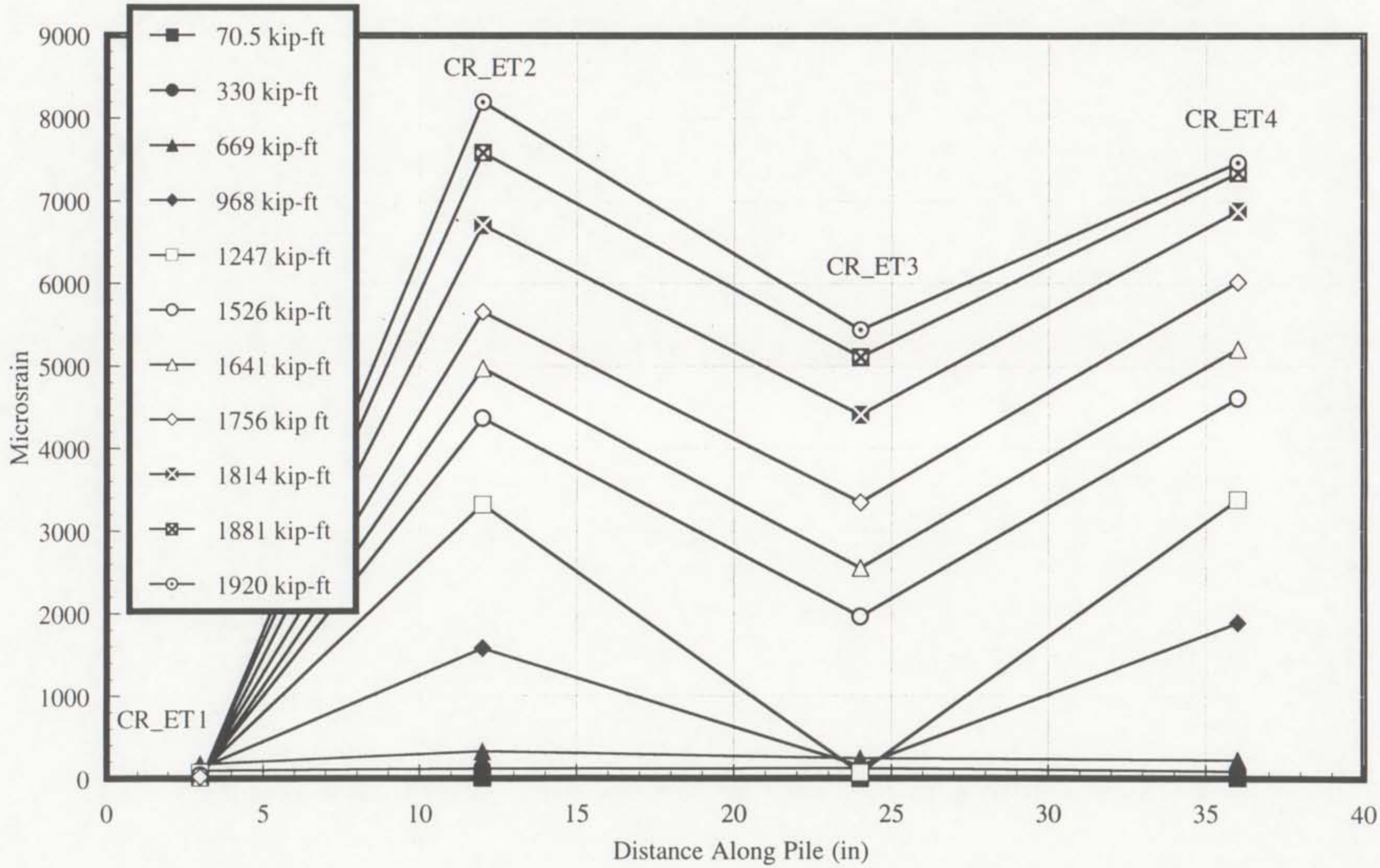


Figure 11

Strains At Distributed Moments Along East Pile Tension Face Crack Gages



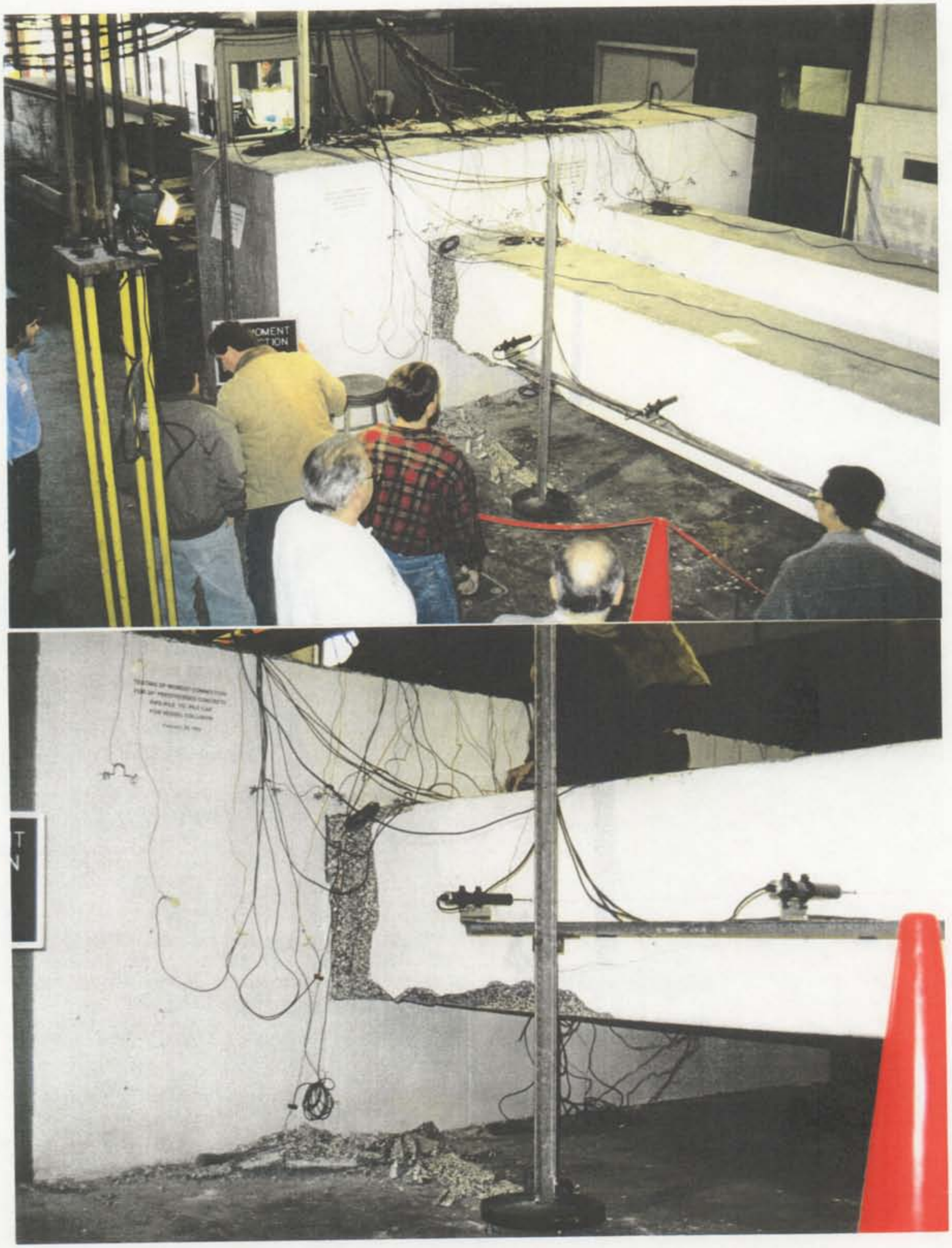


Figure 12- Final mode of failure and the formation of the plastic hinge at the critical region (pile to pile cap interface). Flexural mode of failure with concrete crushing in the compression zone. Failure test moment = 1920.50 k-ft.

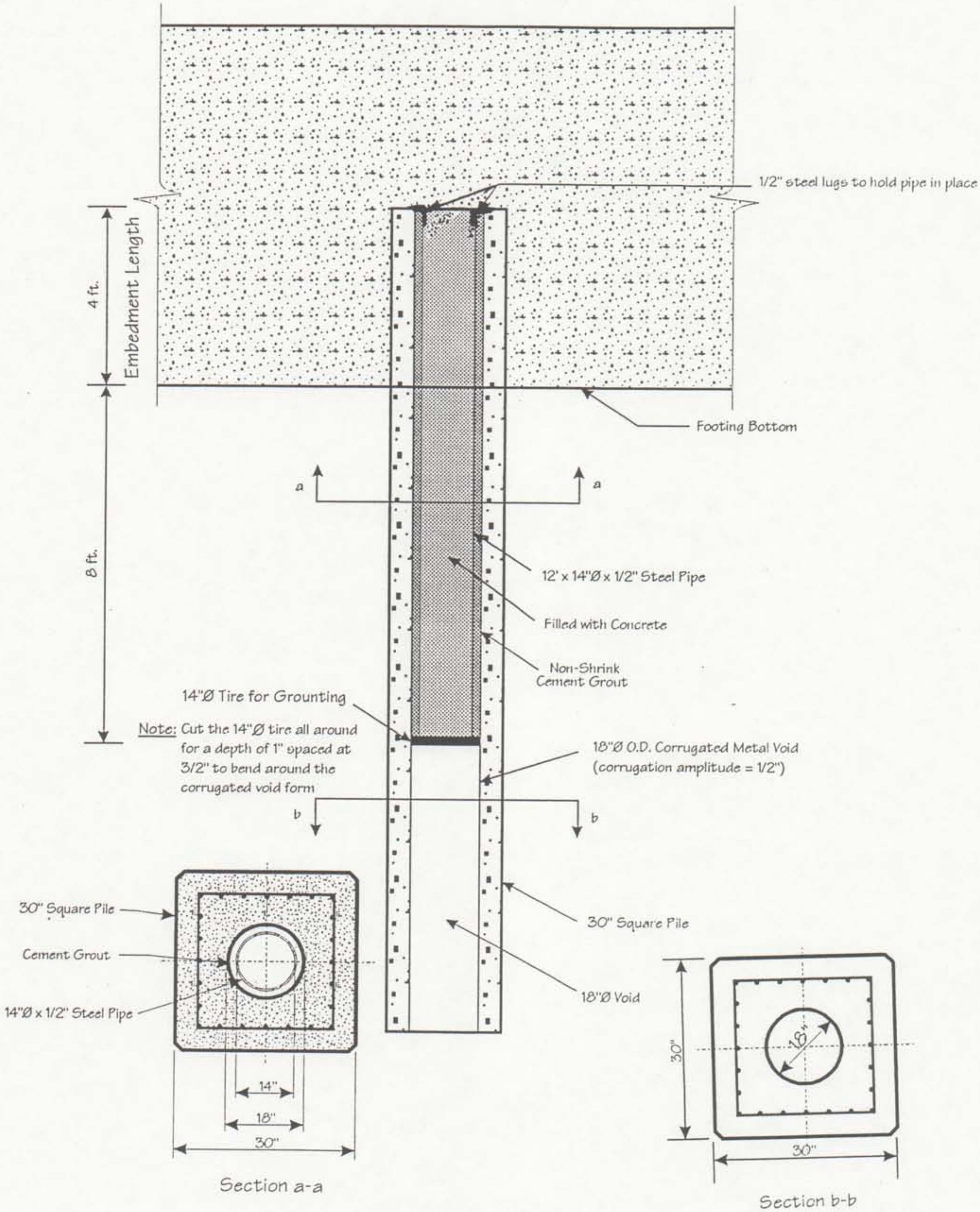
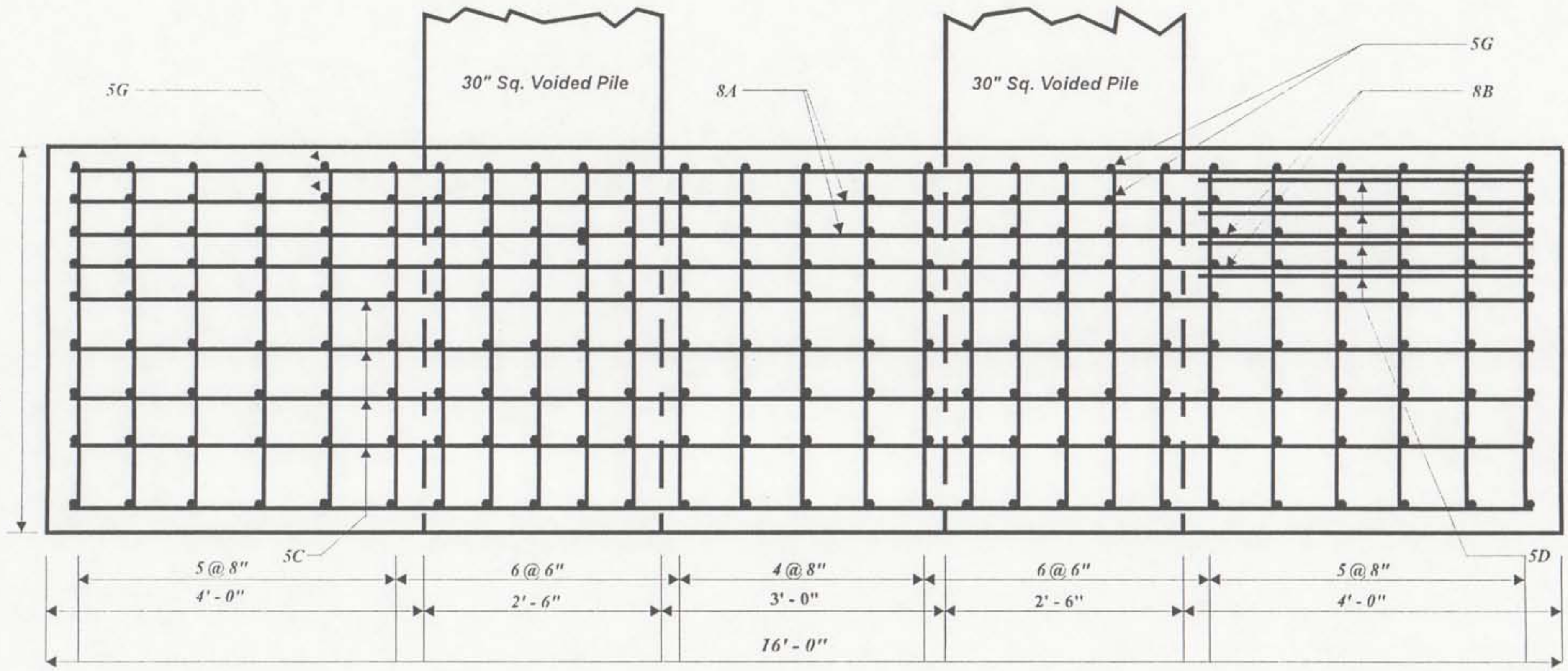


Figure 13- Recommended detail for pipe-pile to pile cap moment connection.

