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Laterally Loaded Pile Cap Connections

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LATERALLY LOADED PILE CAP CONNECTIONS

by

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GRADUATE COMMITTEE APPROVAL

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BRIGHAM YOUNG UNIVERSITY

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ABSTRACT

LATERALLY LOADED PILE CAP CONNECTIONS

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Master of Science

There is presently considerable uncertainty regarding appropriate connection details between driven piles and pile caps. Prior research on the subject suggests that given a proper embedment length, a specialized reinforced connection may not be necessary. Eliminating these costly connection details could save thousands of dollars on both labor and materials. This research study focuses on the importance of the pile-to-cap connection detail with respect to the reinforcement connection and pile embedment length.

Four pile caps were constructed, each with two 40 foot-long steel pipe piles, and were tested with different connection details. Two caps included a reinforced connection detail while the other two relied on their respective embedment lengths. A hydraulic ram was used to apply a cyclic lateral force to each of these pile caps until failure occurred. Load-displacement curves were developed for each pile cap and

strain gauge measurements were used to evaluate tension and bending moments in the pile caps. Comparisons are presented regarding the effect of the connection on pile cap response. An analysis has been conducted to best understand possible failure modes; two computer modeling programs were used and their respective results have been presented and compared to the observed readings.

This thesis provides test data supporting the theory that a proper embedment length acts as an adequate connection in place of a specialized reinforced detail. A pile cap with piles embedded two diameters into the cap performed successfully. In contrast, a cap with piles embedded only one diameter failed after developing a large crack through the entire cap. For the two pile caps with a reinforcing cage connection; the performance was essentially the same for the piles embedded either six inches (.5 diameter) or twelve inches (one diameter) into the cap. The data produced was found to be very similar to what was estimated by the two programs used for analysis (GROUP 4.0 and LPILE 4.0).

ACKNOWLEDGMENTS

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

SYMBOLS AND NOTATION

a = eccentricity of load or distance from last row of trailing piles to point of rotation

A_c = cross sectional area of concrete under consideration

A_s = area of reinforcement

b = width of member

b' = pile spacing

b_f = flange width of steel pile section

c = clear cover of concrete typically 2 to 3inches

C_m = modified characteristic moment parameter

d = distance to extreme fiber

D = pile diameter

d_b = bar diameter

e = eccentricity from point of zero moment to the center of the effective embedment

E = modulus of elasticity

F = the applied force

f'_c = compressive strength of concrete (psi)

f_y = yield strength of steel

F_y = yield strength of steel

h = distance between strain gages

I = moment of inertia

$K_{m\theta}$ = rotational restraint coefficient

$K_{\Delta c}$ = axial stiffness at the top of the piles in compression

$K_{\Delta t}$ = axial stiffness at the top of the piles in tension

L = distance between string potentiometers

L* = distance from lateral loads point of application to the neutral axis of the joint

l_e = embedment length

L_e = embedment length

L_{emb} = embedment depth

M = observed moment during testing

M'_c = modified characteristic moment

M_c = original characteristic moment

M_f = experimental moment resistance

M_j = nominal moment capacity of concrete pile cap

M_p = plastic moment

M_r = theoretical moment resistance

M_{rc} = moment capacity of a concrete filled circular steel pipe

N_u = factored axial load normal to cross section

s = distance between symmetrically placed A_s and A'_s

S_u = soil undrained shear strength

t = thickness of pipe

y = distance from the neutral axis to the compression fiber

V_u = shear capacity

X_1 = amount of deflection observed from string potentiometers at location 1

X_2 = amount of deflection observed from string potentiometers at location 2

x_i = distance from last row of trailing piles to center of pile

z = embedment depth of pile top below ground surface

Z = plastic modulus of steel section alone

α = concrete factor for reinforcement location

β = concrete factor for coating

σ = calculated stress

δ_v = vertical translation

γ = concrete factor for unit weight

γ' = effective unit weight of soil

γ = unit weight of soil

ε_c = observed strain in compression

ε_t = observed strain in tension

Φ = reduction value phi (.75 for shear)

ω = reinforcement index equal to $A_s/(bl_e)$

1 INTRODUCTION

1.1 Background

Piles are a very common foundation choice for bridges, high-rise buildings and other large structures. These piles must be capable of resisting large lateral forces brought on by earthquakes, wind and wave action. Research has shown that the pile cap connection itself can significantly increase the lateral resistance provided by the foundation against these forces. For example, a pile cap providing a fixed-head boundary will produce a stiffer load-deflection curve than a pile cap which allows rotation. However, relatively little research and testing has been performed to evaluate the effect of the pile to pile cap connection on the degree of fixity and overall response of the pile cap.

This research study has focused on the connection detail between the pile and pile cap and its effect on pile cap stiffness and rotation. In order to analyze a pile head under lateral loading it must be determined whether the connection is in a fixed or pinned condition. From a stiffness standpoint, it is desirable to have a pure fixed head connection yet this is seldom achievable in the field. A design assuming a truly fixed head connection would likely result in underestimated values of deflection, as well as incorrect estimates of the magnitudes and locations of bending moments. On the other hand a design assuming a pinned connection which fails to resist moments could result in a very costly over design.

Previous research and testing has shown that piles embedded a limited depth into the pile cap will resist only shear and axial loads while piles embedded an adequate depth will resist moments as well and significantly reduce lateral deflections. It has been determined that this boundary condition is a function of the pile-to-cap embedment length with less importance on the connecting steel reinforcement. This thesis focuses on this connection as a function of reinforced steel and the embedment length. This design must include a connection able to fully develop the piles' capacity while resisting lateral forces and the accompanying moment.

1.2 Objective and Scope

This research has been undertaken to better understand the importance of pile cap connections on lateral pile cap and abutment behavior. The goal in connection design is to provide a connection capable of developing moment capacity equal to the moment demands on the pile while remaining essentially rigid. Ideally, it is desired to eliminate the special reinforcement details and rather provide a proper pile embedment length. This would result in a simpler construction process and lower overall cost.

2 DESCRIPTION OF PROBLEM

2.1 Behavior of Laterally Loaded Pile Groups

Piles are most often placed in groups with a variety of alignment and spacing arrangements. The piles are then capped with a concrete pile cap which encases the piles. On occasion individual piles are used, though this is less common in the field. Driven pile foundations typically consist of steel pipes filled with concrete, steel H sections or pre-stressed concrete. Pile groups perform differently than single piles, due to the soil-pile-soil interaction which is a function of pile spacing. The larger the spacing, the less the overlapping of shear zones and the greater the lateral pile resistance.

Typically, the foundation system is designed so that its capacity will exceed that of the column or structural system above ground. This approach ensures that damage will occur above ground where it can more easily be detected and repaired. Therefore, the designer must be certain that the foundation system will develop its full design capacity. For lateral load conditions, the moment capacity of the pile foundation will typically govern the pile section properties. For a fixed-head pile group the maximum negative moment occurs at the base of the pile cap while the maximum positive moment occurs in the pile at a short depth below the base. It is, therefore, desirable to construct a pile cap that will be strong enough so that the pile can achieve its full moment capacity. In this regard, the connection must be able to resist the large negative moment for the foundation system to be considered efficient. As indicated previously, the moment capacity at the connection depends on both the depth of embedment of the pile and the reinforcement arrangement. This research and testing, which focuses on these issues, is therefore very important to future design and construction of pile systems.

2.2 Literature Review

Due to the extensive use of piles in foundation systems several publications relating to projects or research are available for review. A literature review was conducted to obtain all possible research and/or testing concerning laterally loaded pile caps and their connections. There are multiple issues related to pile foundation system that have been researched and tested prior to this study. While many of these studies focused on the resistance of the surrounding soil or other elements in pile design they were still found to be useful in better understanding the behavior of the pile foundation system. The following is a summary of those publications found to have similar or valuable data with respect to this study. The publications reviewed have been divided into two groups: laboratory and field tests, and modeling and analysis, and then arranged chronologically within these groups.

2.3 Laboratory and Field Test Reviews

2.3.1 Embedded Steel Members (Marcakis, K., and Mitchell, D. (1980))

Marcakis and Mitchell developed an analytical model considered to be conservative in determining a connections capacity based on the results of a series of 25 tests with varying parameters ranging from welded or embedded H piles, pipe piles filled with concrete and empty, to standard steel plates. The current design method outlined in the PCI Design Handbook for connections incorporating embedded structural steel has shown to have several inconsistencies. Multiple design charts were then developed with varying material properties, Figure 2-1 shows an example of one of those charts. With most material properties and member dimensions known the designer would choose suitable values of the embedment length and width, eccentricity, and effective width and be able to enter the appropriate design chart to determine a proper reinforcement connection.

$$V_u = \frac{.85 f_c^{'} b^{'} (L_e - c)}{1 + \dfrac{3.6e}{L_e - c}} \tag{2-1}$$

Marcakis and Mitchell proposed equation 2-1 based on a strut-and-tie approach and using stress distributions along the embedment zone to determine the required embedment length. The moment capacity of a connection can be determined by multiplying the shear capacity as determined in equation 2-1 by the eccentricity from point of zero moment to the center of the effective embedment (e).

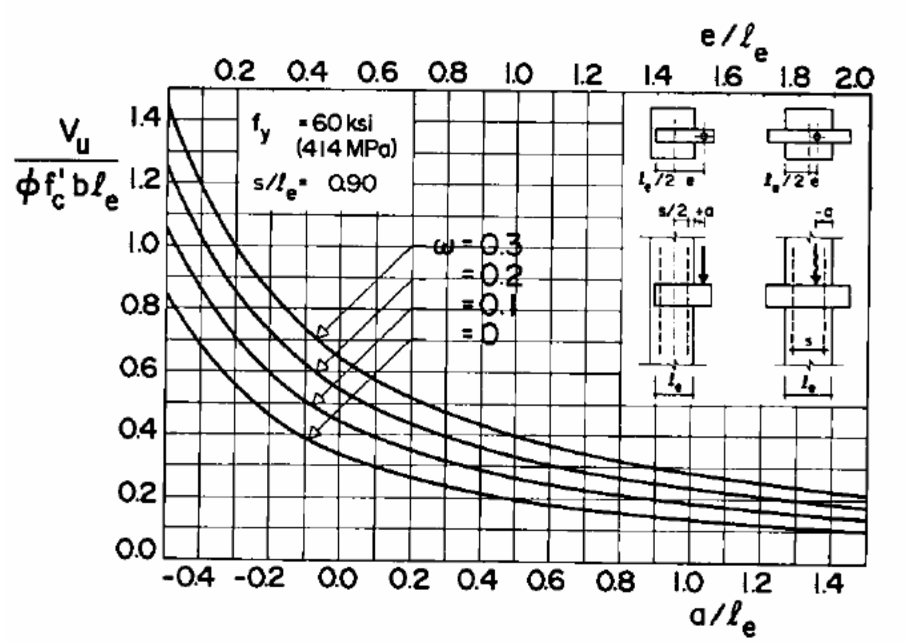


Figure 2-1 Connection design chart by Marcakis and Mitchell.

2.3.2 Lateral Resistance Provided by Pile Groups (Mokwa, R. L. 1999)

Through a comprehensive literature review and over 31 field tests performed on pile groups Mokwa showed that passive pressure on a pile cap provides considerable resistance to lateral loads and that neglecting this resistance could lead to inaccuracies of 100% or more. The literature review evaluated the most widely used techniques as well as the most accurate design methods. Thirty-seven experimental studies were reviewed which provided information on the effects of pile group behavior, thirty of which addressed behavior of laterally loaded pile groups. A group efficiency factor G_e and p or y multipliers represent two of the most common approaches for accounting for pile group interaction effects. Tests showed that the pile caps provide approximately 50% of the overall lateral resistance of the pile group foundation. This was confirmed by previous load tests performed by Beatty (1970), Kim and Singh (1974), Rollins et al. (1997), and Zafir and Vanderpool (1998) which also showed the cap contributing about 50% of total lateral resistance.