Appendix A

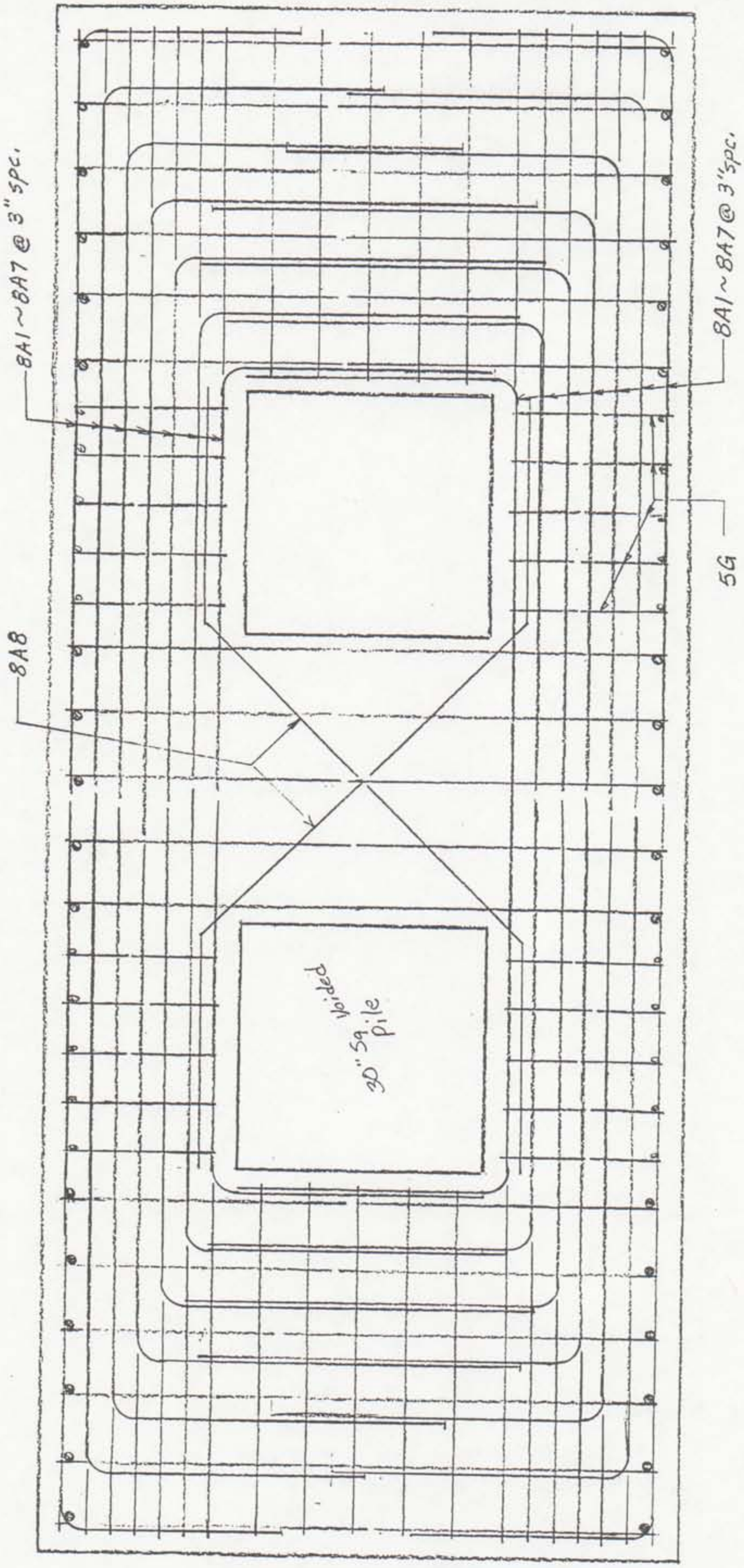
Reinforcement and Dimensions of the Pile Cap

Pile Moment Connection



Elevation

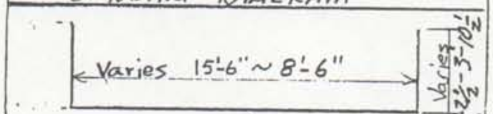
7/31/98



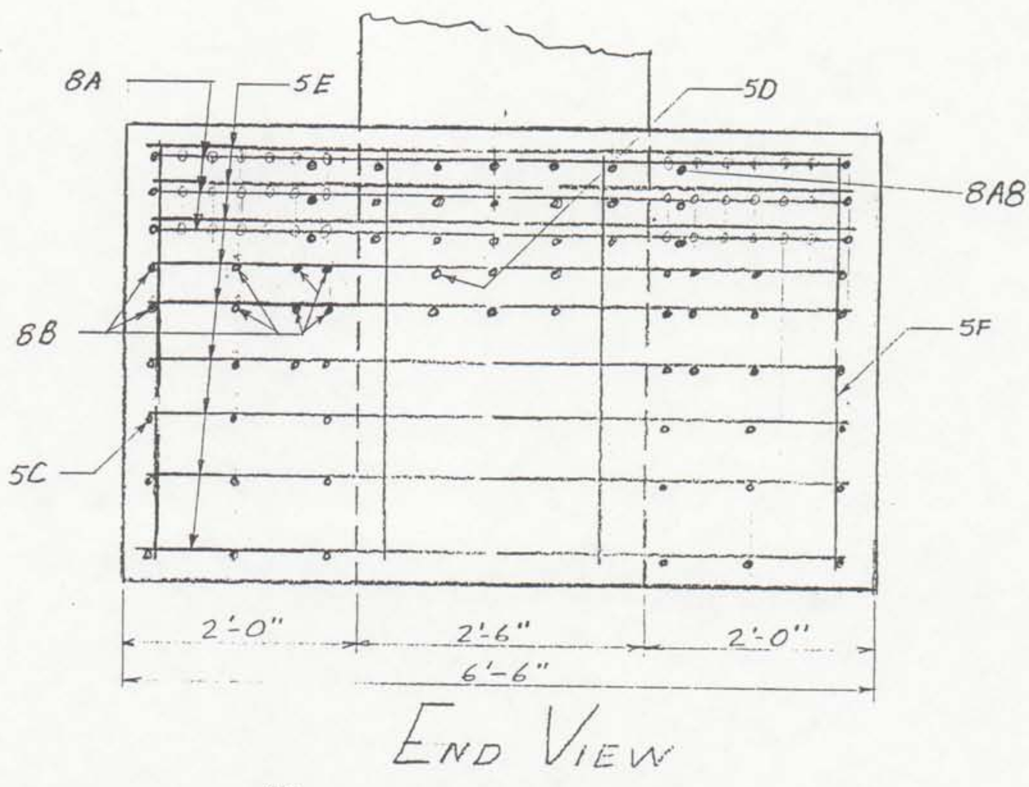
PLAN

BILL OF STEEL			
MARK-SIZE	NO. REQ'D	LENGTH	WEIGHT
8A1	6	20'-0"	321#
8A2	6	20'-0"	321#
8A3	6	20'-0"	321#
8A4	6	20'-0"	321#
8A5	6	18'-7"	298#
8A6	6	17'-5"	279#
8A7	6	16'-3"	260#
8A8	6	10'-0"	160#
8B	16	15'-8"	668#
5C	26	15'-8"	425#
5D	42	3'-8"	161#
5E	153	6'-2"	984#
5F	60	3'-8"	229#
5G	180	1'-8"	313#
			5,061#

BENDING DIAGRAM



BARS 8A1 ~ 8A7



END VIEW

Appendix B

Step-By-Step Procedure
For
Fabrication of the Pipe pile to Pile Cap

-Fabrication of 30" Square piles with 18" O.D. Corrugated Metal Void. Corrugation Amplitude = 1/2".

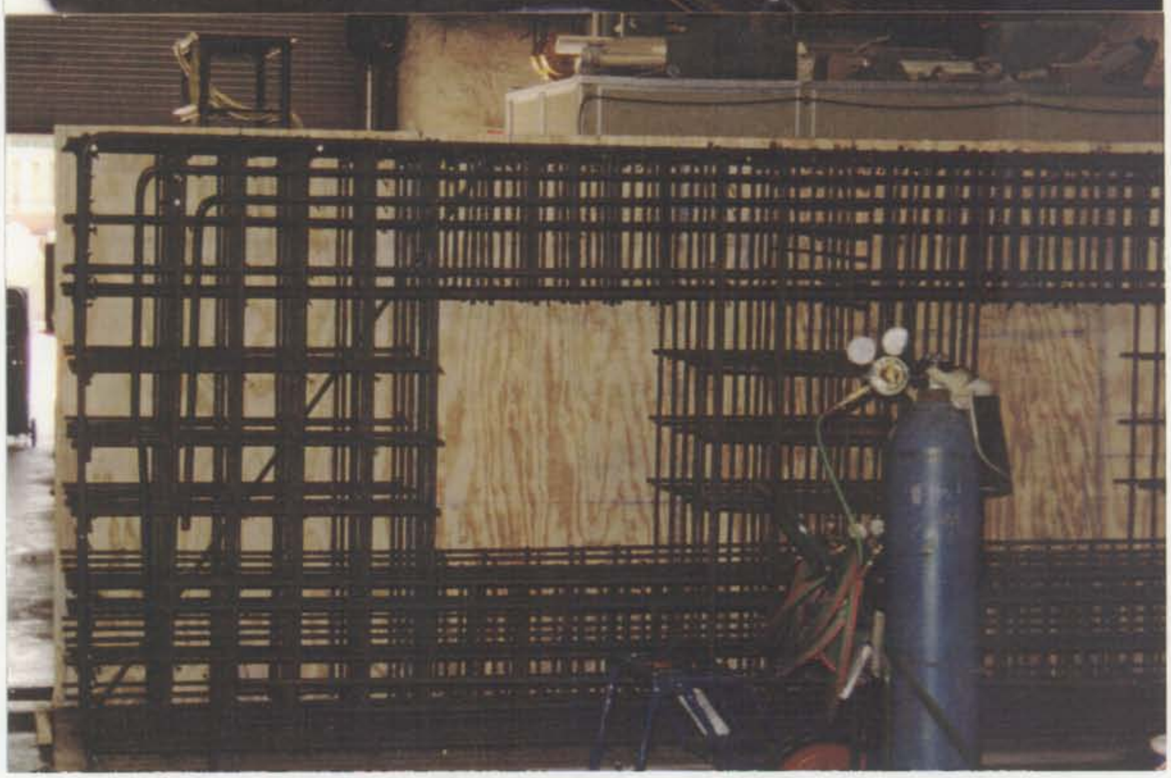
-The Piles were stressed with 24-1/2" Diam. LRS (Special) @ 32,307 Lbs Each.

-Field Instrumentation of the 30" Square piles with 18" O.D. Corrugated Metal Void.

-Use 14" Diameter Steel Pipe (thickness =1/2") with Minimum Yield Strength of 36 ksi. The Pipe was sandblasted to ensure bond between steel and concrete.

-The Steel Pipes were filled with concrete prior to their placement in the PC Piles.

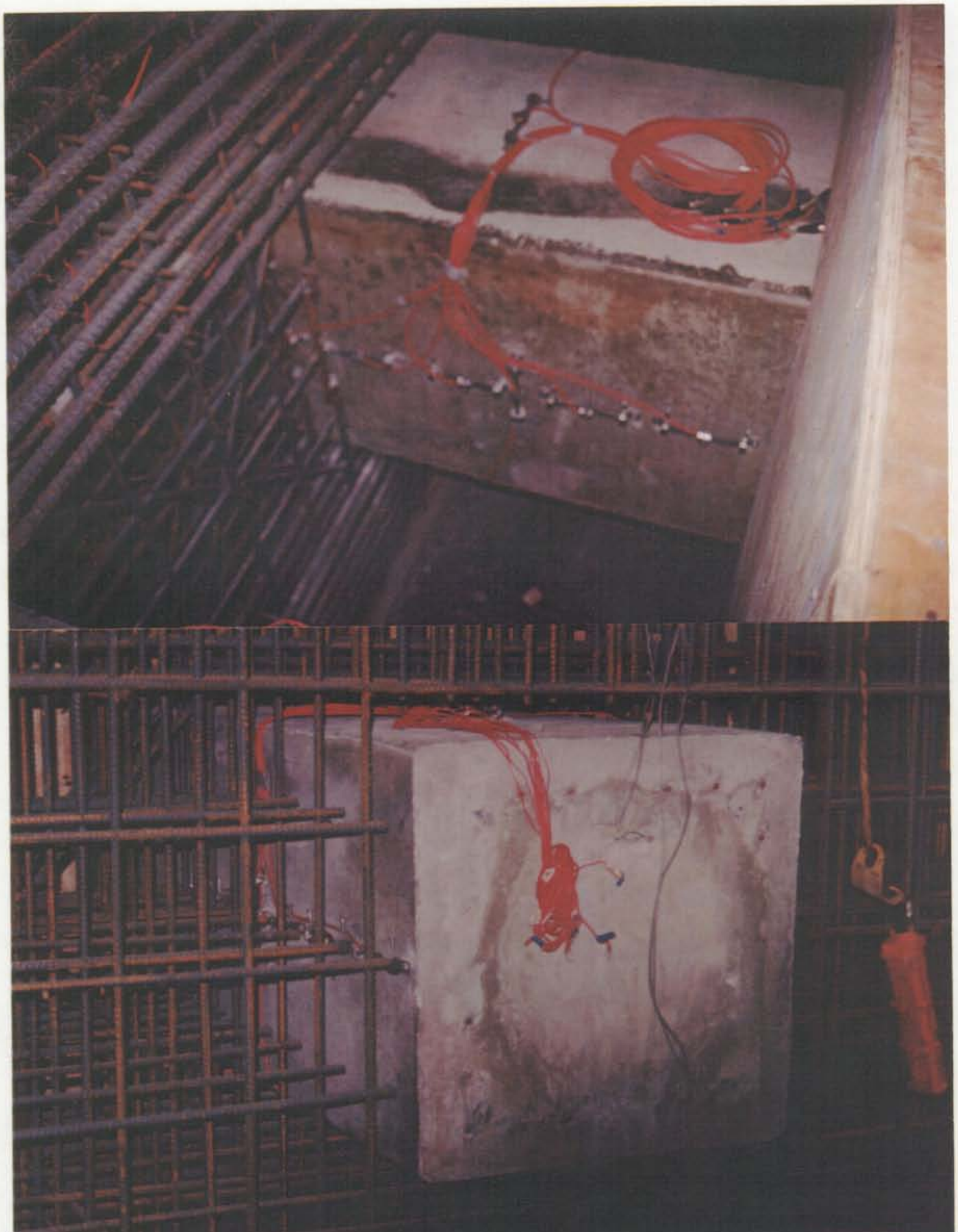
-The 14" Diameter Steel Pipe was installed into the 30" PC Pile.
-Use FDOT approved Grout between the 30" pile and the
14" Diameter Steel Pipe.
-Grouting Process Require the use of Concrete Pump.