The lateral resistance is a function of many factors. These factors, in order of importance, are: stiffness and density of soil in front of the cap, depth of cap embedment, rotational restraint of pile head, pile group axial capacity, and stiffness and density of soil around the piles. An analytical method was also developed using computer programs such as: PYPILE, PYCAP, and LPILE which involve developing p-y curves or a group-equivalent pile value.

2.3.3 Retrofit of Steel Piles to Concrete Caps (Shama, Ayman, and Mander, 2001)

Finite element modeling and results from two full scale tests on pile-to-cap connections were conducted to develop equations for both design and retrofits. Two HP pile groups, representative of construction practice in the eastern U.S., were constructed in a laboratory and tested under cyclic axial and lateral loading until failure. A moment capacity equation was developed and is presented below that was proven helpful in predicting connection performance. A pile–to–cap efficiency ratio (ρ) was created comparing the moment capacity of the pile to the moment capacity of the concrete-pile connection. Figure 2-2 shows the assumed stress distribution through the connection zone.

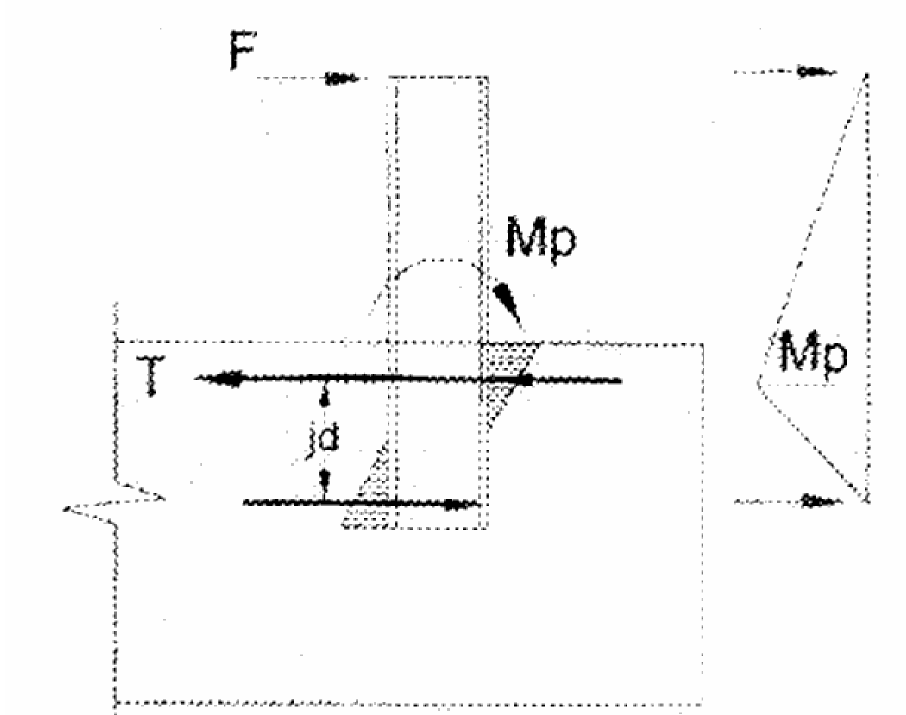


Figure 2-2 Assumed theoretical stress distributions.

Equation 2-2 was derived based upon the theoretical stress distributions shown in Figure 2-2 and results from the load tests.

$$M_j = \frac{f_c b_f L_{emb}^2}{6 + \frac{L_{emb}}{L^*}} \tag{2-2}$$

The moment capacity at the connection is a function of the concrete crushing. As the embedment length is increased the crushing area of concrete increases proportional to the width of the pile which produces a significant larger compression area and therefore larger moment capacity.

2.3.4 Behavior of CIP Pile Cap Connections (Harries, K.A., and Petrou, M. F. 2001)

This study combined previous test results with results from two new lab tests on full-scale pile-to-pile caps with different connection details to provide evidence that no special details are necessary if the proper embedment length is provided. The pile-to-cap assembly was tested as a cantilever beam in a horizontal position. Two separate tests were performed each consisting of an 18 inches square x 18 feet long precast concrete pile embedded 18 inches and 24 inches into a 7 feet x 7 feet x 3 feet pile cap. Each cap was reinforced with No. 7 longitudinal bars on the top and bottom at 6 inches spacing and No. 3 ties at 6 inches spacing in the transverse direction and through the depth of the pile cap. Both piles first began to crack at the interface with a moment of 169 ft-kips and yield displacement at the interface was measured to be 1 inch, which occurred at a moment of 246 ft-kips. Harris and Petrou concluded that the embedment lengths were sufficient to develop the moment capacity without a special connection detail. They concluded that an embedment length equal to the pile width would be sufficient to develop the moment capacity of the pile. This condition would provide a "weak pile, strong pile cap" behavior that permits easier inspection and repair in the event of an earthquake.

2.3.5 Seismic Limit States for CISS Piles (Silva, P. F., and Seible, F. 2001)

Observations from two large-scale cast-in-steel-shell (CISS) piles were correlated with analytical predictions to establish performance limit states. These piles were designed according to Caltrans specifications and built at a 7/12 scale.

Both piles were composite steel shell piles, the first with an unreinforced concrete core and the second with a reinforced concrete core; refer to Figure 2-3 for the piles specifications.

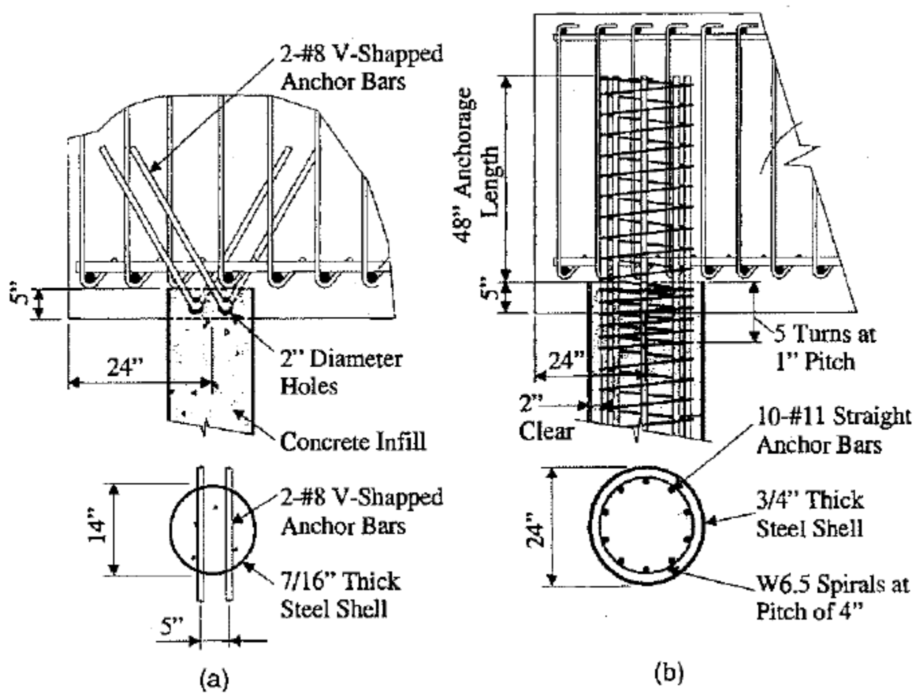


Figure 2-3 Piles and cap configurations.

Test results showed that pile performance due to seismic or any type of lateral force is highly dependent on the embedment length and connection type. Another important observation was that when piles are subjected to combined axial and lateral loads fracture can occur within the connection reinforcement below the design tension load. Observations from both the tests and analytical data were collected in order to develop limit states and better define and predict damage levels.

The following six limit states were defined by Silva and Seible;

- Pile elastic limit – defined based on a performance level such that any noticeable damage does not require repair. This was the first limit state noticed for both tests and was indicated by the development of thin cracks which emanated at 45 degrees from the pile base to the sides of the cap.
- Pile cap concrete cover spalling – occurs due to rotation and prying and is evidenced by extensive damage to the pile cap concrete cover. Unlike the first test this limit state occurred near failure in test two.
- Pile cap joint region cracking – defined as an onset of joint shear cracking typically occurring simultaneously with the pile elastic limit state and visible in both tests by cracks emanating from the seating region. Also defined when principal stresses in the joint region exceed $3.5\sqrt{f_c'}$.
- Pile functional evaluation limit state – moderate damage occurs at this limit state yet the structure does not lose strength and no exposure to reinforcement occurs. Also defined when the anchor bars exceed a strain of 0.0325.
- Pile cap joint shear failure – defined when the principal tensile stresses exceed $5\sqrt{f_c'}$, which correspond to poorly reinforced concrete. This was found to occur only in test two.
- Pile safety evaluation limit state – significant damage occurs, requiring repair or replacement of the structure. This was found to be the limit state in test one with the strain in the anchor bars exceeding the maximum allowable of 0.065.

Test two failed due to large rotations causing exposure to the pile cap reinforcement along the bottom layer.

A better understanding of the limit states defined above will allow the designer to account for inelastic deformations in the piles, thus reducing the number of piles required and the size of the pile cap. This will also reduce the stiffness in the foundation system thus decreasing the column displacement ductility demand. These significant changes will lead to a more economical foundation design and reduce the damage in the column under a seismic event.

2.3.6 Concrete Filled Circular Steel Bridge Piers (Bruneau, M. and Marson, J. 2004)

Bruneau and Marson conducted full-scale laboratory tests on steel pipe with reinforced concrete infill in an effort to evaluate existing design codes used throughout the world to compute moment capacity. Multiple codes exist throughout the world and each has its own equations and assumptions to determine proper design limits. Unfortunately, the accuracy of the various methods and their relative differences are largely unknown. Four specimens were tested with the load applied laterally at the end of the pipe and the failure occurring at the concrete foundation. Table 1 shows the moment capacity from test data and predictions from five separate codes. It is noted that the AISC LRFD 1994 edition underestimated strength capacities by a significant margin while the Eurocode 4 (1994) proved to be the most accurate.

Equation 2-3 was developed to better calculate the moment capacity of a pipe pile with concrete fill. It was also shown that whether the concrete in the pipe is strengthened with reinforcement or not it still provides confinement and delays local buckling.

Table 1 Experiment to calculated strength ratios for specimens tested.

Code	CFST 64 $P = 1000$ kN		CFST 34 $P = 1820$ kN		CFST 42 $P = 1820$ kN	
	Strength (kN m)	M_f/M_r	Strength (kN m)	M_f/M_r	Strength (kN m)	M_f/M_r
Test data	591		444		928	
AISC LRFD (1994)	362	1.64	234	1.90	681	1.36
CAN/S16.1-M94	314	1.88	255	1.74	608	1.53
Eurocode 4 (1994)	522	1.33	402	1.10	918	1.01
CAN/S16.1-M99 (proposal A)	492	1.20	387	1.15	911	1.02
CAN/S16.1-M99 (proposal B)	519	1.14	380	1.17	897	1.04

$$M_{rc} = (Z - 2th_n^2)F_y + \left[\frac{2}{5}(.5D - t)^3 - (.5D - t)h_n^2\right]f_c' \tag{2-3}$$

where

$$h_n = \frac{A_c f_c'}{2Df_c' + 4t(2F_y - f_c')}$$

2.3.7 Steel Pipe Pile-to-Concrete Bent Cap Connections (Montana State University)

Jerry E. Stephens and Ladean R. McKittrick performed laboratory tests on five half-sized steel pipe columns embedded in a concrete pile cap and filled with unreinforced concrete. For each test the pile was embedded 9 inches into the cap with no other reinforcing details provided. Refer to the photograph of the test setup presented in Figure 2-5. With each additional test the amount of steel in both the longitudinal and transverse directions were increased in an effort to evaluate the importance of reinforcing steel in the pile cap to the caps overall moment capacity.