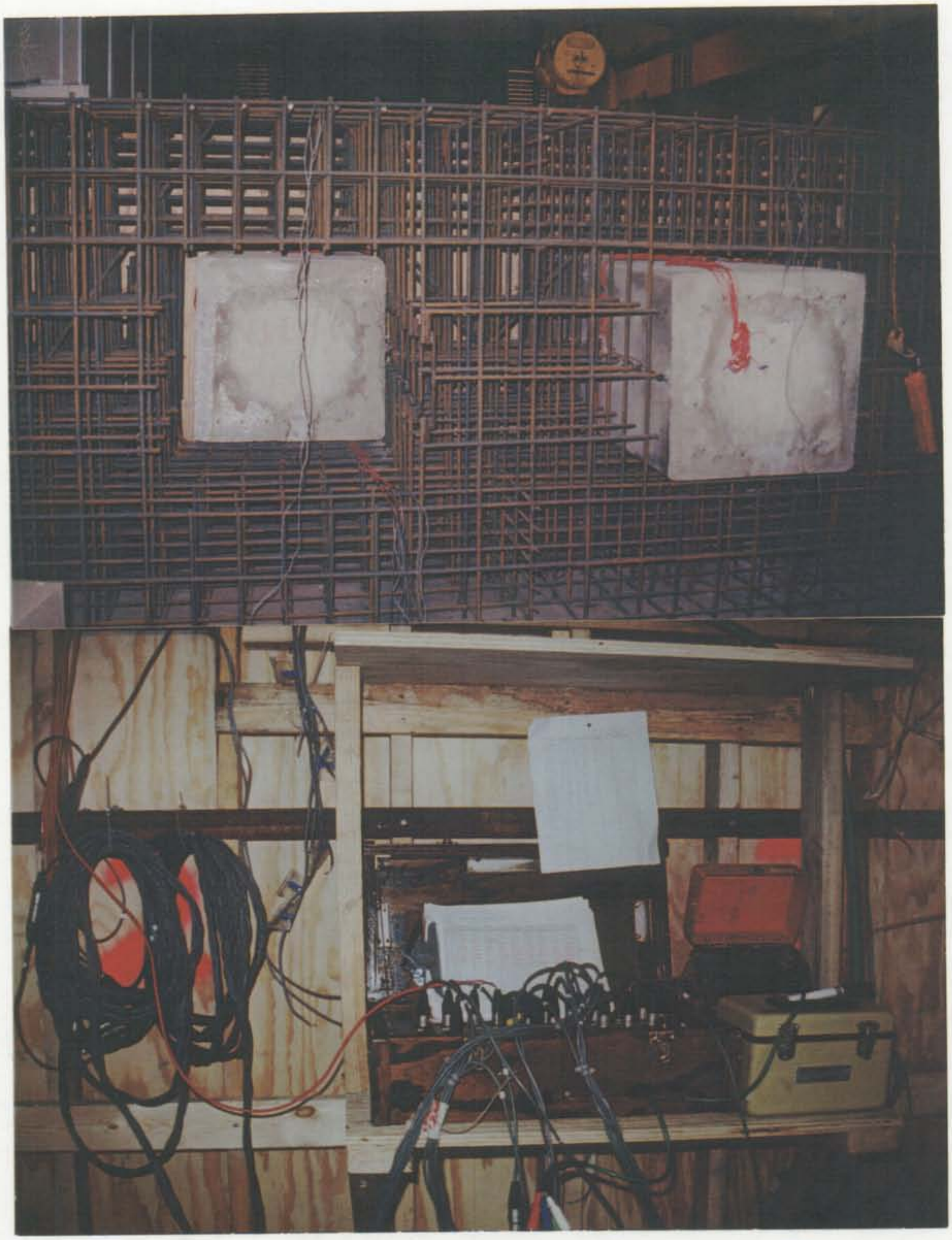
-Build the Form for the Pile Cap. The Form was 16.5 ft. Long, 6.5 ft. Wide and 4 ft. Deep.

-Fabricate and Build the steel Reinforcement gage for the Test Specimen.

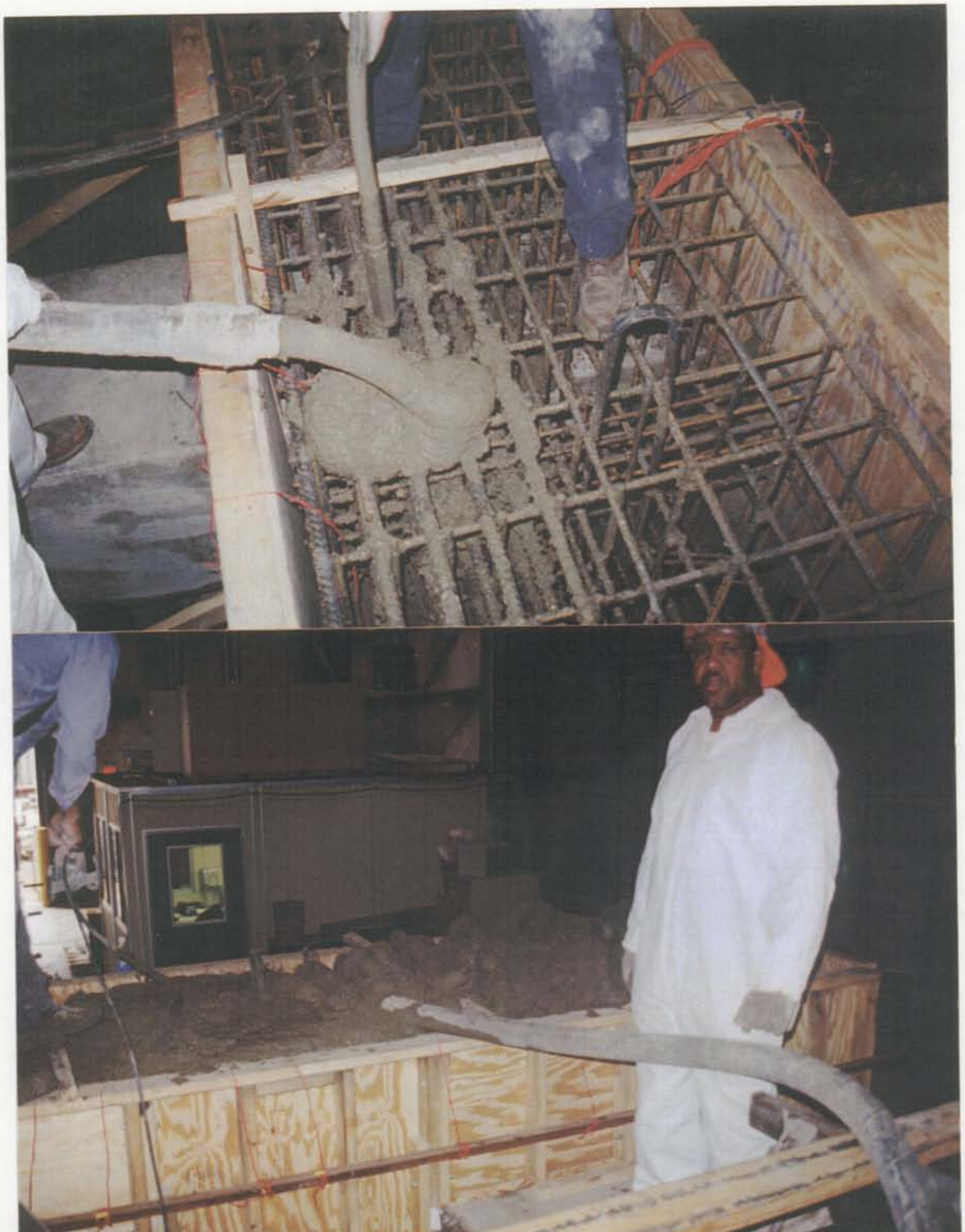
-Installation of the Modified Ends of the 30" PC Piles into the Pile Cap.



-Instrumentation of the 30" Square piles and the Pile Cap prior to Concrete Casting of the Pile Cap.



-All the Embedded Vibrating Wire Strain Gages were Connected to a Data Acquisition System Prior to Concrete Casting.

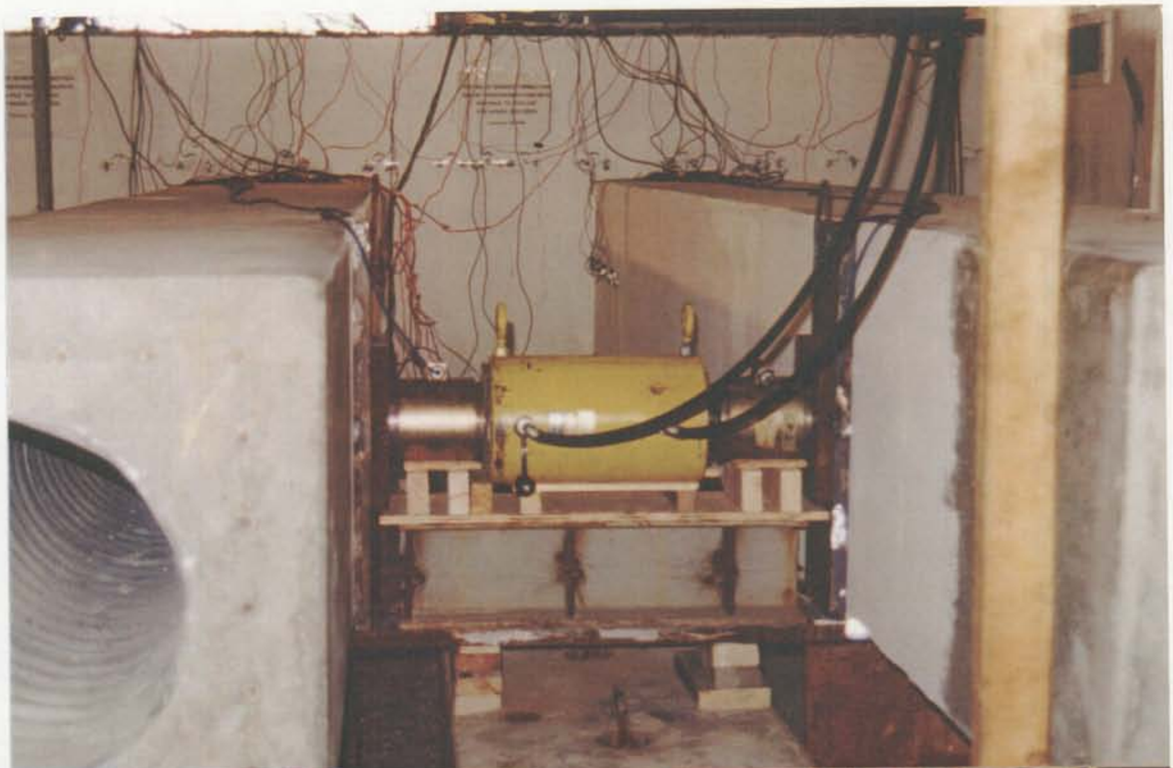


-Concrete Casting of the Pile Cap by using a Concrete Pump and an Interior Concrete Vibrator.

-Strip the Form from the Pile Cap Test Specimen.
-Instrumentation of the Test Specimen.

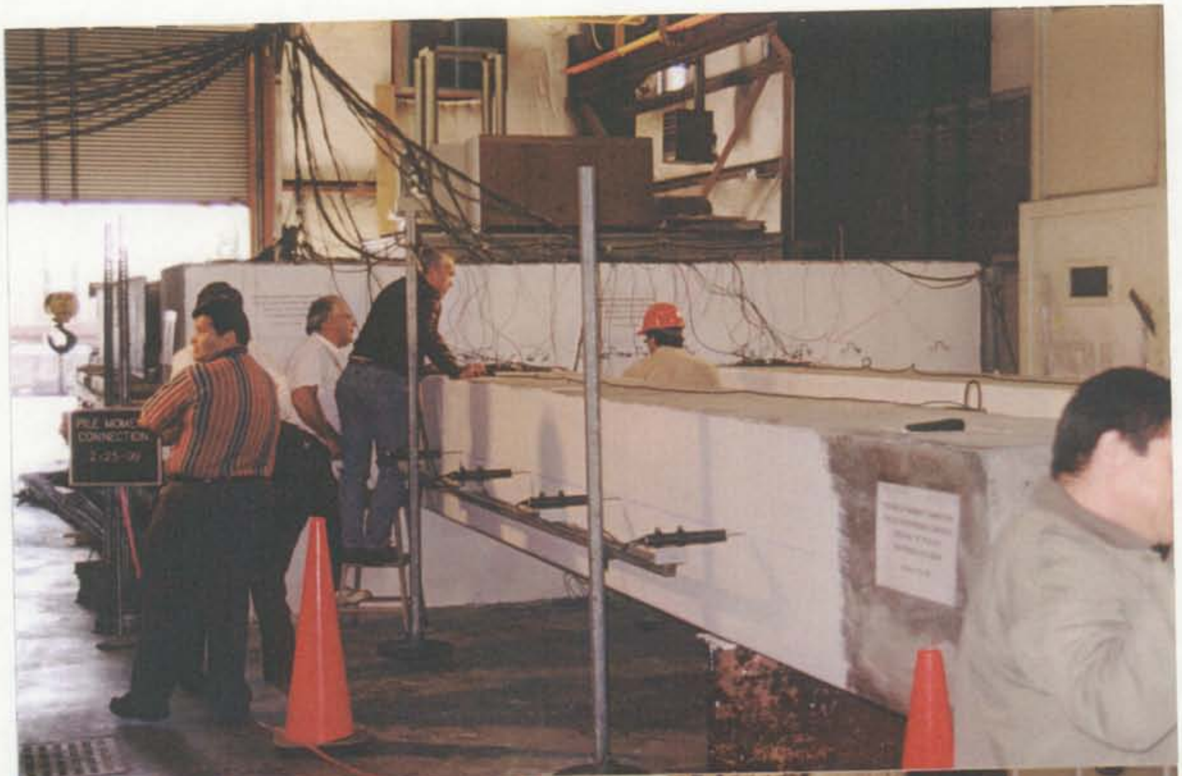
-Test Setup of the Test Specimen.
-Installation of the LVDT's to Measure deflections along the Test Specimen.

-Test Setup with the 30" PC Piles Embedded 48" in the Pile Cap and extended 21 ft. from the face of the Pile Cap.



-The Hydraulic Jack and the Load Cells (Loading Points) were Placed at 16 ft. from the Face of the Pile Cap.

-All the Gages are Connected to a Central Data Acquisition System.

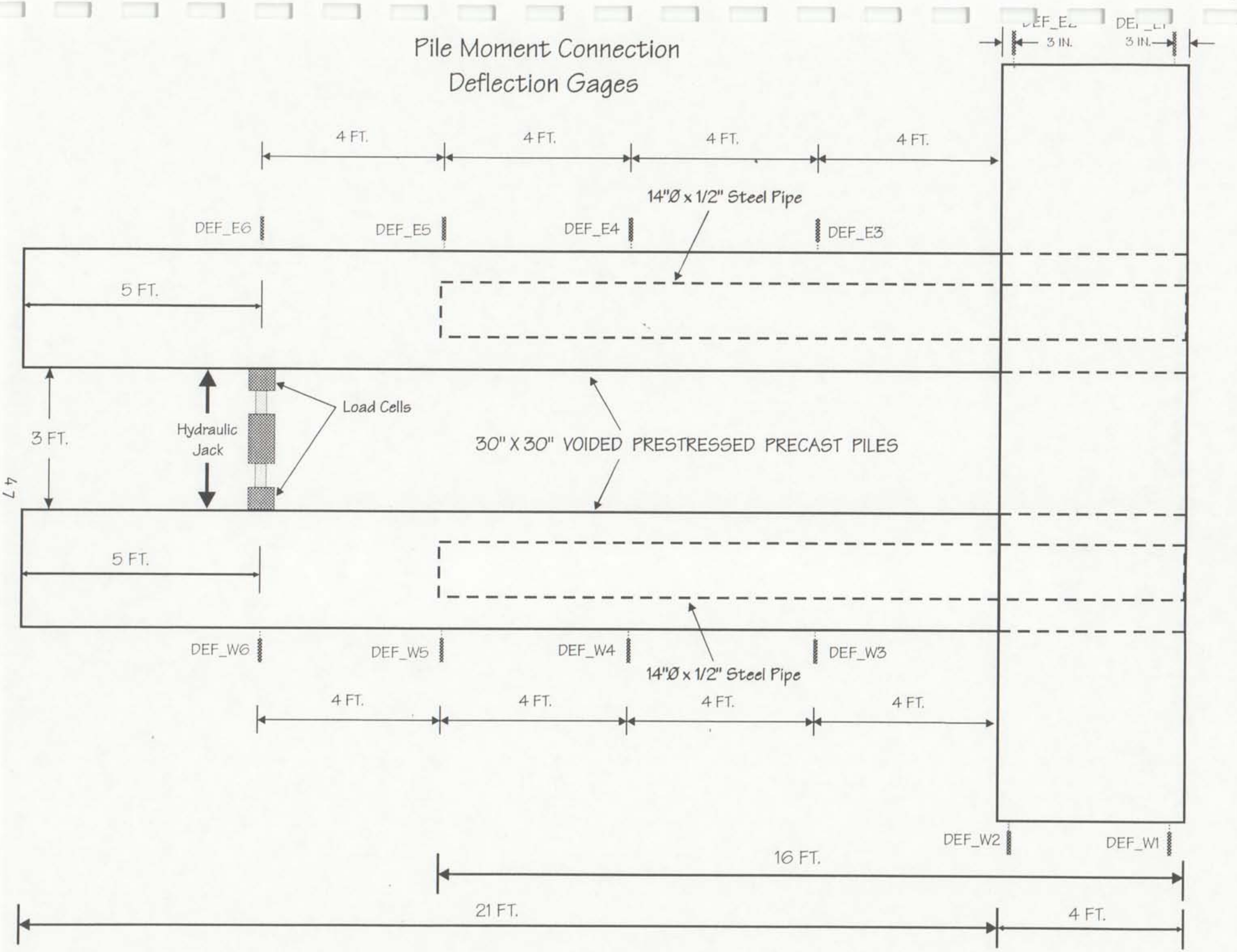


-Testing in Progress.
-Visual Inspection of the Test Specimen during the Test

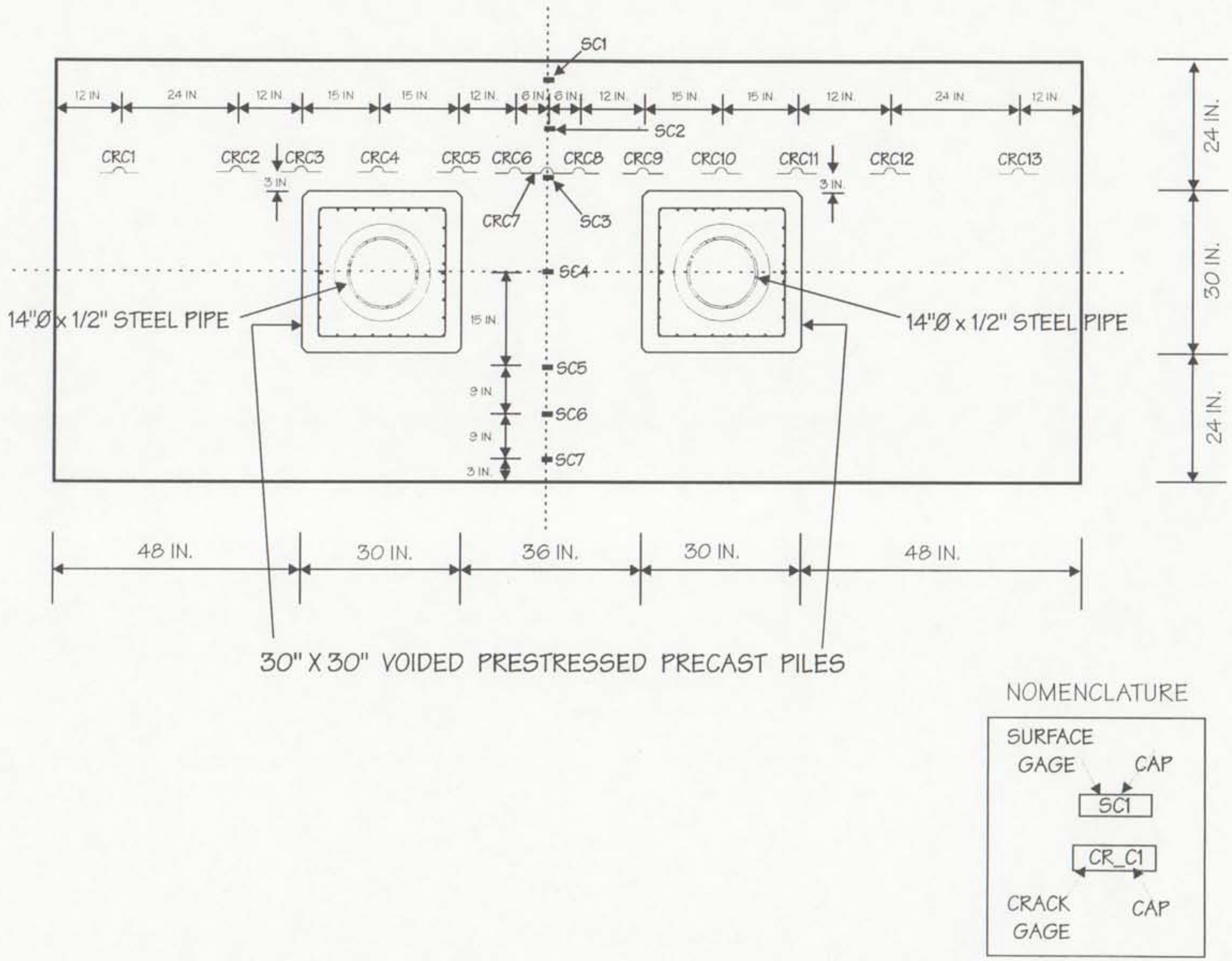
Appendix C

Test Setup and Instrumentation
Of
The Pipe pile to Pile Cap Specimen

Pile Moment Connection Deflection Gages

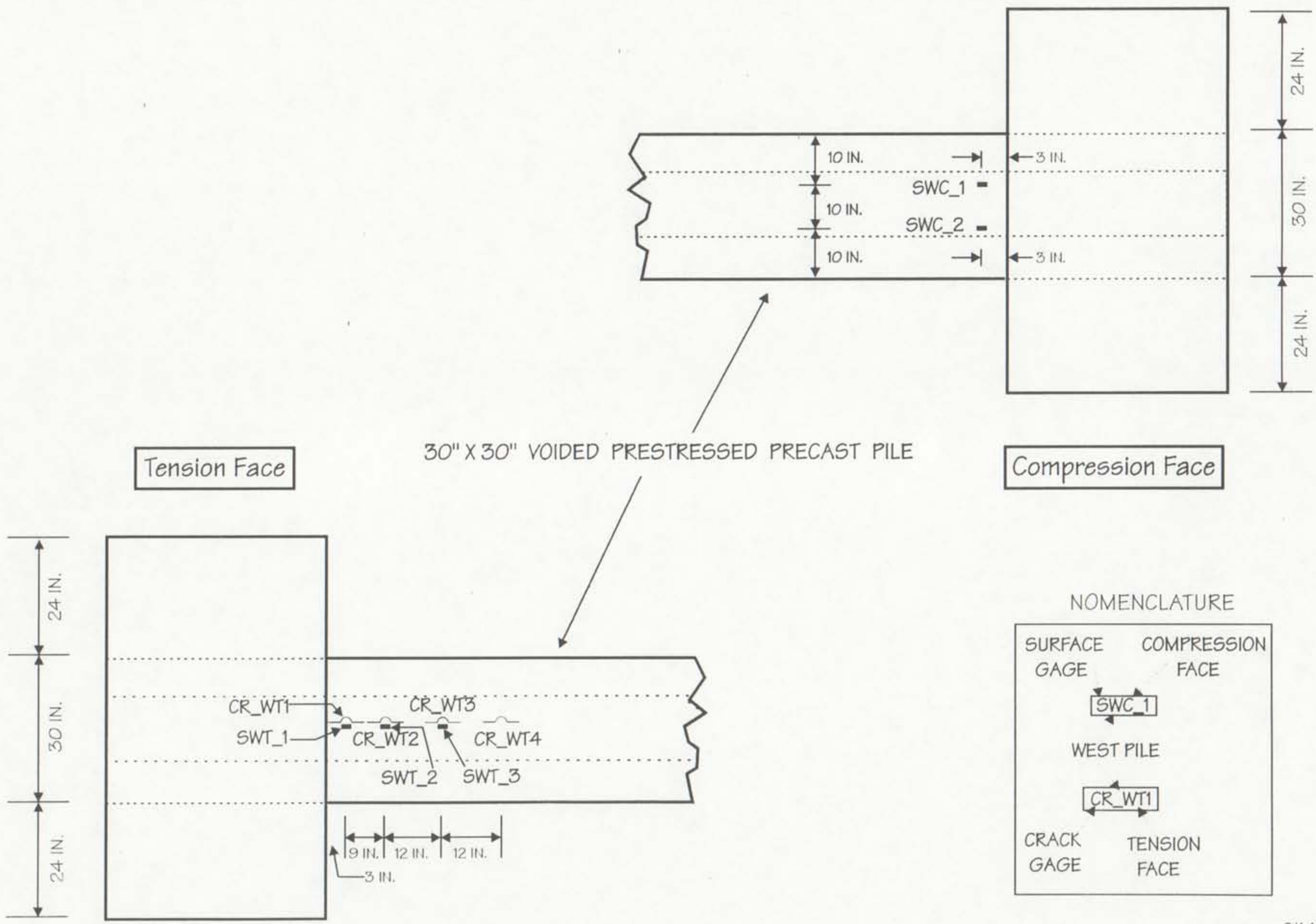


Pile Moment Connection
Surface Gages



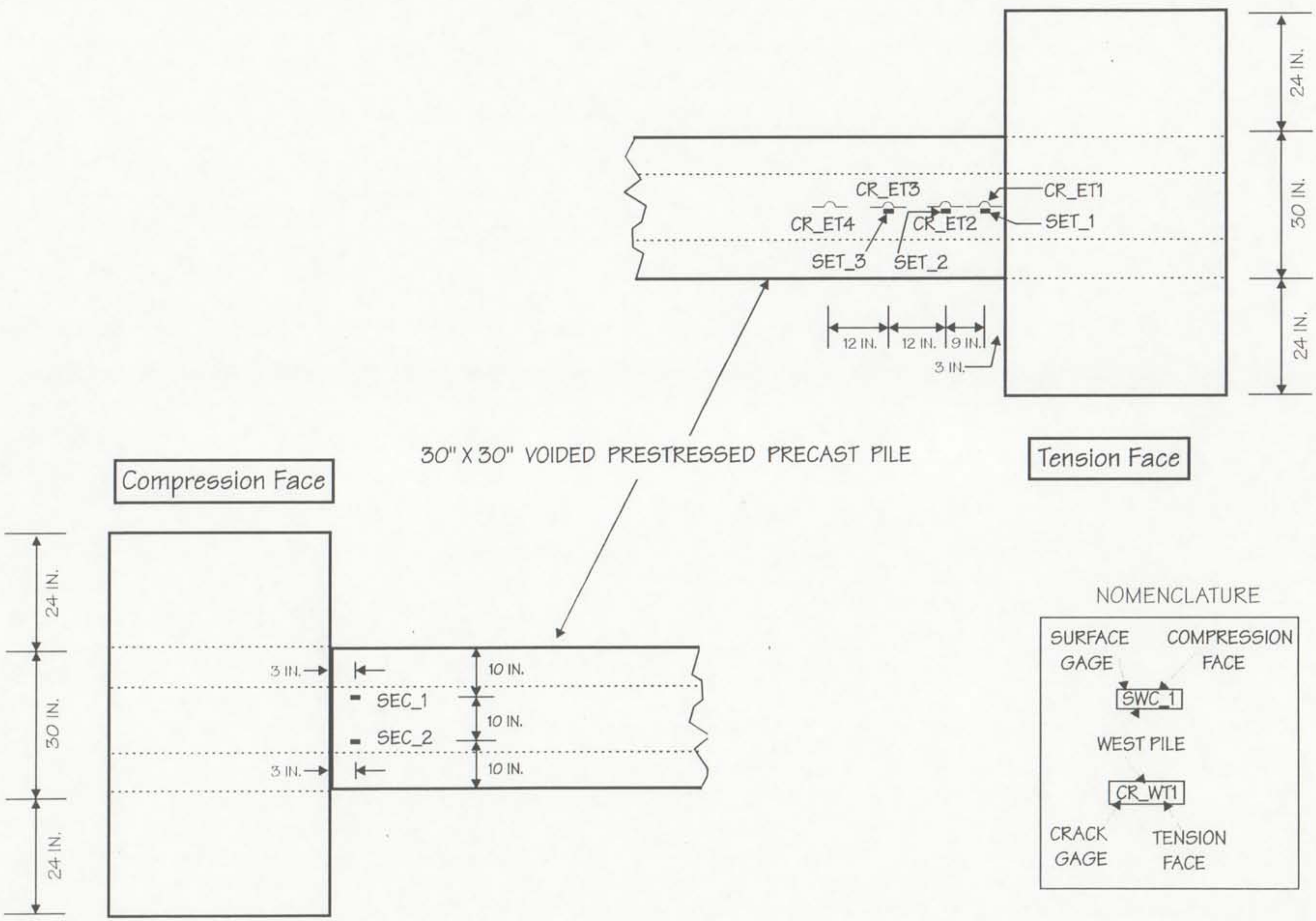
2/9/99

Pile Moment Connection
Surface Gages
West Pile



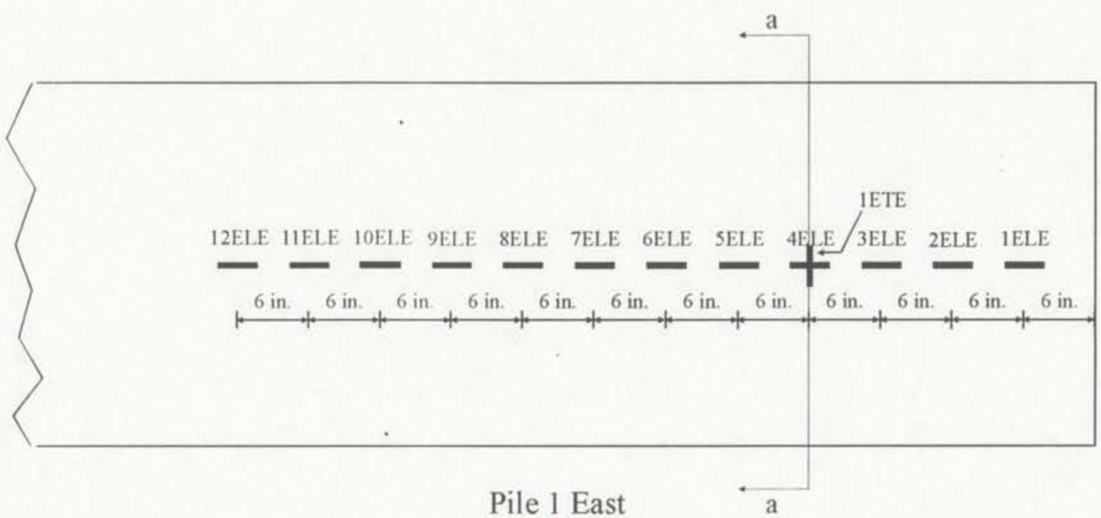
2/14/99

Pile Moment Connection
Surface Gages
East Pile

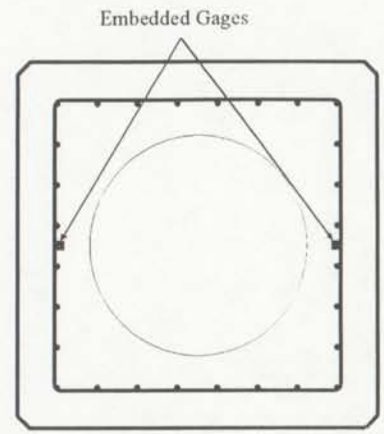


2/14/99

Pile 1 (East) Embedded Gages



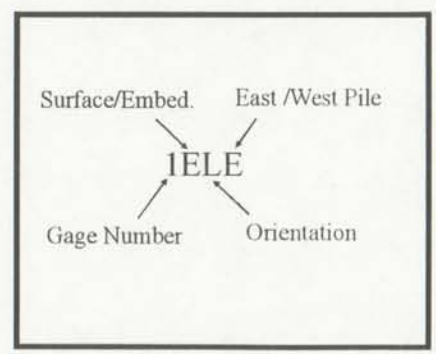
Pile 1 East
Tension Face



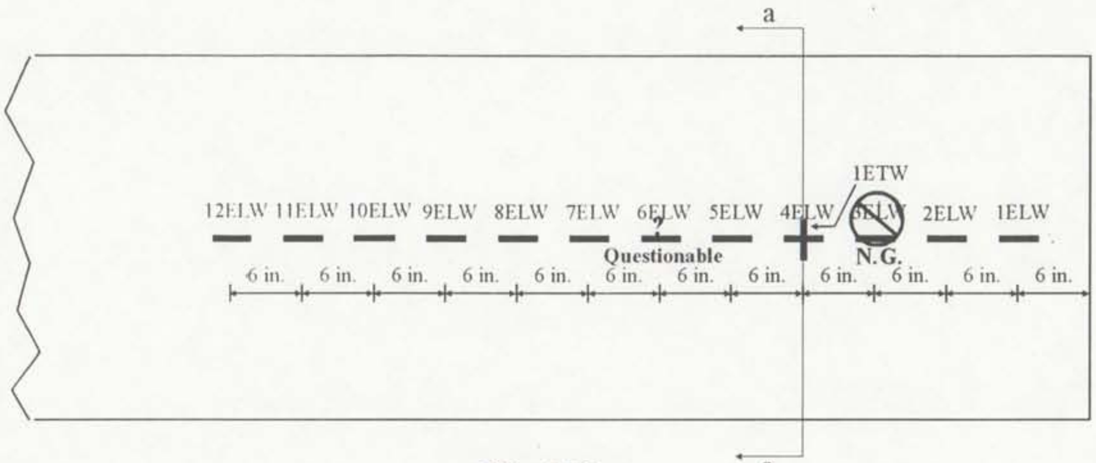
Section a-a

Pile 1 East
Compression Face

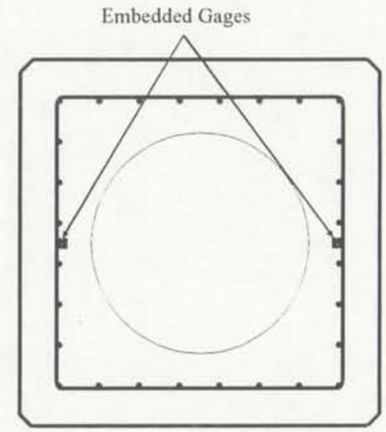
Gage Nomenclature



Pile 2 (West) Embedded Gages



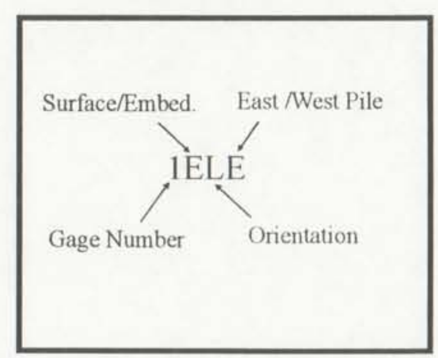
Pile 2 West
Tension Face



Section a-a

Pile 2 West
Compression Face

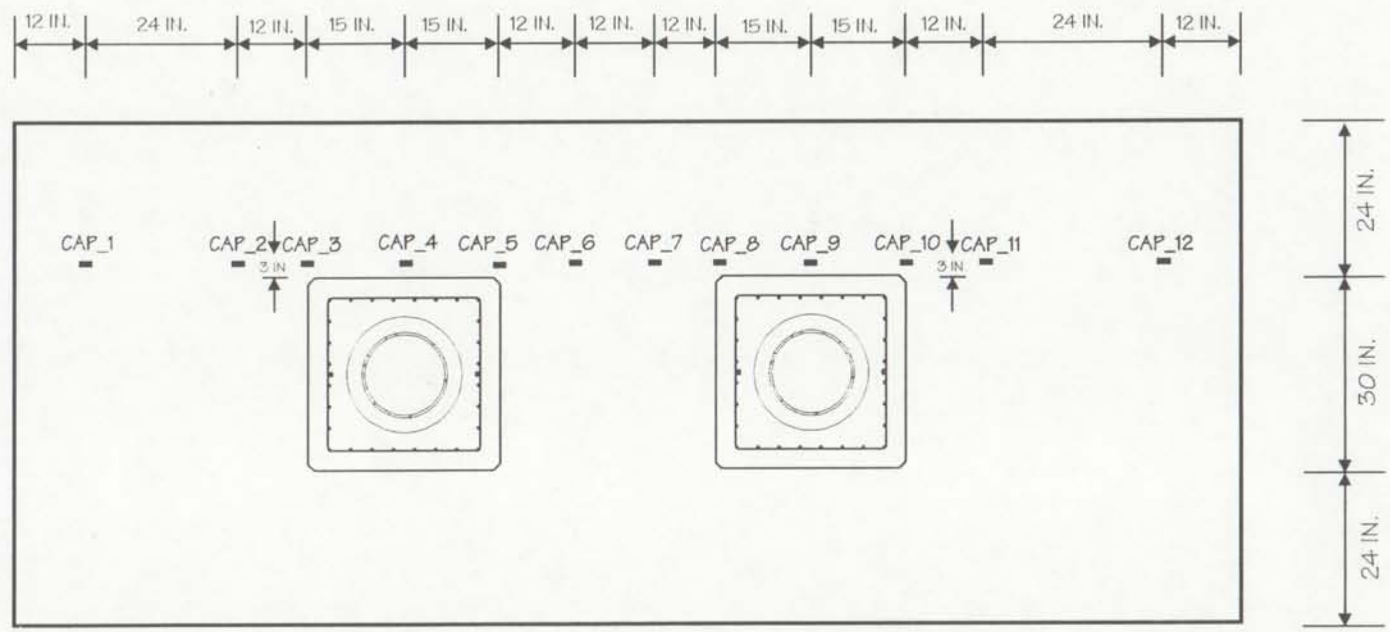
Gage Nomenclature



Pile Moment Connection

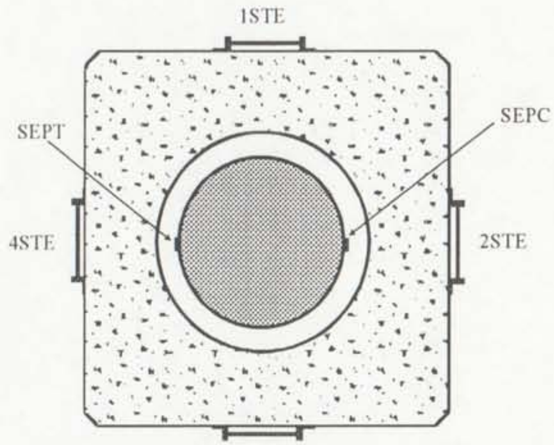
Surface Gages

Pile Cap Embedded Gages

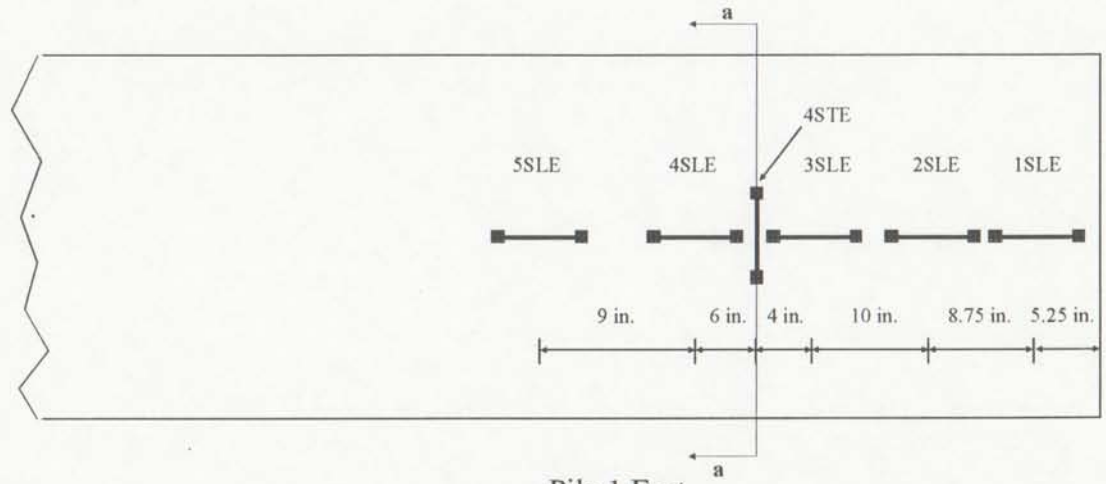


2/14/99

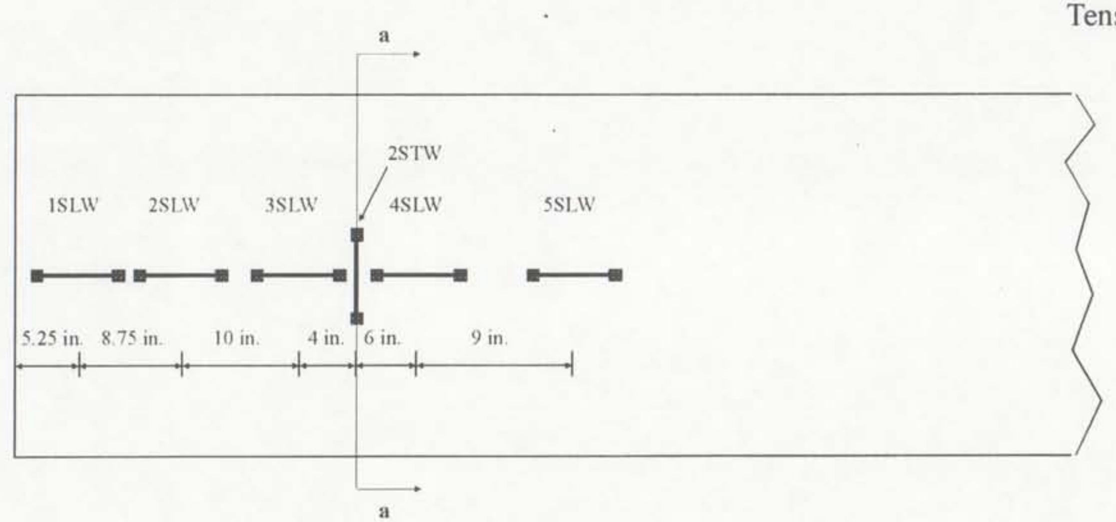
Surface Gage (vwsg) Wiring Diagram



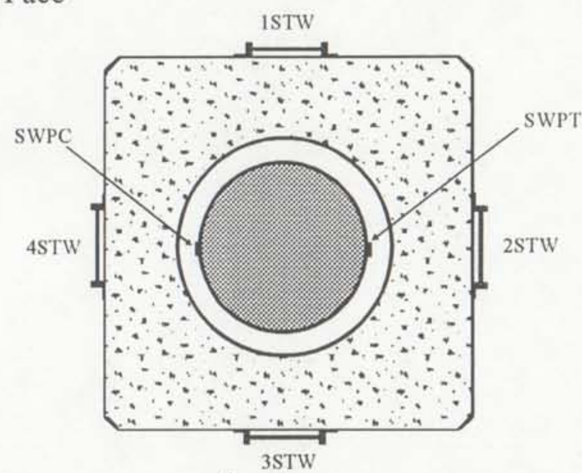
Section a-a



Pile 1 East
Tension Face

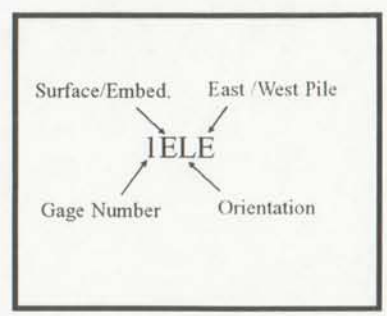


Pile 2 West
Tension Face



Section a-a

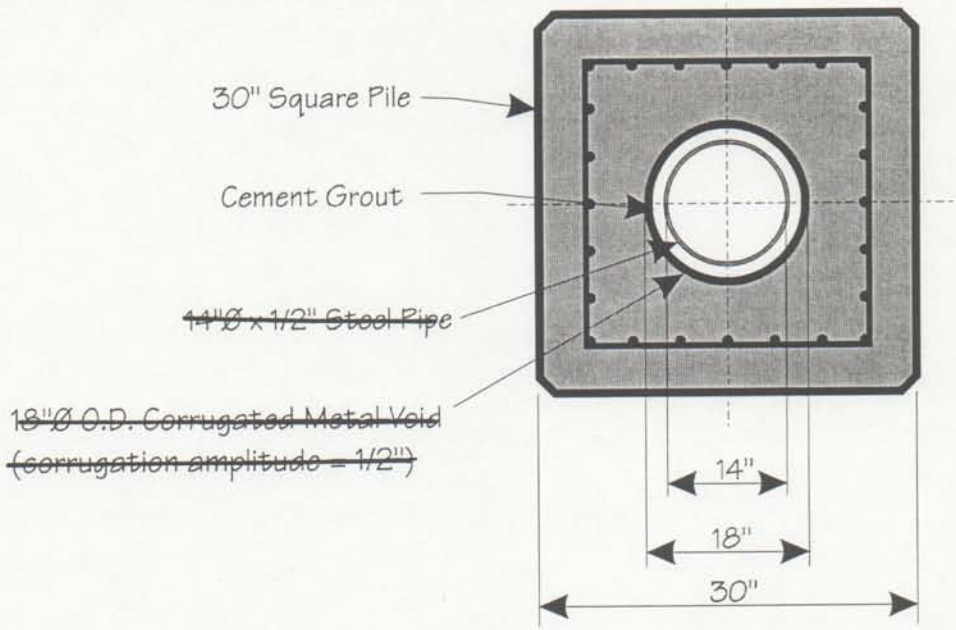
Gage Nomenclature



Appendix D

Calculations
Of
Nominal Moment Capacities

Case 1
Nominal Moment Capacity of Pile Connection
(Pile without the 18" Corrugated Metal Void)



$f_{pu} := 270 \cdot ksi$

$f_c := 10129 \cdot psi$

$f_{ps} := 251 \cdot ksi$

Neglecting Effect of Compression Steel

$A_{s1} := .167 \cdot in^2$

$A_{st} := 17 \cdot A_{s1}$

$$y_p := \frac{(7 \cdot 3 + 2 \cdot (6 + 12 + 15 + 18 + 21))}{17} \cdot in$$

$y_p = 9.706 \cdot in$

$$a := \frac{(A_{st} \cdot f_{pu})}{.85 \cdot f_c \cdot (30 \cdot in)}$$

$d := 30 \cdot in - y_p$

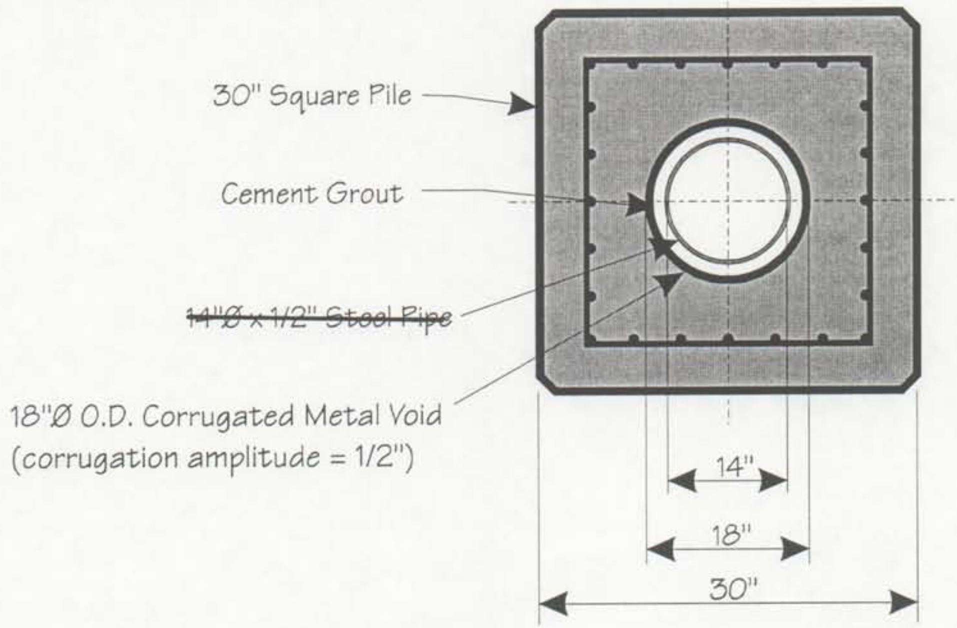
$a = 2.968 \cdot in$

$d = 20.294 \cdot in$

$$M_n := \left[A_{st} \cdot f_{ps} \cdot \left(d - \frac{a}{2}\right)\right]$$

$M_n = 1117 \cdot kip \cdot ft$

Case 2
Nominal Moment Capacity of Pile Connection