By using ½ scale models there was only 4.5 inches of concrete cover provided around the pile; it is recommended that at least 1 foot of concrete surround each pile.

Tests 1 and 2 had pile cap reinforcing steel ratios in the longitudinal and transverse directions of 0.41 and 0.09%, respectively. With increasing lateral loads the caps failed through concrete cracking in the cap, it appeared the reinforcing steel was unable to carry the tension forces. Tests 3 and 3a had increased steel ratios and the same type of failure occurred. Test Cap 4 had longitudinal and transverse ratios of 2.83 and 0.7% respectively, and this caused a failure in the form of a plastic hinge in the steel pile and only nominal concrete cracking was noticed. With the dramatic increase in steel for Test Cap 4 as shown in Figure 2-4, constructability concerns developed regarding the amount, size and spacing of the reinforcement.

Hand calculations and Finite Element Modeling were used to analyze each of the four tests. The simple hand calculations proved valuable in predicting the nature of failure though were less accurate in predicting the load at which failure occurred. The finite element analysis did not appear to be capable of accurately modeling concrete damage under cyclic loads.

Figure 2-4 Reinforcing cage for pile cap Model 4 Montana State University.

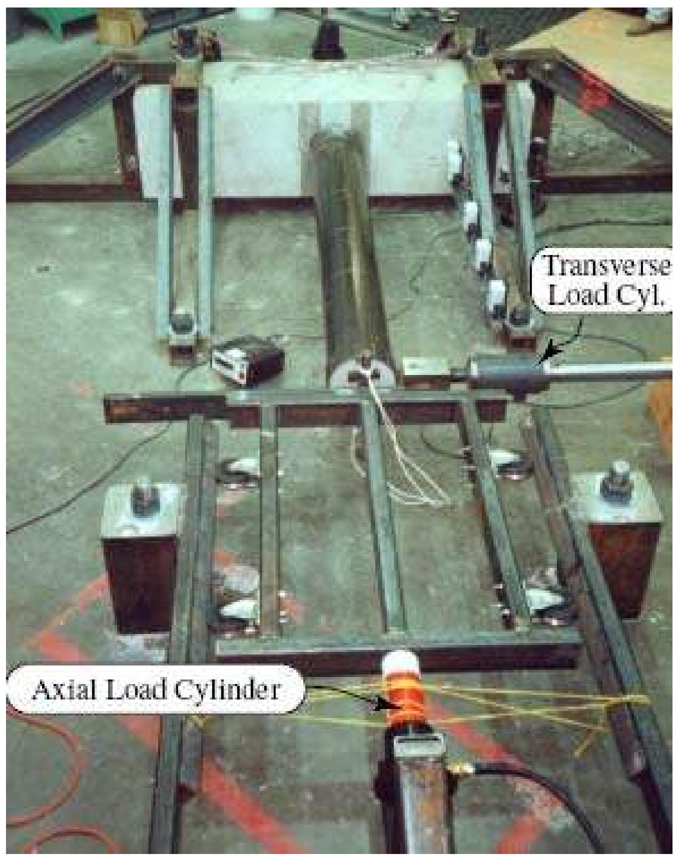


Figure 2-5 Pile and cap setup for testing at Montana State University.

2.4 Case Studies and Computer Modeling Reviews

2.4.1 Fixity of Members Embedded in Concrete (Army Corps of Engineers 1984)

The Army Corps of Engineers builds many structures such as bridges, locks and buildings that utilize pile foundations. This has been noted as significant part of the overall cost of construction. To better understand the ability to achieve a fixed head connection was undertaken by Fernando Castilla, Phillippe Martin, and John Link. They utilized finite element and finite difference computer modeling programs such as CERL, ANSYS, and COM 622 to better understand the situation.

Although previous Corp design practice assumed that an HP pile embedded 1 foot into a pile cap would act as a pinned connection, computer analysis in this study indicated that such a value was unrealistic. According to the analysis, a 1 foot embedment length actually developed 61-83 percent of the fixed-head moment and therefore could be considered partially fixed. The study concluded that for HP piles the ratio of embedment length to pile width should be greater than two in order to obtain full fixity.

2.4.2 Rotational Restraint of Pile Caps (Mokwa, R. L., and Duncan, J. M. 2003)

Using data from testing in 1999, Mokwa and Duncan developed a procedure to estimate the moment restraint which would allow proper estimation of the actual pile head rotational stiffness which would be between the fixed and free conditions. The value of the rotational restraint coefficient, $K_{m\theta}$ is a function of the amount of movement required to mobilize the tensile and compressive loads in the pile. The amount of rotation is a function of the magnitude of the lateral load and rotational stiffness $K_{m\theta}$. Figure 2-6 shows the free body diagrams used to derive the equation for $K_{m\theta}$ (equation 2-4).

$$K_{M\theta} = \frac{\Delta M}{\Delta \theta} = \sum_{i=1}^{n} \left[K_{\Delta c} (x_i - a)^2 \right] + \sum_{i=1}^{n} \left[K_{\Delta t} (x_i - a)^2 \right] \quad \textbf{(2-4)}$$

Figure 2-7 shows one of the load-deflection curves from testing and compares them to the curves predicted using fixed-head and free-head conditions as well as the rotationally restrained stiffness defined using equation 2-4.

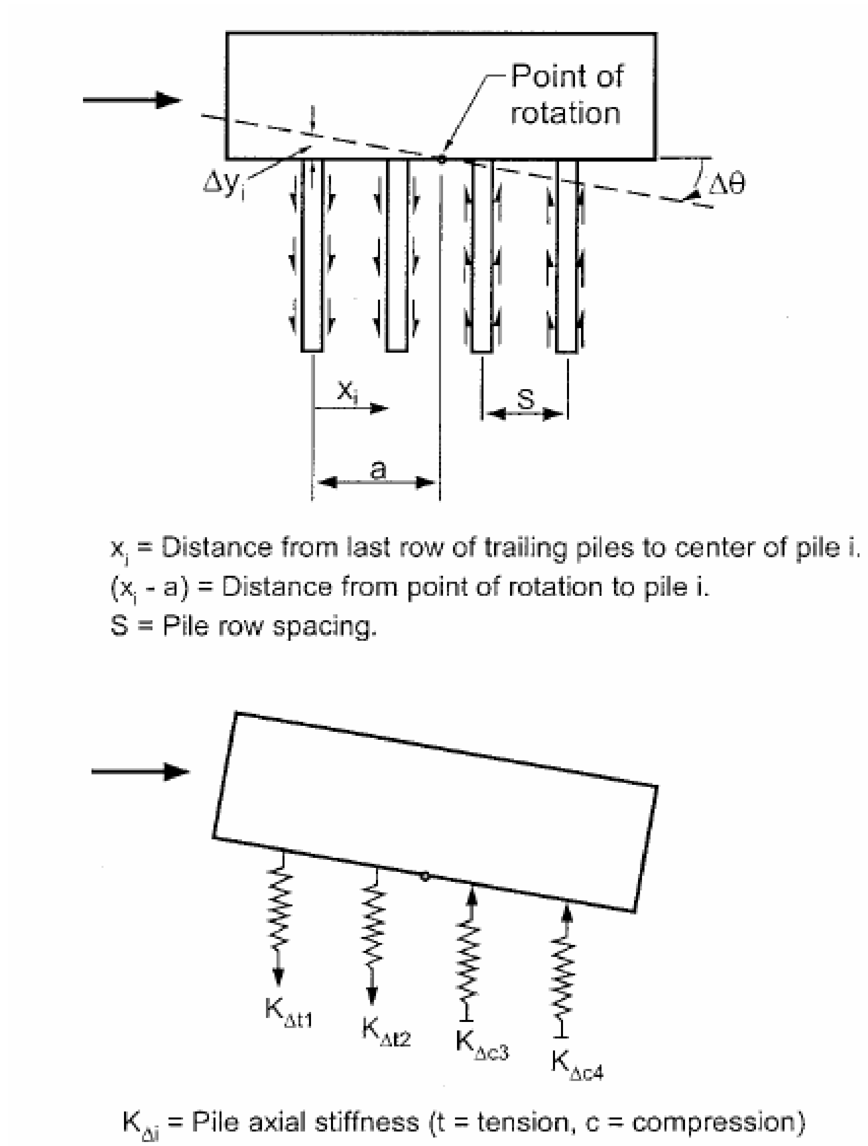
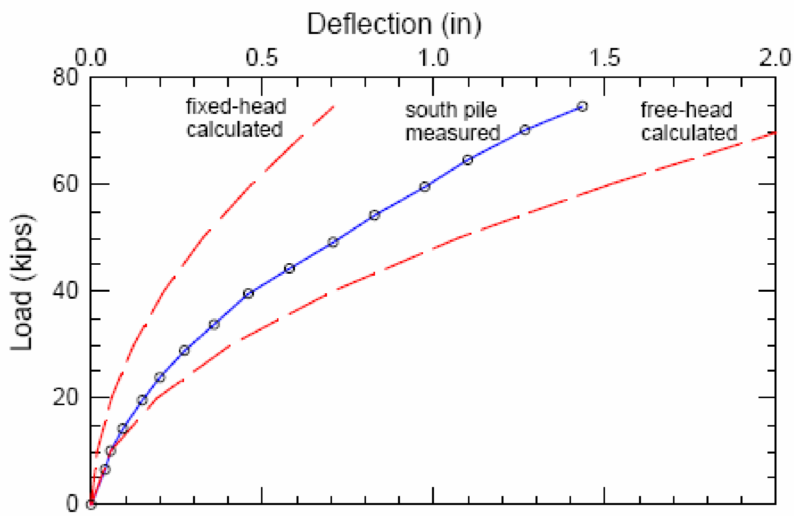
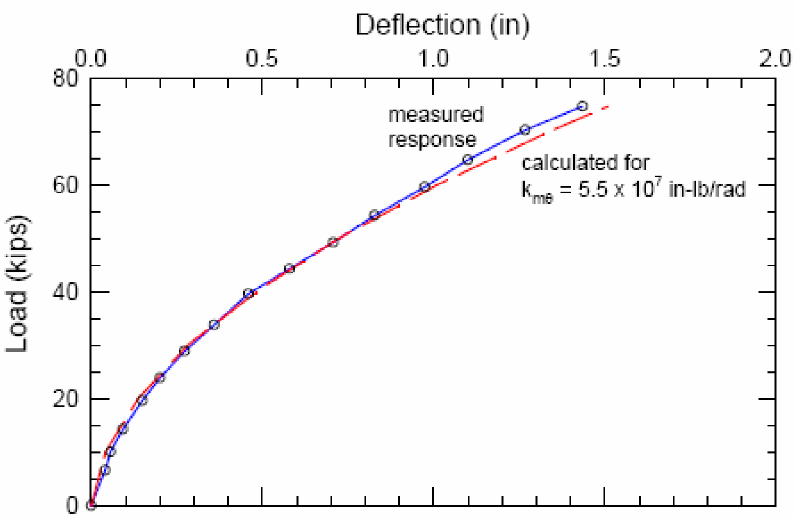


Figure 2-6 Free body diagrams.



(a) Fixed-head and free-head boundary conditions.



(b) Rotationally restrained pile-head boundary condition, best fit $k_{m\theta}$ value.

Figure 2-7 Load vs. Deflection curves comparing boundary conditions.

2.4.3 The Modified Characteristic Load Method (Ooi, P., Chang, B., Wang, S. 2004)

The characteristic load method (CLM) used to estimate lateral deflections and maximum bending moments in piles has been found less useful due to its limits and assumptions. CLM does not account for embedment depth, or pile group interaction which with recent research has been found to have a greater impact on the overall resistance of a pile system. A modified characteristic load method (MCLM) has been developed to account for these factors and then used to evaluate five case studies from around the world.