(Pile with the 18" Corrugated Metal Void)



$f_{pu} := 270 \cdot ksi$ $\quad f_{corg} := 50 \cdot ksi$ $\quad f_c := 10129 \cdot psi$

$f_{ps} := 251 \cdot ksi$

Neglecting Effect of Compression Steel

$A_{s1} := .167 \cdot in^2$

$A_{st} := 17 \cdot A_{s1}$

$A_{corg} := 2.94 \cdot in^2$

$$y_p := \frac{(7 \cdot 3 + 2 \cdot (6 + 12 + 15 + 18 + 21))}{17} \cdot in$$

$y_p = 9.706 \cdot in$

$$a := \frac{\left(A_{st} \cdot f_{pu}\right) + \left(f_{corg} \cdot A_{corg}\right)}{.85 \cdot f_c \cdot (30 \cdot in)}$$

$d := 30 \cdot in - y_p$

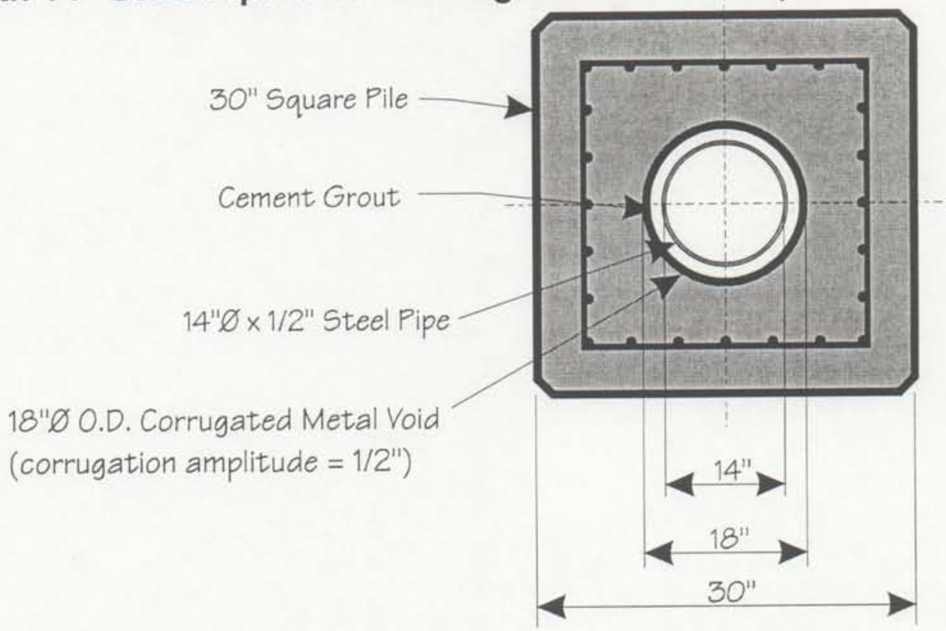
$a = 3.537 \cdot in$

$d = 20.294 \cdot in$

$$M_n := A_{st} \cdot f_{ps} \cdot \left(d - \frac{a}{2}\right) + \left(f_{corg} \cdot A_{corg}\right) \cdot \left(15 \cdot in - \frac{a}{2}\right)$$

$M_n = 1262.2 \cdot kip \cdot ft$

Case 3
Nominal Moment Capacity of Pile Connection
(Pile with 14" Steel Pipe & 18" Corrugated Metal Void)



$f_{pu} := 270 \cdot ksi$ $\quad f_{corg} := 50 \cdot ksi$ $\quad f_c := 10129 \cdot psi$

$f_{ps} := 251 \cdot ksi$ $\quad f_{pipe} := 36 \cdot ksi$

Neglecting Effect of Compression Steel

$A_{s1} := .167 \cdot in^2$

$A_{st} := 17 \cdot A_{s1}$

$A_{corg} := 2.94 \cdot in^2$

$A_{pipe} := 21.21 \cdot in^2$

$$y_p := \frac{(7 \cdot 3 + 2 \cdot (6 + 12 + 15 + 18 + 21))}{17} \cdot in$$

$y_p = 9.706 \circ in$

$$a := \frac{(A_{st} \cdot f_{pu}) + (f_{pipe} \cdot A_{pipe}) + (f_{corg} \cdot A_{corg})}{.85 \cdot f_c \cdot (30 \cdot in)}$$

$a = 6.493 \circ in$

$d := 30 \cdot in - y_p$

$d = 20.294 \circ in$

$$M_n := A_{st} \cdot f_{ps} \cdot \left(d - \frac{a}{2}\right) + (f_{corg} \cdot A_{corg}) \cdot \left(15 \cdot in - \frac{a}{2}\right) + (f_{pipe} \cdot A_{pipe}) \cdot \left(15 \cdot in - \frac{a}{2}\right)$$

$M_n = 1904.2 \circ kip \cdot ft$

Response Version 1 Input Data-File

```
Copyright 1990 A. Felber
Name of Section:                          30 in pile
Units M/U 'Metric/U.S.Customary':              U
Number of Concrete Types (1-5):              2
   Type        f'c        ec'            fcr     Tension Stiffening
   Number     [psi]  [Milli-Strain]     [psi]    Factor
       1      10129     -3.000        755      0.70
       2      10129     -3.000        755      0.70
Number of Rebar Types  (1-5):                3
   Type  Elastic Modulus    fy         esh        esrupt        fu
   Number      [ksi]          [ksi]      [---Milli-Strain--]  [ksi]
       1  29000.00       36.00    10.000    40.000      60.00
       2  29000.00       50.00    10.000    40.000      60.00
       3  29000.00       60.00    10.000    40.000      60.00
Number of Tendon Types  (1-5):               1
   Type      [Ramberg-Osgood-Factors--]      Elastic Modulus   fpu    eprupt
   Number         A          B          C          [ksi]         [ksi] [Milli-Strain]
       1      0.025   118.000    10.000  28000.00     270.00     40.000
Height of Section:                          30.00      in
Distance to Moment Axis:                    18.00      in
Shear Y/N 'Yes/No':                               Y
   Web width  (bw)   :                      30.00      in
   Shear depth (jd) :                       24.00      in
   Distance to web strain ex :              15.00      in
   Distance to center of web :              15.00      in
   Longitudinal crack spacing:              15.00      in
   Maximum Aggregate size     :              0.75      in
Stirrups Y/N 'Yes/No':                            Y
   Transverse crack spacing   :              6.00      in
   Area of Stirrups (Av)      :              0.11      in^2
   Stirrup Spacing  (s)       :              4.00      in
   Stirrup (Rebar) Type       :              3
Number of Concrete Layers (1-20):            8
   Layer        y       bottom width   top width    height      Type
   Number     [in]          [in]          [in]        [in]        Number
       1      0.00      30.00       30.00       3.00         2
       2      3.00      30.00       30.00       4.00         2
       3      7.00      30.00       30.00       4.00         1
       4     11.00      30.00       30.00       4.00         1
       5     15.00      30.00       30.00       4.00         1
       6     19.00      30.00       30.00       4.00         1
       7     23.00      30.00       30.00       4.00         2
       8     27.00      30.00       30.00       3.00         2
```

Input Data-File

Number of Rebar Layers (0-10) : 10

Layer Number	y [in]	Area [in^2]	Type Number
1	6.00	0.60	2
2	8.00	4.24	1
3	11.00	0.60	2
4	12.00	4.24	1
5	15.00	0.60	2
6	15.00	4.24	1
7	18.00	4.24	1
8	19.00	0.60	2
9	22.00	4.24	1
10	24.00	0.60	2

Number of Tendon Layers (0-10) : 7

Layer Number	y [in]	Area [in^2]	Prestrain [Milli-Strain]	Type Number
1	3.00	1.17	6.909	1
2	6.00	0.33	6.909	1
3	12.00	0.33	6.909	1
4	15.00	0.33	6.909	1
5	18.00	0.33	6.909	1
6	24.00	0.33	6.909	1
7	27.00	1.17	6.909	1

Consider displaced Concrete Y/N: N
Thermal & Shrinkage Strains Y/N : N
Initial Strains Y/N : N

RESPONSE OUTPUT FILE

ANALYSIS: Axial-Load + dN/dM
SECTION NAME : 30 in pile
CONCRETE MODEL: Parabolic Material Factors: C:0.600 S:0.850 P:0.900
TENSION-STIFF.: No FACTORED: No ACCURACY: well_done
Axial-Load: 0.00kip Moment: 0.00kft Shear: 0.00kip
dN/dM: 0.00 dN/dV: 0.00 dM/dV: 0.00