It was concluded that MCLM provided reasonable estimates of pile group behavior and also agreed with generally accepted computer models such as GROUP. Figure 2-8 compares the predicted and measured lateral displacements for a case study from Las Vegas Nevada. This test consisted of a 2 X 2 group of drilled shafts laterally loaded with the soil around the cap completely excavated. The proposed method as well as GROUP provided very accurate estimations of deflection. The case studies evaluated also indicate that pile groups appear to act as a fixed head condition at small lateral loads with the degree of fixity decreasing at higher loads. This could be caused by multiple scenarios: reduced rotational restraint, insufficient embedment, inadequate reinforcing of the pile to the cap, and/or pile foundation cracking.

Equations 2-5 to 2-7 presented show the procedure developed in the Modified Characteristic Load Method. Essentially the moment calculated from the CLM is multiplied by a correction factor C_m. This factor as presented above takes into account the embedment depth, pile diameter, and soil parameters that were not accounted for in the CLM. The equation for C_m depends on whether it is in clays or sands, equation 2-6 should be used when dealing with clays and equation 2-7 when dealing with sands.

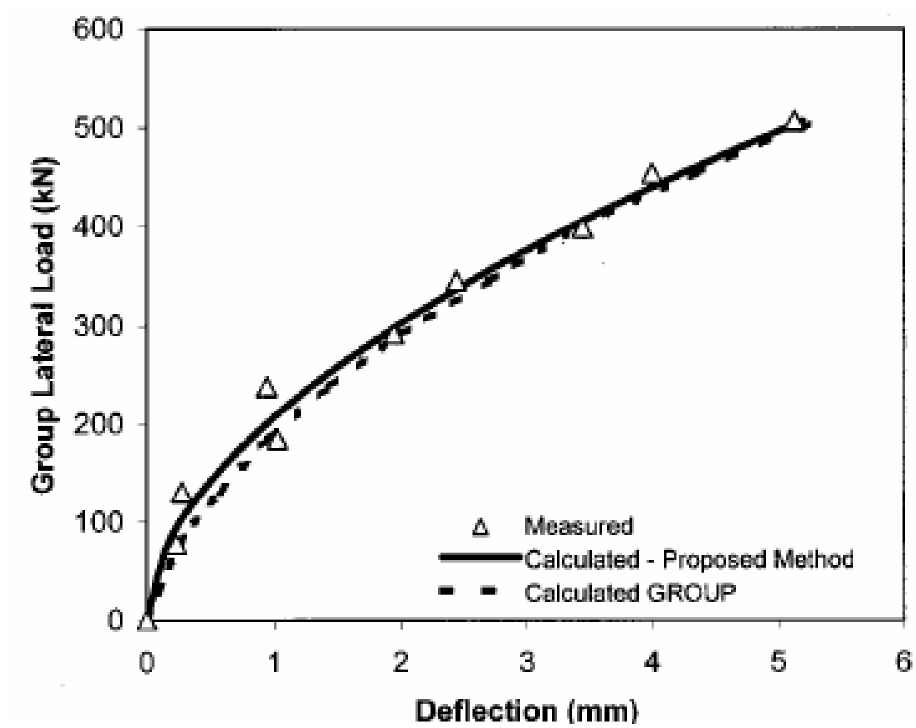


Figure 2-8 Predicted and measured displacements for second Las Vegas load test.

$$M'_c = M_c C_m \quad \textbf{(2-5)}$$

For Clays

$$C_m = \left[1 + \frac{z}{60D}\right]^{\frac{S_u}{10yD}+.2} \quad \textbf{(2-6)}$$

For Sands

$$C_m = \left[1 + \frac{z}{2D}\left(\frac{y'}{y}\right)^{.1}\right]^{.03\phi-.65} \quad \textbf{(2-7)}$$

2.5 Summary of Literature Review

There has been a significant amount of research conducted pertaining to the lateral resistance of the pile foundation system. There has also been a considerable amount of testing conducted aimed at developing equations to evaluate the moment capacity at the pile to pile cap connection. All of the research and testing reviewed has established the connection detail as a crucial element in developing the piles capacity. Some of the most valuable points are presented here:

1. Kostas Marcakis and Denis Mitchell produced multiple design charts based on testing enabling the designer to come up with a proper connection. An example of these charts is presented in Figure 2-1. Equation 2-1 was developed incorporating the importance of the embedment length by calculating the stresses at the connection face.
2. Robert Mowka conducted extensive research regarding piles and group effects, and developed a rotational restraint coefficient $K_{m\theta}$. The amount of cap rotation is a function of the lateral load as well as the coefficient $K_{m\theta}$ shown in equation 2-4.
3. Equation 2-2 was derived based upon the theoretical stress distributions shown in Figure 2-2 and results from load tests. The moment capacity at the connection is a function of the concrete crushing.
4. After testing two precast concrete piles the recommended embedment length should be taken as the larger of the piles diameter or 12 inches.
5. Based off of two 7/12 scale tests 6 performance limit states were well defined that account for inelastic deformations in the piles.

6. Half scale single piles were tested at Montana State University with various amounts of steel in the cap which showed that the moment capacity of the pile system is also a function of the caps reinforcement.
7. The Army Corps of Engineers utilized finite element and finite difference computer modeling programs and determined that an HP pile embedded 1 foot would provide up to 83 percent of the fixed-head moment and therefore could be considered partially fixed. The study concluded that for HP piles the ratio of embedment length to pile width should be greater than two in order to obtain full fixity.
8. The Characteristic Load Method (CLM) was modified to account for group interaction effects and when compared to computer programs such as GROUP found to be quite accurate in estimating deflections. It is noted that the current research also uses GROUP to estimate deflections and rotations.

Table 2 summarized the publications reviewed that directly pertain to a pile caps connection under lateral loads. While much research and testing has been conducted on pile groups only few are related to the connection between the pile and pile cap; those that have been reviewed are summarized in this table. By preparing this table it is noted that only a few tests have been performed all of which have been conducted in a laboratory with a similar test setup. By conducting this literature review the results of the current testing can be better understood.

Table 2 Summary of reviewed pile cap tests.

Title	Test	Pile characteristics	Cap Characteristics	Connection	Objective	Reference
Retrofit of Steel Piles to Concrete Caps	Test 1 and 2 Full Scale Laboratory	HP10X42	7’ x 9’ x 3’ CIP	12” embedment	Define criterion for pile system retrofits	ACI Structural Journal, V.99, No.1, 2001, pp 185-192
Behavior of CIP Pile Cap Connections	Test 1 Full Scale Laboratory	18”x 18” x 18’ Prestressed Concrete	7’ x 7’ x 3’ CIP	24” embedment	Show that no special connection detail is required	PCI JOURNAL, V. 46, No. 4, July-August 2001, pp.82
	Test 2 Full Scale Laboratory	18”x 18” x 18’ Prestressed Concrete	7’ x 7’ x 3’ CIP	18” embedment	Show that no special connection detail is required	
Seismic Limit States for CISS Piles	Test 1 Full Scale Laboratory	14” Steel Pipe with unreinforced concrete fill	24’ x 24’ x 5’ CIP	5” embedment with 2 #8 V-shaped bars 30” long	Define Limit States	ACI Structural Journal, V.98, No.1, 2001, pp 36-49
	Test 2 Full Scale Laboratory	24” Steel Pipe with reinforced concrete fill	24’ x 24’ x 5’ CIP	5” embedment with 10 #11 bars 53” into cap	Define Limit States	
Steel Pipe Pile-to-Concrete Bent Cap Connections	Tests 1-5 ½ scale Laboratory models	8” Steel Pipe with unreinforced concrete fill	69”x 18”x 18” CIP	9” embedment	Test systems capacity with various steel ratios	Report No. FHWA/MT-05-001/8144

2.5.1 Limitations of Current Understanding

The current expectation that one foot embedment length is adequate has only been tested in laboratories. Previous research has focused on the connections' ability to resist large moments with failure mainly consisting of concrete crushing. This is mainly due to the testing procedures which include fixing the pile cap while applying the lateral force to the tip of the pile, though under a seismic event the cap is free to move and rotate. These limitations indicate the importance and need to better understand how a pile group will react under an actual seismic event. The current testing addresses these limitations by applying the force on the pile cap while the pile remains in the ground.

Eliminating special reinforced connections has not yet been accepted in design. In fact much of the current pile group design includes not only a special reinforced connection detail but also a significant embedment length. The current research involves full scale field tests which will consider in-situ effects and allow the cap to move and rotate more resembling an actual seismic event. This research will clearly contribute to a better understanding how pile groups act under large lateral forces.

3 TEST SETUP

3.1 General Remarks

A total of four pile caps were tested each supported by two piles driven to a depth of 40 feet. These four pile caps were laterally loaded independent of each other using a hydraulic ram. As indicated in the literature review, the majority of tests involving pile caps have been performed on either scale models or on laboratory specimens. These tests are significant in that they consider the complete pile/pile cap/soil system under in-situ conditions rather than a laboratory setting. Also, prior testing has fixed the pile cap and applied the lateral force to the tip of the piles without soil involved. However, under in-service load conditions, the pile cap would not be fully fixed. This test setup also takes into account the pile group interaction effects while the prior testing typically included only single piles.

The purpose of this testing is to compare the performance of four connection details between the piles and the pile cap. There are two basic details involved with the connection between the pile and the pile-cap. The first detail involves the length to which the pile is embedded into the pile cap and the second is the reinforcement connection extending from the pile cap a proper development length into the pile.

Therefore each of the four pile caps were configured with the same geometry with the exception of the connection.

Prior research has shown that a proper embedment length alone can be sufficient to develop the moment capacity of the pile and it may suffice to ignore any type of reinforcement connection which can be very costly to both fabricate and construct in the field. Another type of practice although less common involves leaving the piles hollow; this lack of concrete makes a reinforced connection more difficult to fabricate and analyze. As shown in the literature review section, a length equal to at least one pile diameter should be embedded into the pile cap to fully develop the moment capacity of the pile. To evaluate this finding under field conditions, it was decided to test a pile cap with piles embedded one pile diameter and compare its performance with pile caps that have shorter as well as longer embedment lengths.

3.2 Site Description

The site used for the construction and testing of all four pile caps was located at 600 N and South Temple in Salt Lake City Utah. This is a Utah Department of Transportation (UDOT) test site where other pile testing had been performed previously. The soil profile at the test site can be seen in Figure 3-1 along with all the soil properties developed from previous field testing (Rollins et al, 2003). The soil profile generally consists of stiff clay with two thin sand layers to a depth of 4.09 m which is the depth range which has the greatest effect on the lateral pile response. The water table was located at a depth of approximately 1.07 m during the time of the testing. The piles extended through an underlying soft clay layer and into a stiffer clay layer below. A picture of the site prior to construction of the pile caps is provided in Figure 3-2.

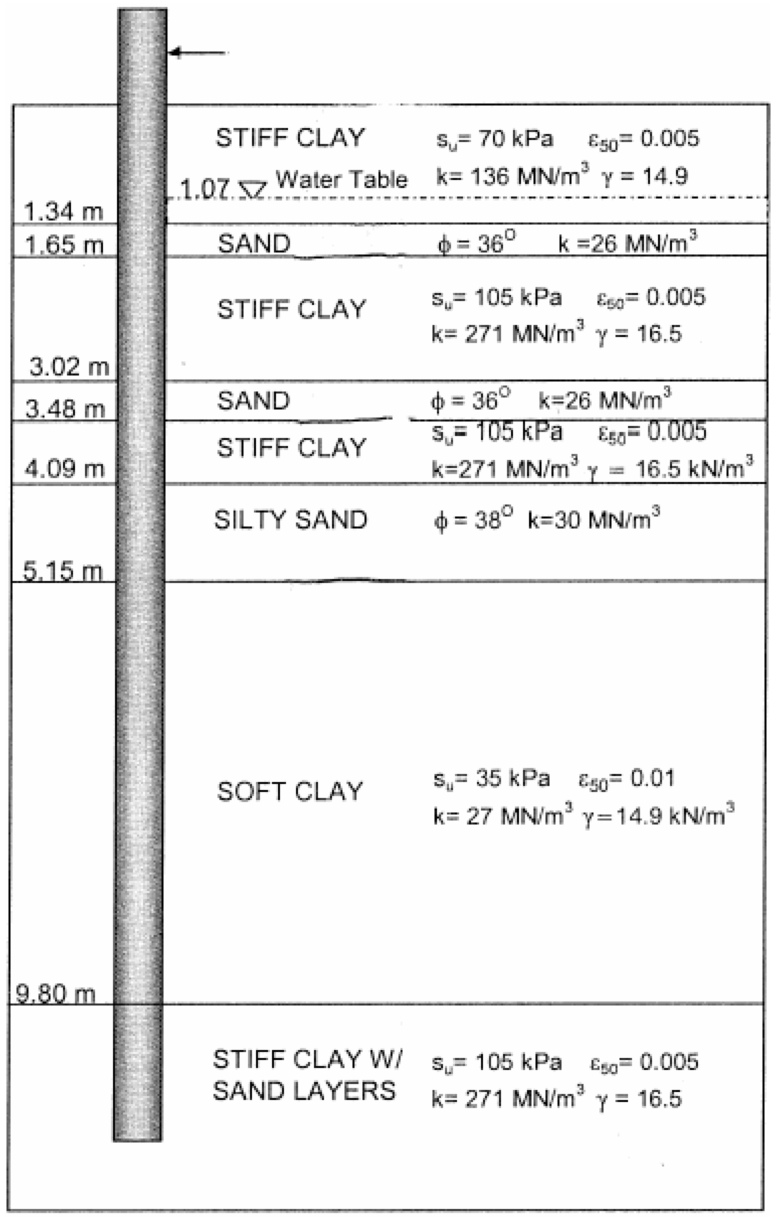


Figure 3-1 Soil Profile for the South Temple, Salt Lake City Test Site.

Figure 3-2 Photograph of the South Temple, Salt Lake City Test Site.

3.3 Materials

The materials used in the construction of all four pile caps were consistent with what is typically used in the field, that is: concrete with a 4,000 psi compressive strength and rebar with a yield strength of 60,000 psi. The driven piles were of steel with a modulus of elasticity of 29,000 ksi and yield stress of 57,000 psi.

3.4 Pile and Cap Description

All of the tests conducted consisted of a 6 ½ foot long concrete pile cap encompassing two circular steel pipe piles driven to a depth of 40 feet and spaced at 3

½ feet on centers. Each pile had an inside diameter of 12 inches with a 3/8 inch wall thickness. All pile caps were 3 feet wide and 3 feet tall and reinforcing grids of #7 bars spaced at 6 inches on centers in the longitudinal and transverse directions both top and bottom with a minimum 3 inches of clear cover on the top and 3 inches on the bottom. Figure 3-4 through Figure 3-7 show the piles and caps. Small holes were cut in the piles so that the longitudinal bars from the bottom reinforcement grid could extend through the piles; however, the transverse bars were cut off to prevent an excessive amount of holes in the piles. Figure 3-3 is an isometric view of the piles and cap. This drawing shows an embedment length of 12 inches which varies with each test.

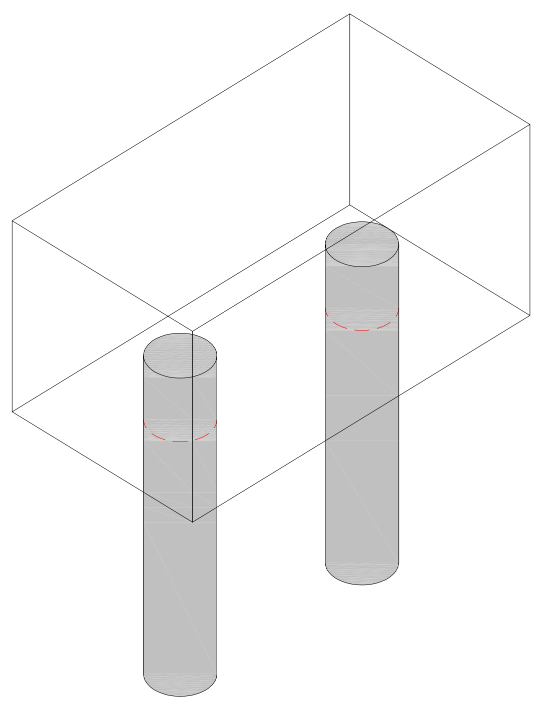


Figure 3-3 Isometric view of typical pile cap configuration.

Figure 3-4 Photograph of the South Temple, Salt Lake City Test Site.

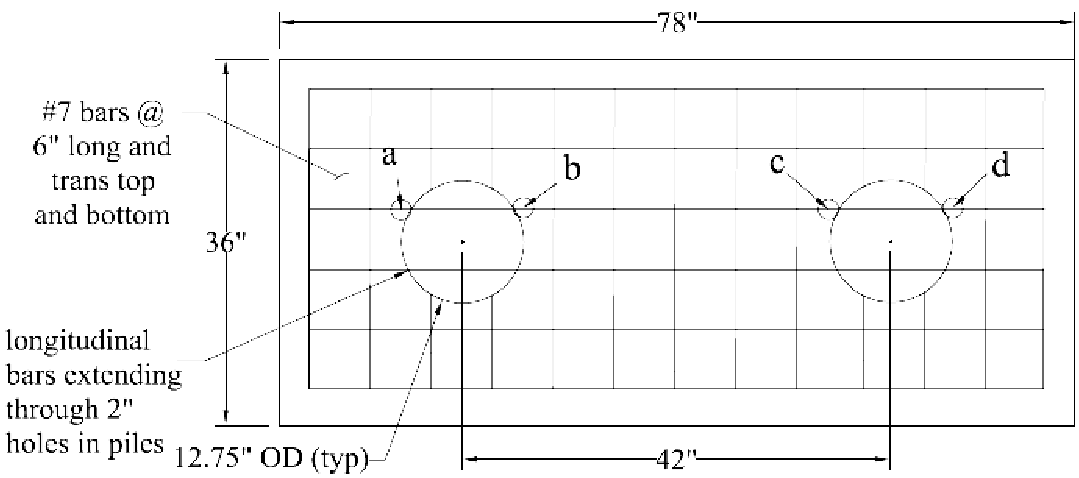
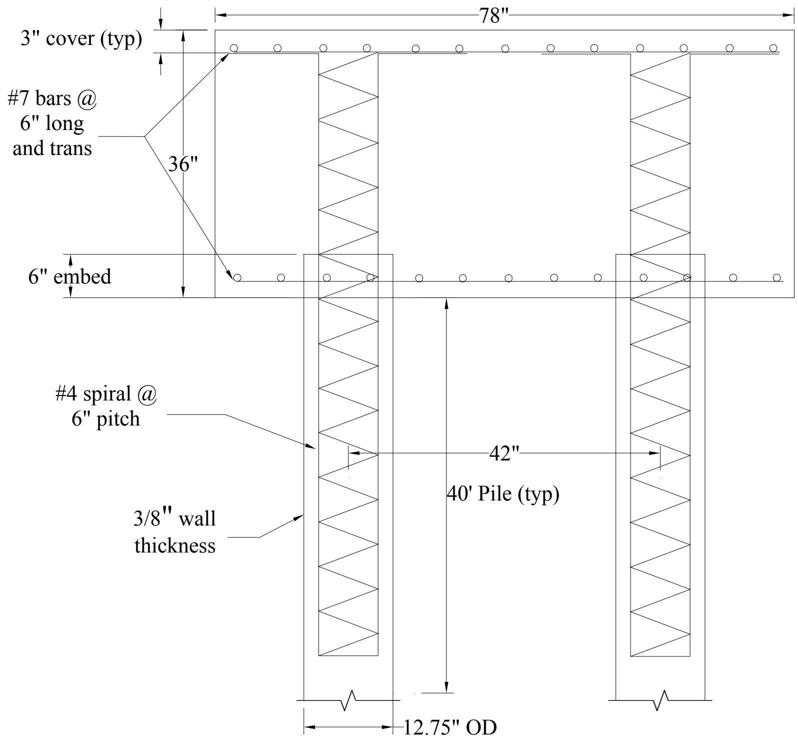


Figure 3-5 Pile Cap plan view dimensions (typical all caps).



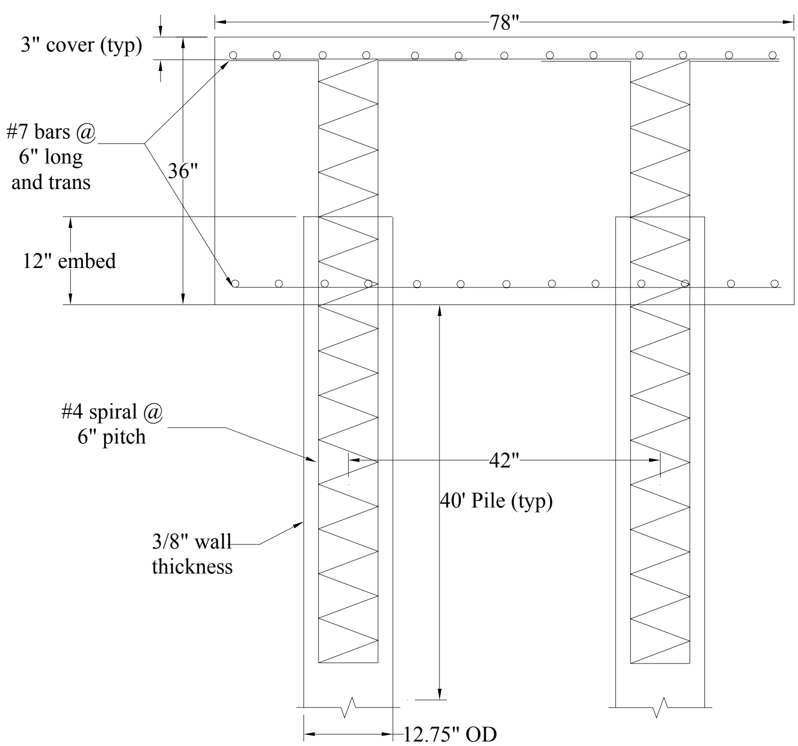


Figure 3-6 Dimensions for Test Cap 1 (above) and Test Cap 2 (below).