Axial-Load kips	**Moment** ft*kip	**Curvature** rad/10^6in	**@-Axis** [-------Milli-Strain------]	**Bottom**	**Top**	**Iter.**
0.30	0.08	0.00	-0.113	-0.113	-0.113	1
1.51	664.73	26.67	-0.140	0.340	-0.460	2
0.08	877.46	53.33	-0.063	0.897	-0.703	3
0.78	1079.95	80.00	0.027	1.467	-0.933	4
0.42	1272.12	106.67	0.120	2.040	-1.160	5
-0.42	1408.07	133.33	0.229	2.629	-1.371	6
-1.87	1513.09	160.00	0.347	3.227	-1.573	7
-1.43	1585.31	186.67	0.481	3.841	-1.759	8
-0.13	1645.65	213.33	0.619	4.459	-1.941	9
-0.99	1684.75	240.00	0.773	5.093	-2.107	10
-0.78	1719.86	266.67	0.927	5.727	-2.273	11
-0.55	1749.20	293.33	1.083	6.363	-2.437	12
-0.36	1776.18	320.00	1.239	6.999	-2.601	13
-0.38	1791.11	346.67	1.411	7.651	-2.749	14
-0.08	1804.02	373.33	1.583	8.303	-2.897	15
0.1	1815.47	400.00	1.754	8.954	-3.046	16
0.22	1825.54	426.67	1.924	9.604	-3.196	17
0.34	1834.28	453.33	2.092	10.252	-3.348	18
-0.50	1841.80	480.00	2.257	10.897	-3.503	19
0.54	1846.41	506.67	2.423	11.543	-3.657	20
0.55	1850.25	533.33	2.585	12.185	-3.815	21
0.56	1852.70	560.00	2.743	12.823	-3.977	22
0.54	1853.64	586.67	2.895	13.455	-4.145	23
0.50	1852.97	613.33	3.040	14.080	-4.320	24
0.41	1850.30	640.00	3.177	14.697	-4.503	25
0.29	1845.16	666.67	3.304	15.304	-4.696	26
0.11	1838.25	693.33	3.414	15.894	-4.906	27
-0.08	1826.93	720.00	3.505	16.465	-5.135	28
-0.25	1808.88	746.67	3.568	17.008	-5.392	29
-0.16	1779.29	773.33	3.584	17.504	-5.696	30
0.73	1724.63	800.00	3.499	17.899	-6.101	31
0.04	1663.02	826.67	3.362	18.242	-6.558	32
1.35	1609.39	853.33	3.238	18.598	-7.002	33
-0.11	1561.01	880.00	3.100	18.940	-7.460	34
-0.02	1520.64	906.67	2.987	19.307	-7.893	35
-0.03	1486.40	933.33	2.886	19.686	-8.314	36
0.01	1457.39	960.00	2.799	20.079	-8.721	37
0.01	1432.86	986.67	2.723	20.483	-9.117	38
-0.75	1385.97	1013.33	2.382	20.622	-9.778	39
-0.25	1345.35	1040.00	2.059	20.779	-10.421	40
-0.15	1310.97	1066.67	1.756	20.956	-11.044	41
-0.18	1282.15	1093.33	1.475	21.155	-11.645	42
1.26	1259.51	1120.00	1.240	21.400	-12.200	43
0.02	1247.04	1146.67	1.146	21.786	-12.614	44
-0.62	1236.70	1173.33	1.062	22.182	-13.018	45

Axial-Load	Moment	Curvature	@-Axis	Bottom	Top	Iter.
kips	ft*kip	rad/10^6in	[-------Milli-Strain------]			
0.10	1228.42	1200.00	0.990	22.590	-13.410	46
-0.02	1221.45	1226.67	0.919	22.999	-13.801	47
-0.00	1215.85	1253.33	0.854	23.414	-14.186	48
-0.03	1211.45	1280.00	0.793	23.833	-14.567	49
0.01	1208.12	1306.67	0.737	24.257	-14.943	50
-0.01	1205.73	1333.33	0.684	24.684	-15.316	51
-0.01	1204.19	1360.00	0.635	25.115	-15.685	52
-0.01	1203.40	1386.67	0.590	25.550	-16.050	53
-0.01	1203.28	1413.33	0.547	25.987	-16.413	54
-0.01	1203.73	1440.00	0.508	26.428	-16.772	55
-0.01	1204.68	1466.67	0.471	26.871	-17.129	56
-0.00	1206.06	1493.33	0.437	27.317	-17.483	57
0.00	1207.77	1520.00	0.404	27.764	-17.836	58
0.01	1209.72	1546.67	0.373	28.213	-18.187	59
-0.02	1211.82	1573.33	0.343	28.663	-18.537	60
0.07	1213.97	1600.00	0.314	29.114	-18.886	61
-0.00	1216.12	1626.67	0.283	29.563	-19.237	62
0.03	1218.22	1653.33	0.250	30.010	-19.590	63
0.03	1220.06	1680.00	0.215	30.455	-19.945	64
0.30	0.08	0.00	-0.113	-0.113	-0.113	1
-0.30	-665.23	-26.67	0.019	-0.461	0.339	2
-0.01	-877.49	-53.33	0.257	-0.703	0.897	3
-1.21	-1081.13	-80.00	0.506	-0.934	1.466	4
-0.66	-1272.76	-106.67	0.759	-1.161	2.039	5
-0.61	-1408.44	-133.33	1.028	-1.372	2.628	6
-1.25	-1513.46	-160.00	1.307	-1.573	3.227	7
-0.34	-1585.01	-186.67	1.602	-1.758	3.842	8
-0.85	-1646.42	-213.33	1.899	-1.941	4.459	9
-0.36	-1684.63	-240.00	2.213	-2.107	5.093	10
-0.12	-1719.60	-266.67	2.527	-2.273	5.727	11
-0.06	-1748.99	-293.33	2.844	-2.436	6.364	12
-1.86	-1777.84	-320.00	3.158	-2.602	6.998	13
-0.70	-1791.63	-346.67	3.491	-2.749	7.651	14
0.35	-1803.61	-373.33	3.824	-2.896	8.304	15
1.35	-1814.12	-400.00	4.156	-3.044	8.956	16
-0.01	-1825.67	-426.67	4.484	-3.196	9.604	17
-0.02	-1834.48	-453.33	4.811	-3.349	10.251	18
0.01	-1841.52	-480.00	5.138	-3.502	10.898	19
0.65	-1846.03	-506.67	5.464	-3.656	11.544	20
-0.08	-1850.60	-533.33	5.784	-3.816	12.184	21
-0.10	-1853.07	-560.00	6.102	-3.978	12.822	22
-0.12	-1854.00	-586.67	6.414	-4.146	13.454	23
-0.14	-1853.32	-613.33	6.719	-4.321	14.079	24
-0.16	-1850.62	-640.00	7.016	-4.504	14.696	25
-0.18	-1845.43	-666.67	7.303	-4.697	15.303	26
-0.19	-1838.44	-693.33	7.574	-4.906	15.894	27
-0.18	-1827.04	-720.00	7.825	-5.135	16.465	28
-0.15	-1808.93	-746.67	8.048	-5.392	17.008	29
1.81	-1778.51	-773.33	8.231	-5.689	17.511	30
0.01	-1724.30	-800.00	8.295	-6.105	17.895	31
0.00	-1663.00	-826.67	8.322	-6.558	18.242	32

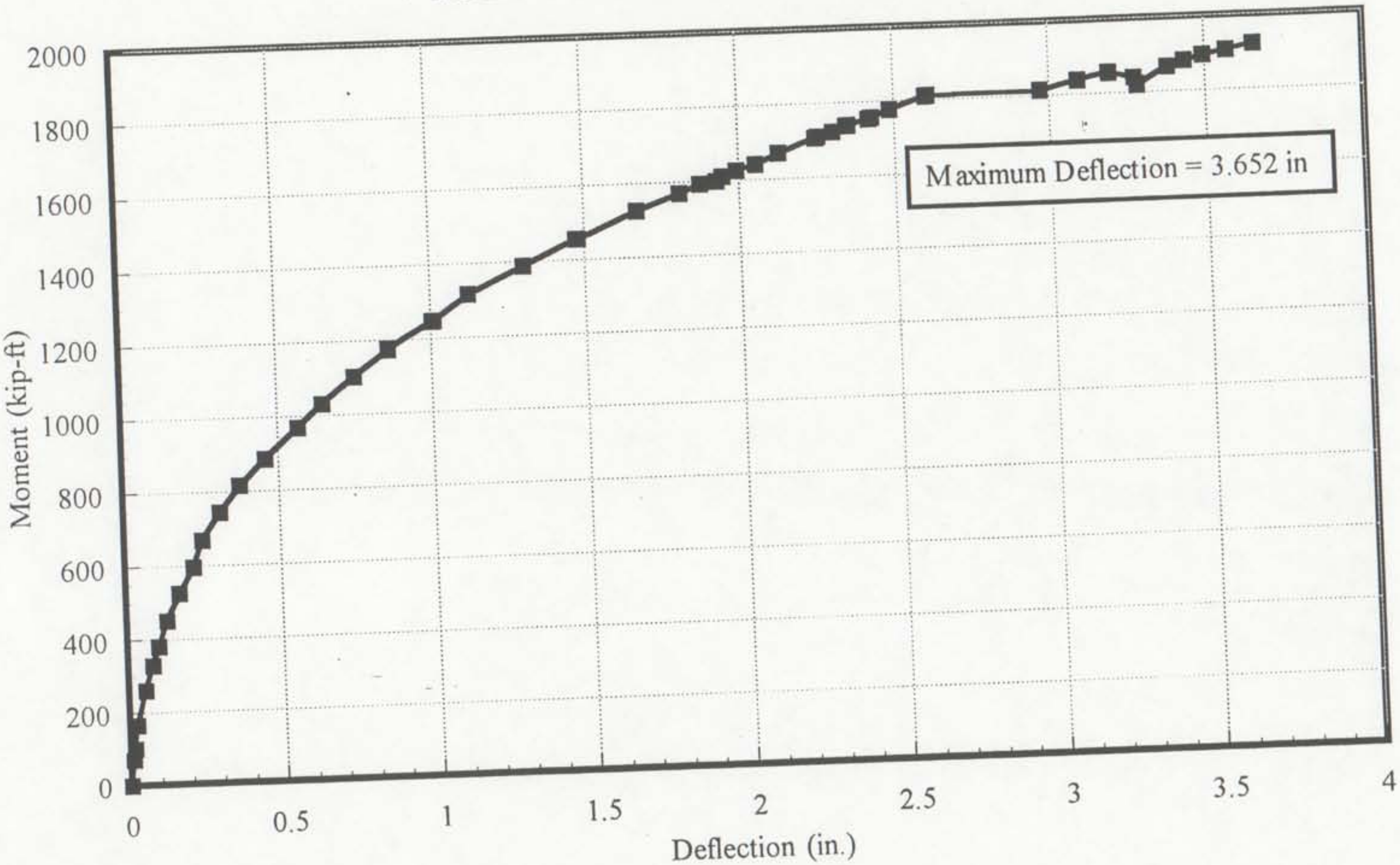
Axial-Load kips	Moment ft*kip	Curvature rad/10^6in	@-Axis	Bottom	Top	Iter.
			[-------Milli-Strain------]			
-0.03	-1608.82	-853.33	8.350	-7.010	18.590	33
-0.00	-1561.06	-880.00	8.381	-7.459	18.941	34
-0.01	-1520.65	-906.67	8.427	-7.893	19.307	35
0.00	-1486.40	-933.33	8.487	-8.313	19.687	36
0.01	-1457.39	-960.00	8.559	-8.721	20.079	37
-1.84	-1433.35	-986.67	8.632	-9.128	20.472	38
-0.04	-1386.73	-1013.33	8.472	-9.768	20.632	39
-0.05	-1345.58	-1040.00	8.302	-10.418	20.782	40
-0.05	-1311.08	-1066.67	8.157	-11.043	20.957	41
-0.07	-1282.28	-1093.33	8.036	-11.644	21.156	42
0.98	-1258.94	-1120.00	7.958	-12.202	21.398	43
0.02	-1247.03	-1146.67	8.026	-12.614	21.786	44
0.01	-1236.85	-1173.33	8.105	-13.015	22.185	45
-0.00	-1228.40	-1200.00	8.190	-13.410	22.590	46
-0.01	-1221.46	-1226.67	8.279	-13.801	22.999	47
1.04	-1215.54	-1253.33	8.379	-14.181	23.419	48
1.18	-1211.08	-1280.00	8.479	-14.561	23.839	49
0.52	-1207.95	-1306.67	8.579	-14.941	24.259	50
-0.92	-1206.04	-1333.33	8.679	-15.321	24.679	51
0.00	-1204.19	-1360.00	8.795	-15.685	25.115	52
-0.01	-1203.41	-1386.67	8.910	-16.050	25.550	53
0.00	-1203.27	-1413.33	9.027	-16.413	25.987	54
0.00	-1203.73	-1440.00	9.148	-16.772	26.428	55
-0.00	-1204.69	-1466.67	9.271	-17.129	26.871	56
0.00	-1206.06	-1493.33	9.397	-17.483	27.317	57
-0.01	-1207.77	-1520.00	9.524	-17.836	27.764	58
-0.00	-1209.72	-1546.67	9.653	-18.187	28.213	59
0.00	-1211.82	-1573.33	9.783	-18.537	28.663	60
0.04	-1213.95	-1600.00	9.914	-18.886	29.114	61
0.09	-1216.08	-1626.67	10.043	-19.237	29.563	62
0.01	-1218.21	-1653.33	10.170	-19.590	30.010	63
0.02	-1220.05	-1680.00	10.295	-19.945	30.455	64