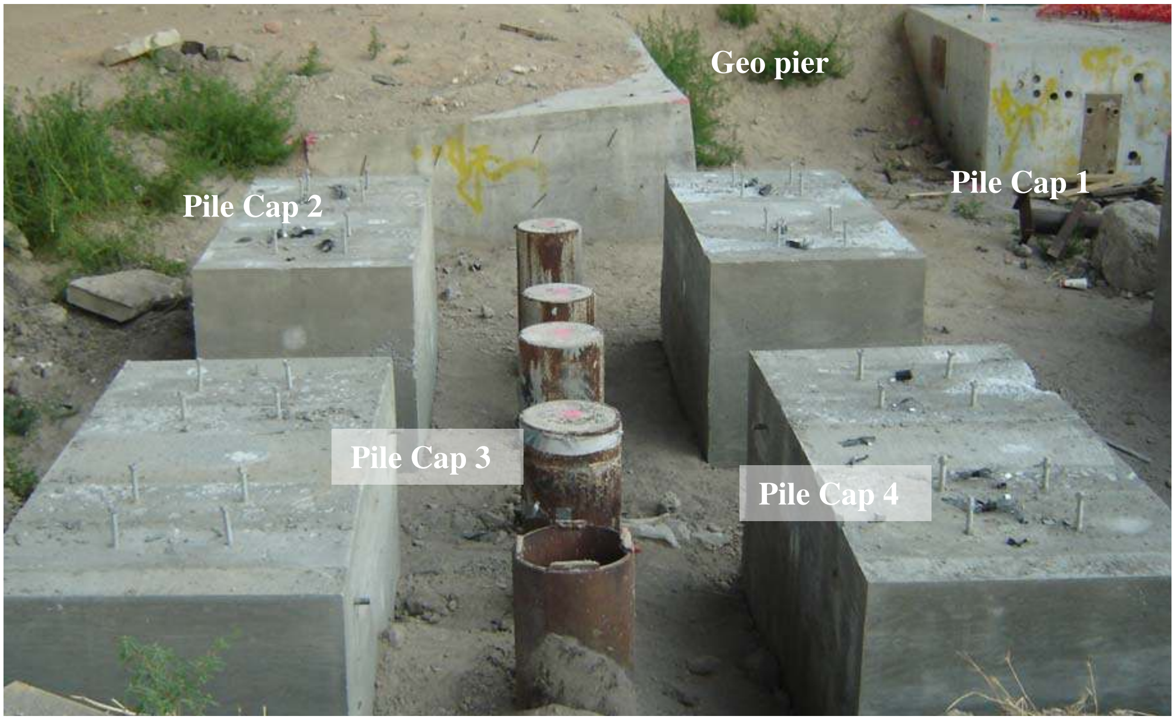


Figure 3-7 Photograph of the South Temple, Salt Lake City Test Site.

3.5 Instrumentation

Electrical resistance type strain gauges (Texas Measurements Group type FLA-6-11) were installed on the reinforcing bars as well as on the piles. In order to properly install these strain gauges, each gauge location was thoroughly prepared by grinding, sanding, and cleaning a flat, smooth area on either the pile surface or reinforcing steel bars. Figure 3-5 shows the reinforcing grid in the longitudinal and transverse directions in a plan view. This is typical of all caps; also shown is the location of the strain gauges (a, b, c, and d) installed on the bottom grid which is also typical of all four caps. Strain gauges are represented as circles on the drawings and labeled with a letter corresponding to its respective location; this is consistent throughout this thesis.

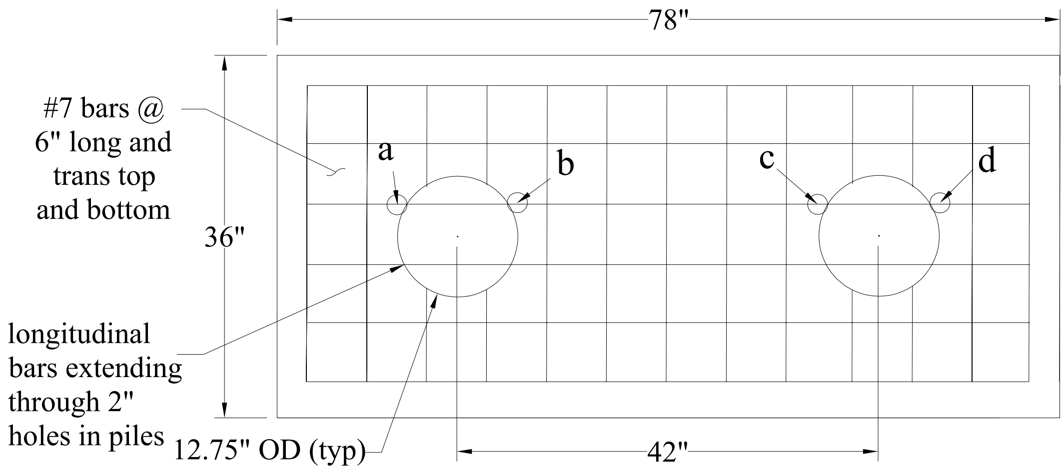


Figure 3-8 Pile-Cap and instrumentation plan view typical all caps.

To determine the displacement and rotation of each cap as a function of the applied force, six string potentiometers (string pots) were installed on the exterior of each cap to be tested. Two were placed on the top of the cap at a center location and spaced six feet apart so that they were approximately 3 inches from the front and back edges of the cap as shown in Figure 3-9. These two string potentiometers measuring displacements made it possible to calculate the pile cap rotation.

Along the front of the cap four additional string potentiometers were installed as shown in Figure 3-9. Three were placed at the elevation of the loading point, one foot above grade with one potentiometer at the center of the cap and two spaced at a distance of 3 inches from the edge of the cap on either side. The last string pot was located 21 inches directly above the center string potentiometer which placed it about

3 inches below the top of the cap. The displacement readings of three lower string potentiometers yielded an average displacement value and provided an indication of rotation of the cap about the vertical axis, while the difference between upper and lower displacements was used to calculate a rotation value about a horizontal axis and confirm the rotation obtained by the two string potentiometers that were placed on the top of the cap. Figure 3-10 shows a photograph of the setup of the string potentiometers. It is important to notice in the photograph that each string pot was connected to an independent reference frame that was not in contact with the pile cap, but was supported at a minimum distance of 10 to 15 feet from the test cap. This setup was the key to obtaining undisturbed displacement and rotation values.

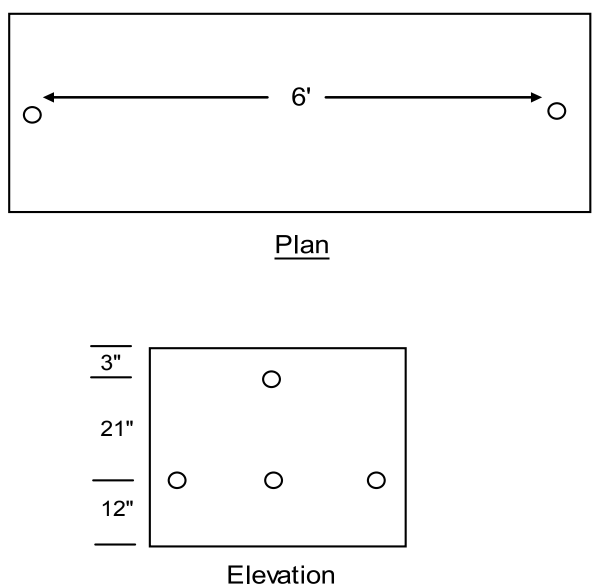


Figure 3-9 String potentiometer locations (typical all caps).

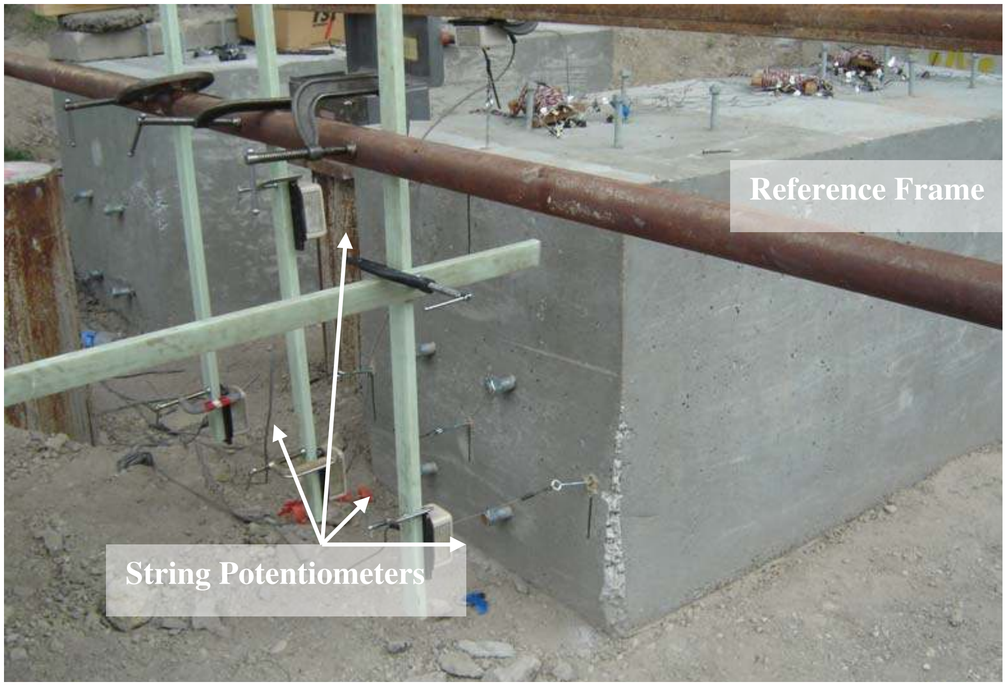


Figure 3-10 Photograph of string potentiometers setup (typical all caps).

3.5.1 Test Layout for Pile Cap 1

In the first test, an embedment length of 6 inches was provided along with a reinforcing bar connection detail consisting of a #4 spiral at a 6 inch pitch with 4 #6 longitudinal bars embedded 4 feet down into each pile and extending 33 inches above grade. Each vertical bar included a one foot section after a 90° bend which was tied to the top reinforcement grid. Both piles were filled with concrete. This is a standard UDOT connection detail and a cross section can be seen in Figure 3-11, with Figure 3-12 showing a cross section of the front elevation. A photograph of the connection provided for Pile Cap 1 is presented also in Figure 3-13. Twenty quarter bridge, resistance type strain gauges (Texas Measurements Group type FLA-6-11) were installed on test cap 1: four along the bottom reinforcing grid, six on each of the vertical connecting bars, and two on each of the piles at grade, their locations are shown in Figure 3-11. Despite preparations for protecting the gauges prior to pouring the concrete, some of the gauges malfunctioned and did not provide useable data.

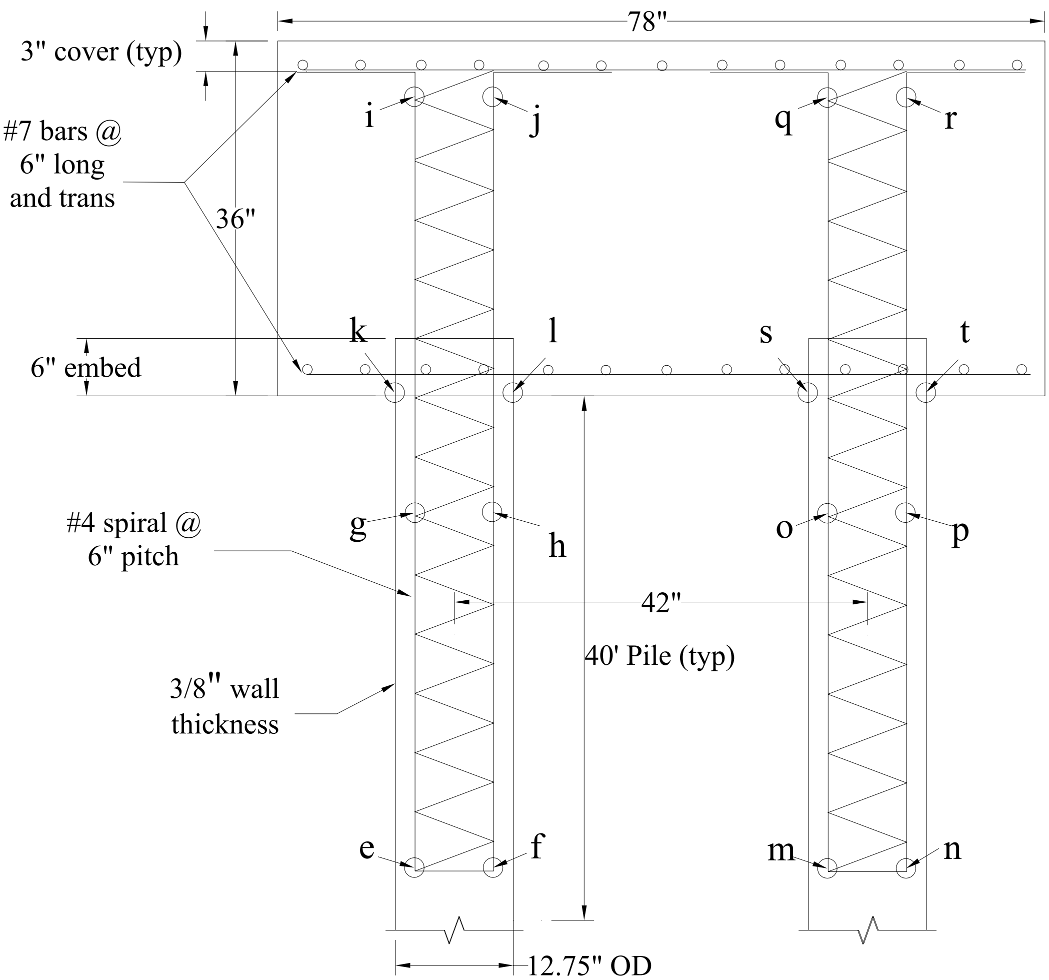


Figure 3-11 Pile Cap 1 with construction details and instrumentation layout.

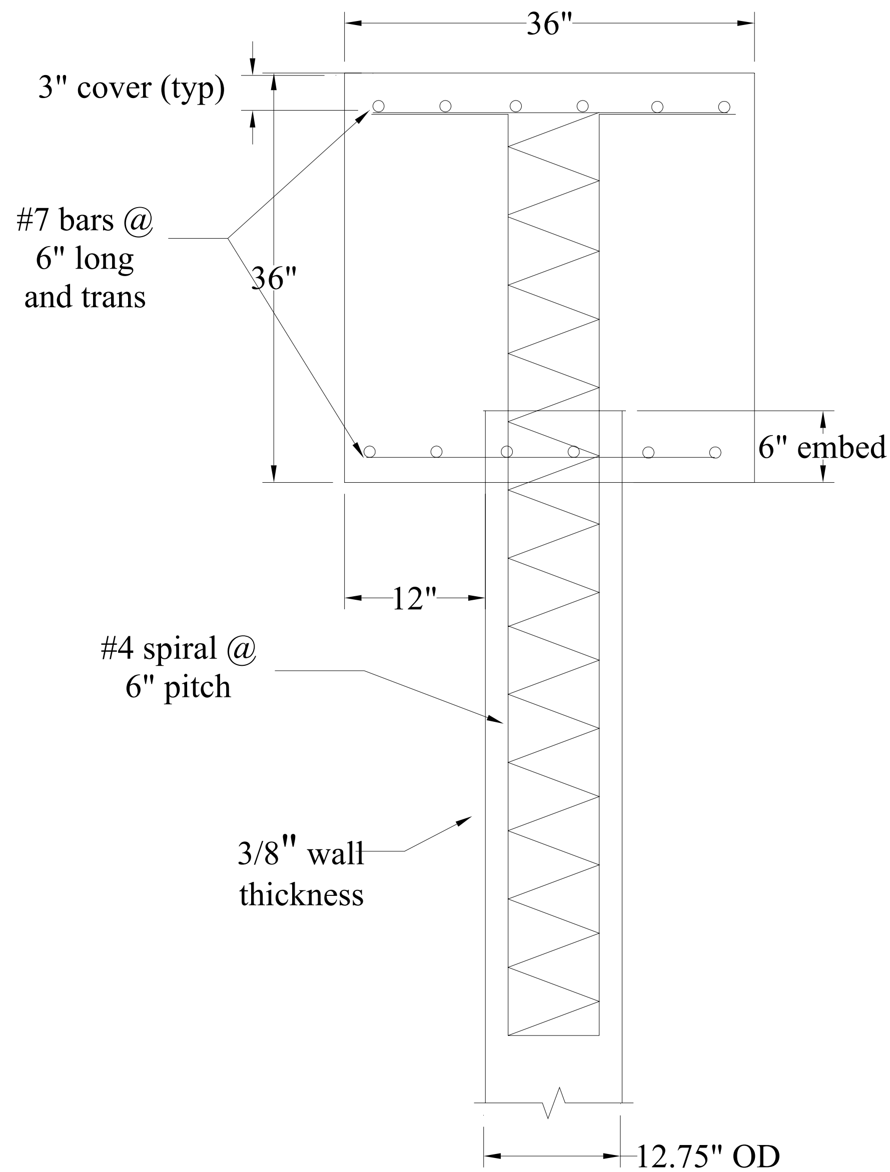


Figure 3-12 Front elevation view of Test Cap 1.

Figure 3-13 Photograph of Pile Cap 1 reinforcing.

Pile Cap 1 was approximately 8 feet from a large Geo pier cap and it was therefore convenient to use the Geo pier cap as a reaction for applying the load. As shown in the photo in Figure 3-14 a swivel head was attached to the back face of the pile cap with four 1 inch diameter cast-in-place all thread bolts embedded 5 inches into the cap and tied to two vertically placed rebar that were tied to the bottom and top reinforcing grids. The swivel head was then bolted to a 300 kip load cell which was in turn bolted to the hydraulic ram. The hydraulic ram was bolted to a circular steel spacer that was then bolted to the Geo pier cap. Since the center of the pile cap was slightly off the edge of the Geo pier cap, two angle pieces had to be attached to the Geo pier cap to completely support the hydraulic ram as it connected to the cap. All of these connections were designed so that a load of 150 kips could be applied without causing distress to any of the elements.

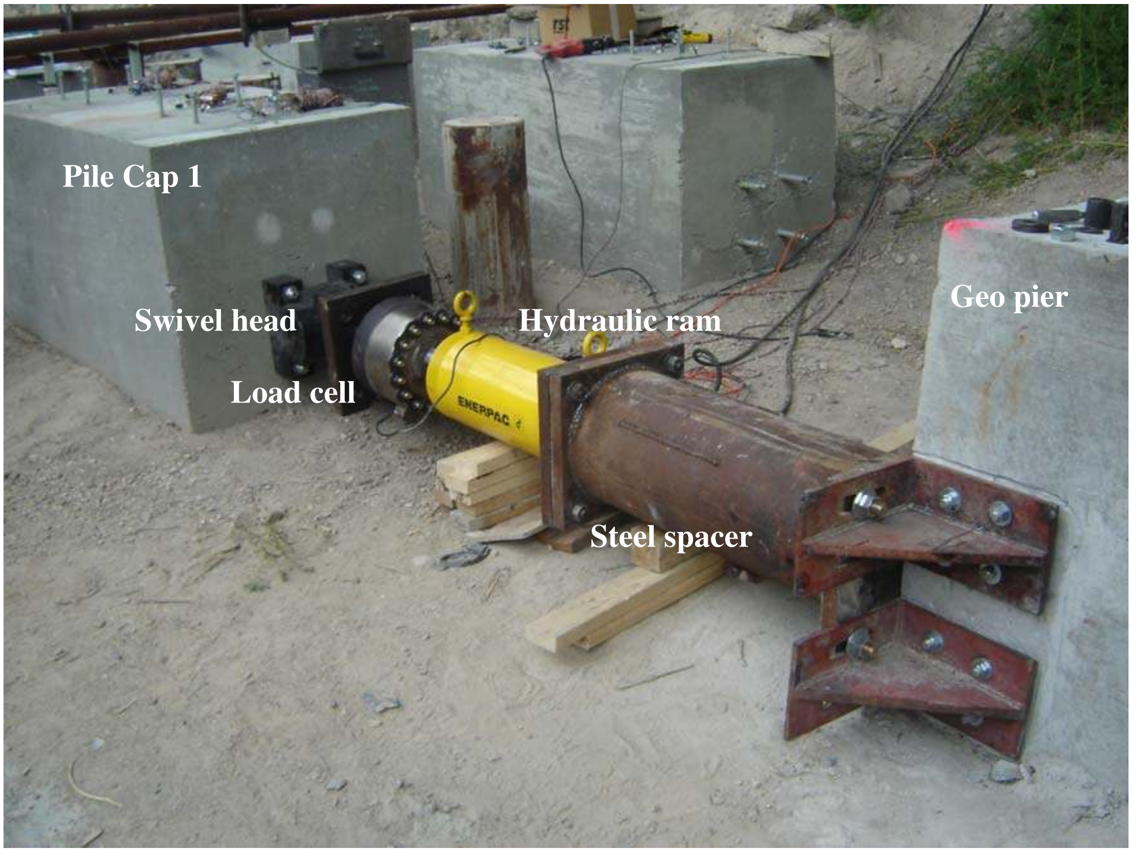


Figure 3-14 Photograph of test set-up for Pile Cap 1.

3.5.2 Test Layout for Pile Cap 2

As shown in Figure 3-15, the connection detail for pile cap 2 was essentially the same as that for pile cap 1, except that the embedment length of the steel pipe pile was increased from 6 inches to 12 inches. Also shown in Figure 3-15 is the location of strain gauges, note these are the same as with Pile Cap 1. Both piles were also filled with concrete.

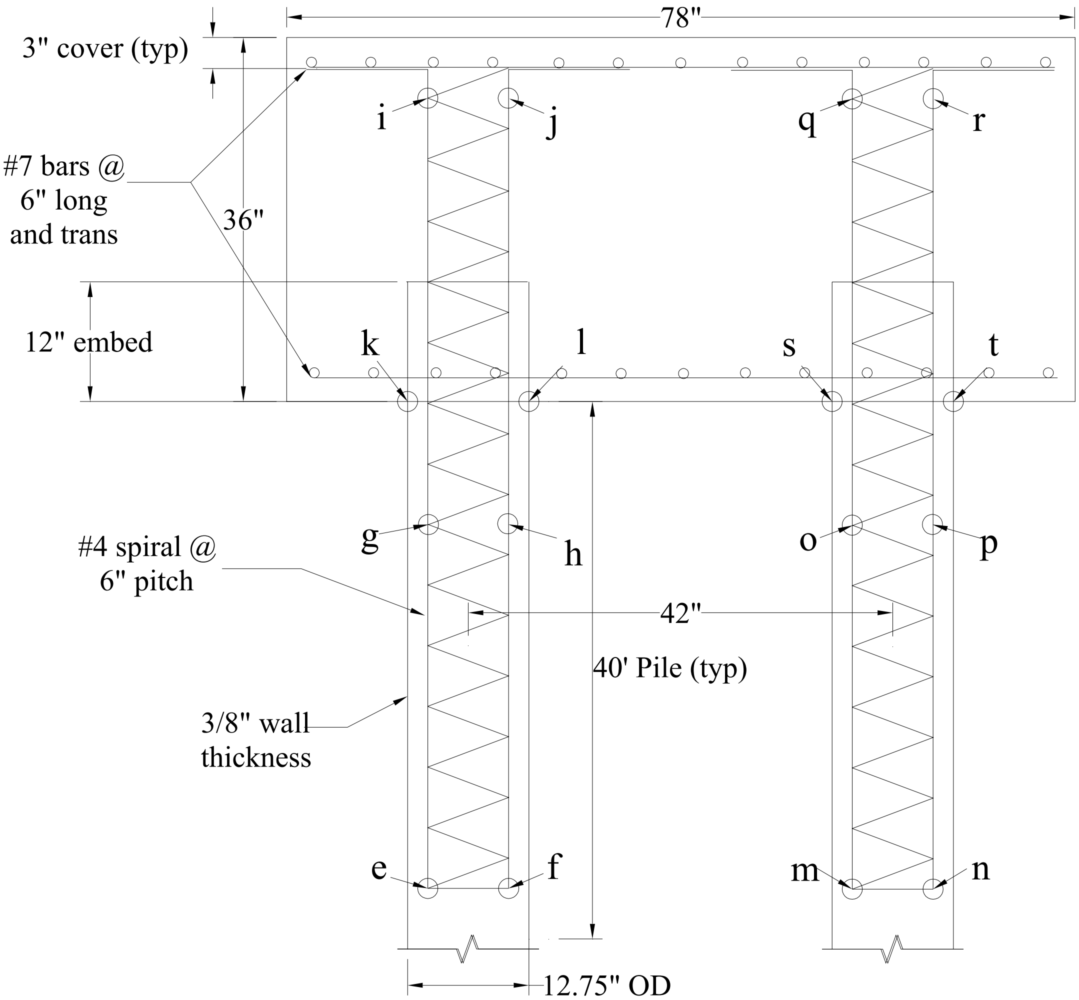


Figure 3-15 Pile Cap 2 with construction details and instrumentation layout.