END OF RESPONSE OUTPUT FILE

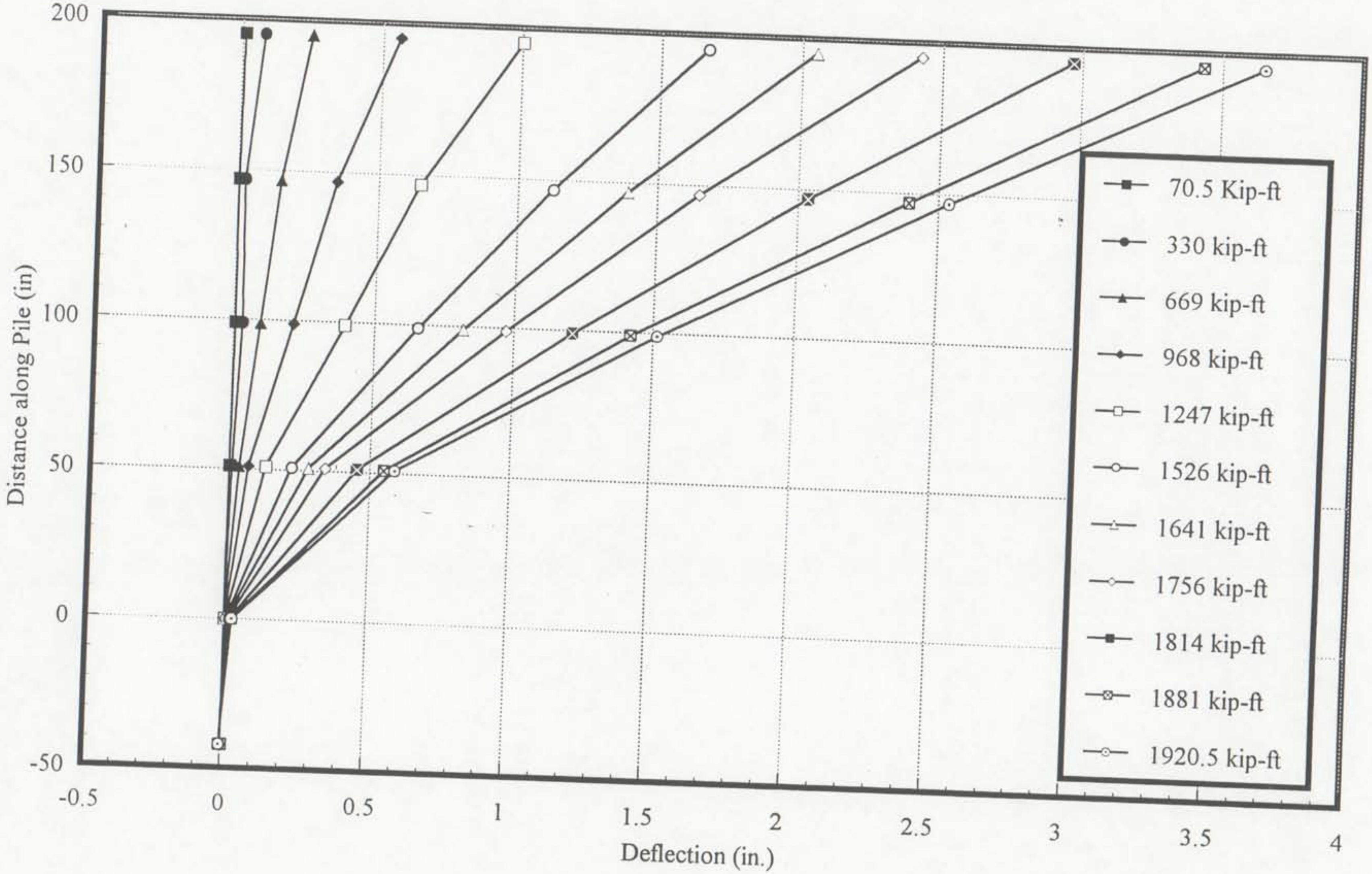
Appendix E

General Plots of Test Results

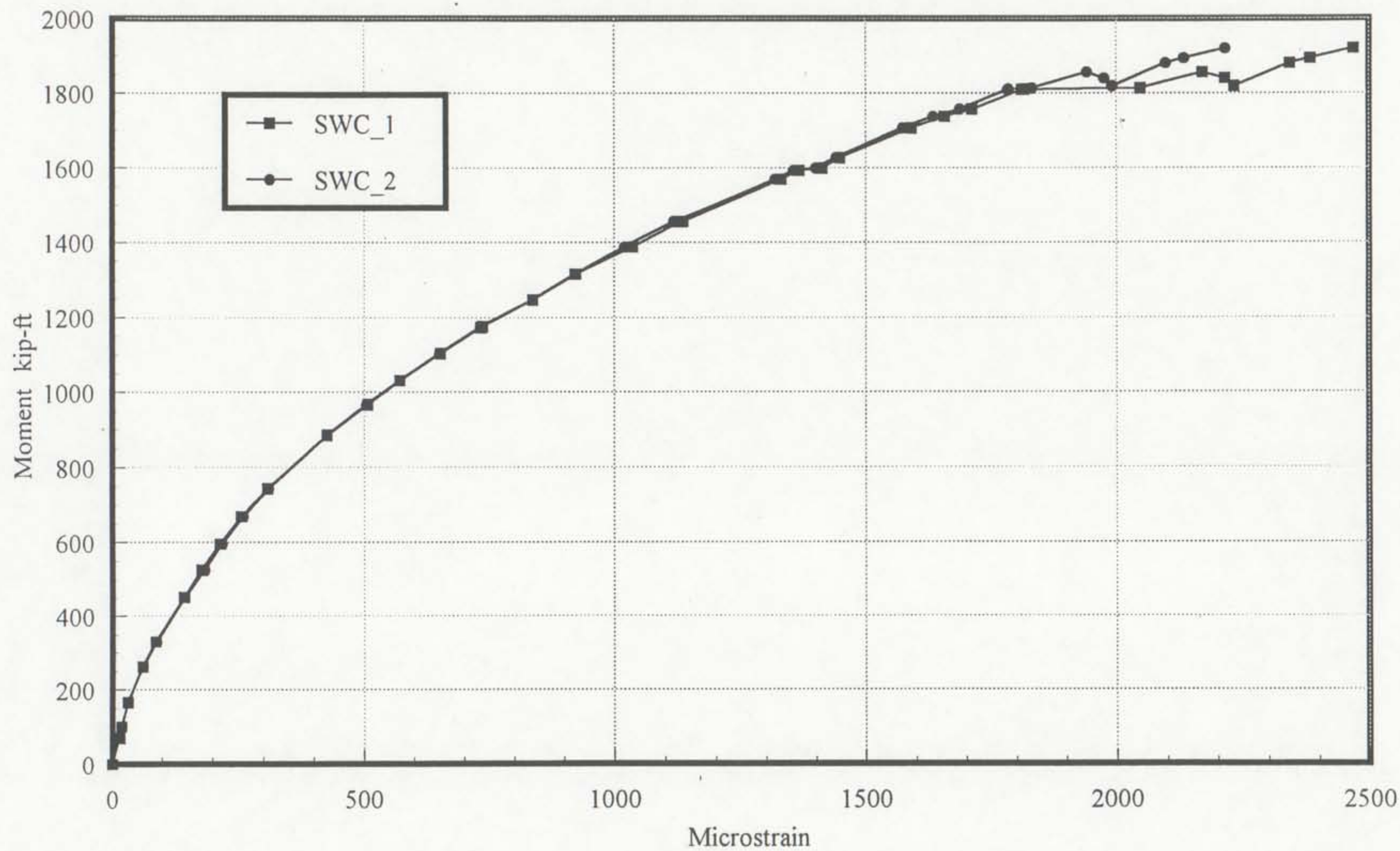
Moment vs Deflection Along West Pile



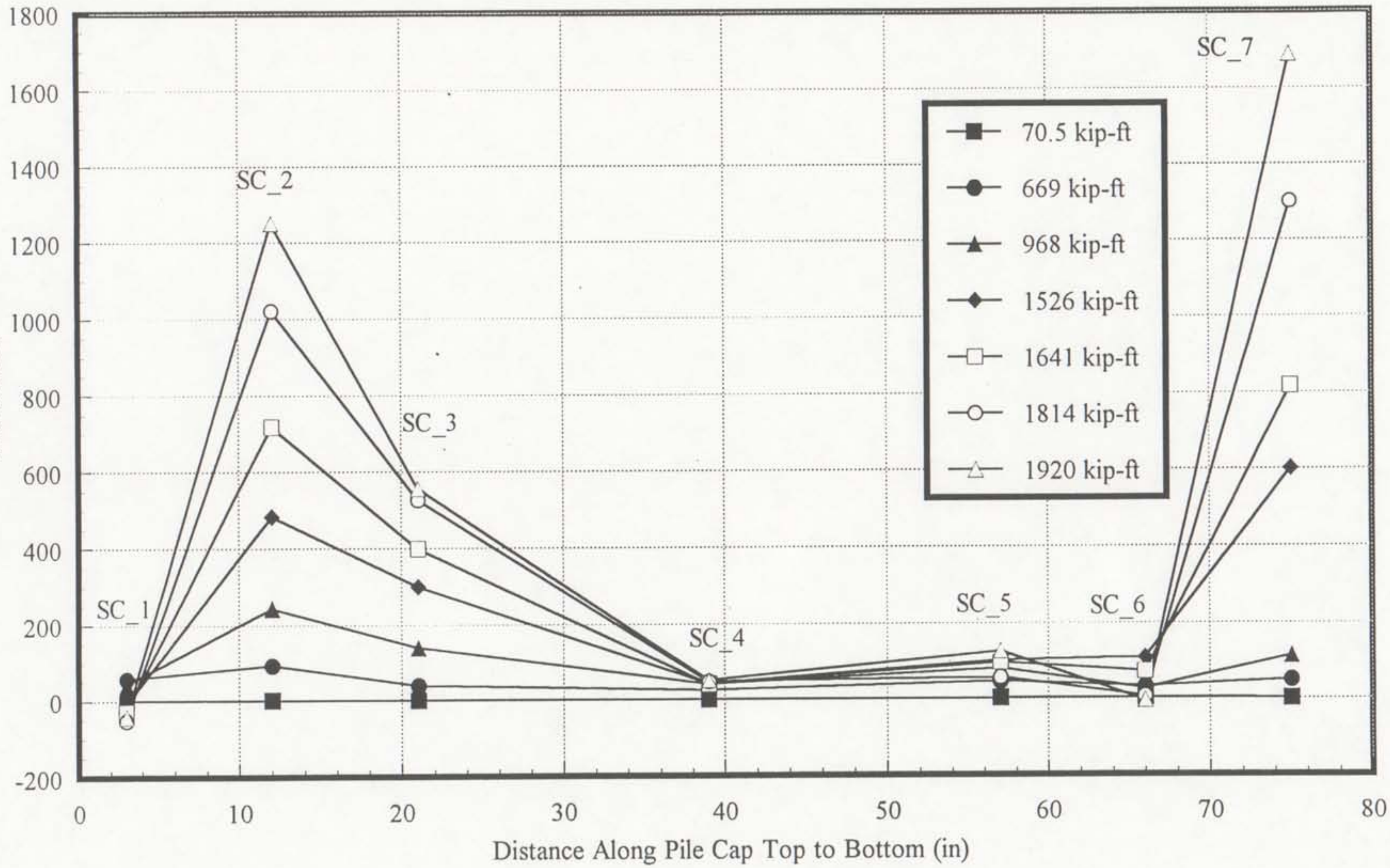
Deflections at Distributed Moments Along West Pile



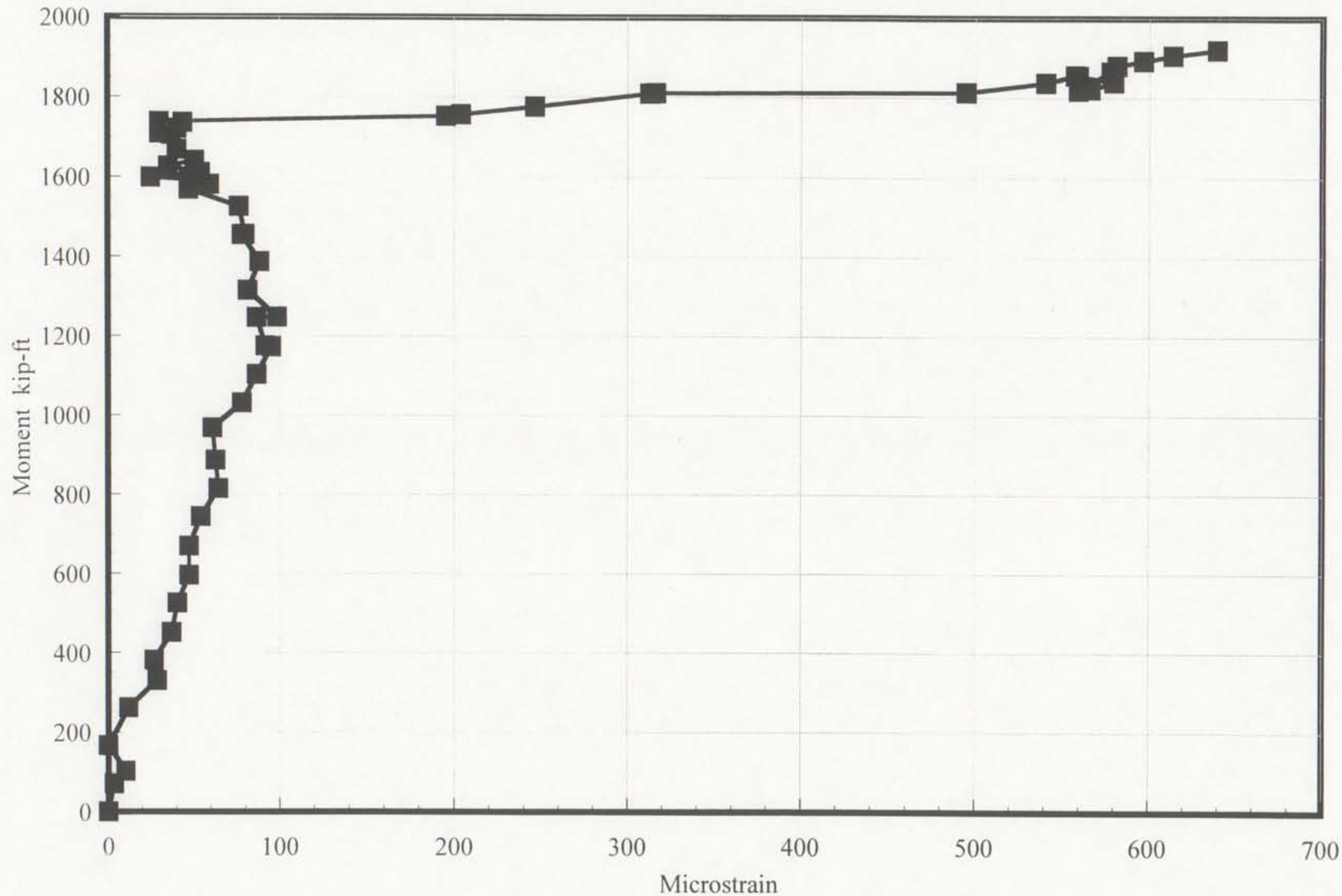
Moment vs Strain West Pile Compression Face Surface Gages



Strain at Distributed Moments Along Pile Cap from Top to Bottom



Moment vs Strain Pile Cap Crack Gage (CRC_8) over West Pile



Moment vs Strain West Pile Embedded Gage ELW_9