Figure 3-16 shows a photograph of the test setup for Test Cap 2, the same connections were utilized and the Geo pier (not pictured) was used again as a fixed base at which to counter the applied force.

Figure 3-16 Photograph of test set-up for Pile Cap 2.

3.5.3 Test Layout for Pile Cap 3

The third test also provided a 12 inch embedment length; however no reinforcing cage connection detail was provided (refer to Figure 3-17). The piles were capped off with a metal plate and remained hollow as requested by the Oregon Department of Transportation to simulate a typical detail used in Oregon. Due to a lack of reinforcing detail the location of strain gauges was limited and only eight were used: four along the bottom reinforcing grid as with all the caps and four on the piles as shown in Figure 3-17.

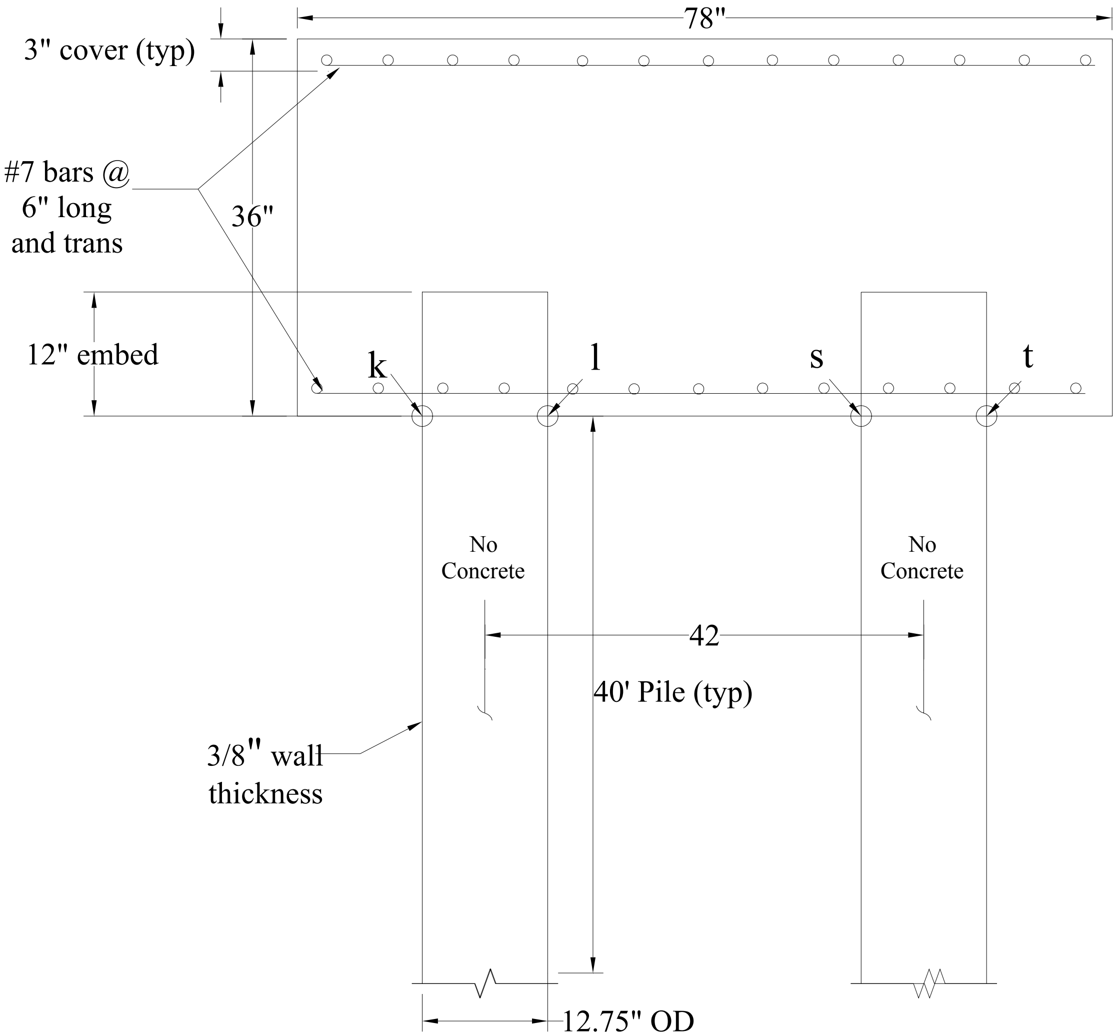


Figure 3-17 Pile Cap 3 with construction details and instrumentation layout.

Figure 3-18 provides a photograph of the pile cap, hydraulic ram set-up and reference frame during the test on Pile Cap 3. The Geo pier cap was again used to provide the reaction for the load test by placing a steel strut between Pile Cap 2 and the Geo pier Cap. Figure 3-19 shows a closer view of the test setup for Pile Cap 3 including the positions and anchoring used for the string potentiometers. A more compact swivel head was used due to the space constraints between the pile caps.

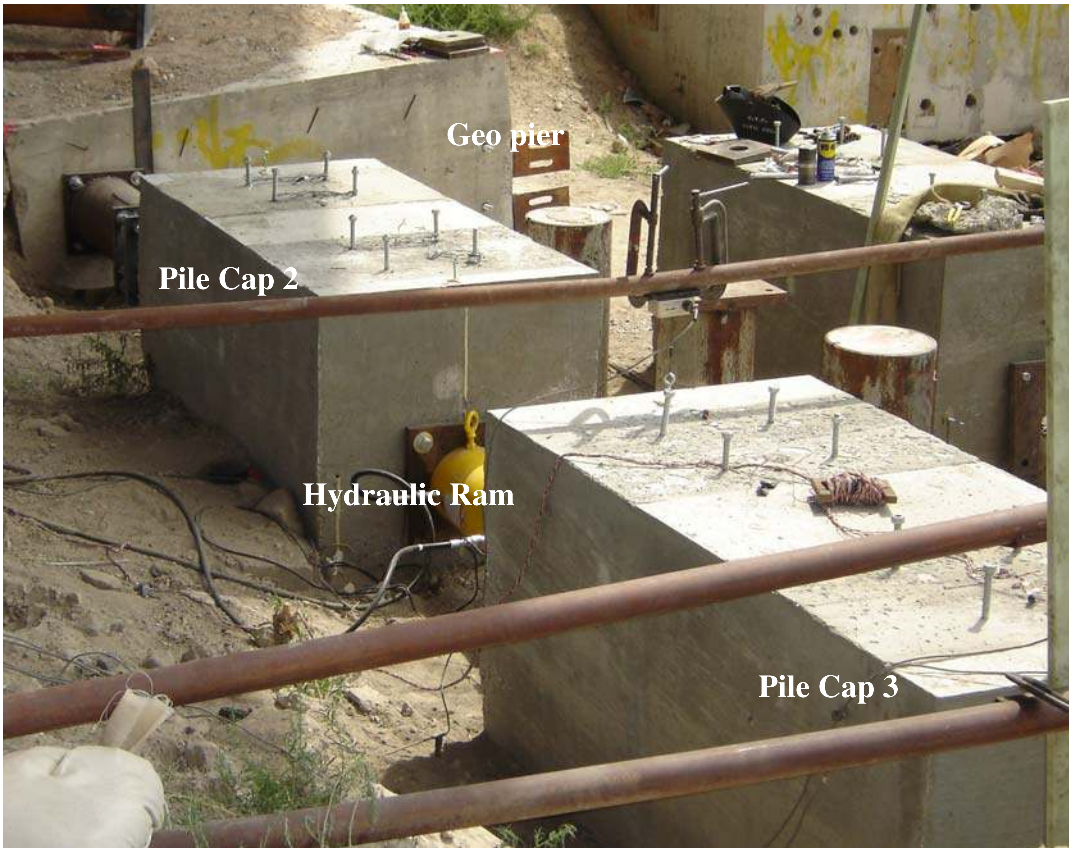


Figure 3-18 Photograph of equipment arrangement for test on Pile Cap 3.

Figure 3-19 Side view of test setup prior to loading Pile Cap 3.

3.5.4 Test Layout for Pile Cap 4

The geometry of the Pile Cap 4 is shown in Figure 3-21; a 24 inch pile embedment length was provided but no reinforcing cage connection detail was included. However, both piles were filled with concrete in contrast to Pile Cap 3 where the piles were left hollow. Since the rear pile had been previously filled with concrete strain gauges were not able to be installed. Two #6 rebar were placed in the front pile with six strain gauges attached as shown in Figure 3-21. With the increase in embedment length an additional 4 strain gauges were installed on the tops of the piles as shown in Figure 3-21.

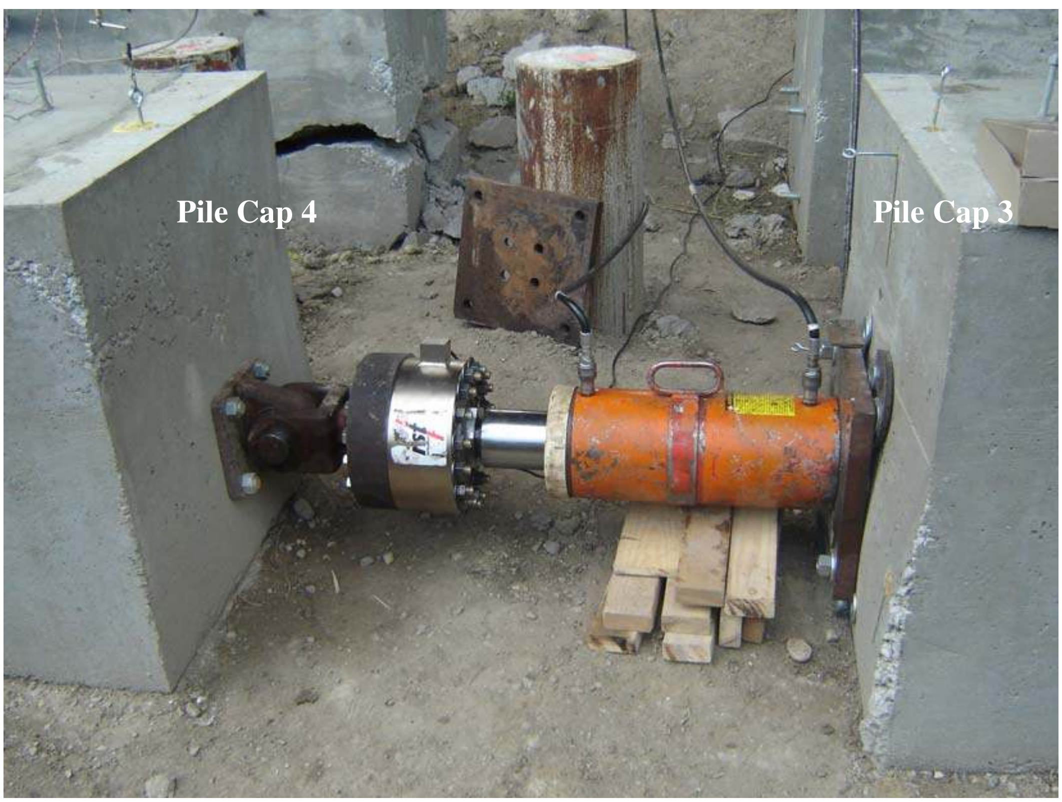


Figure 3-20 Photograph of test layout for test on Pile Cap 4.

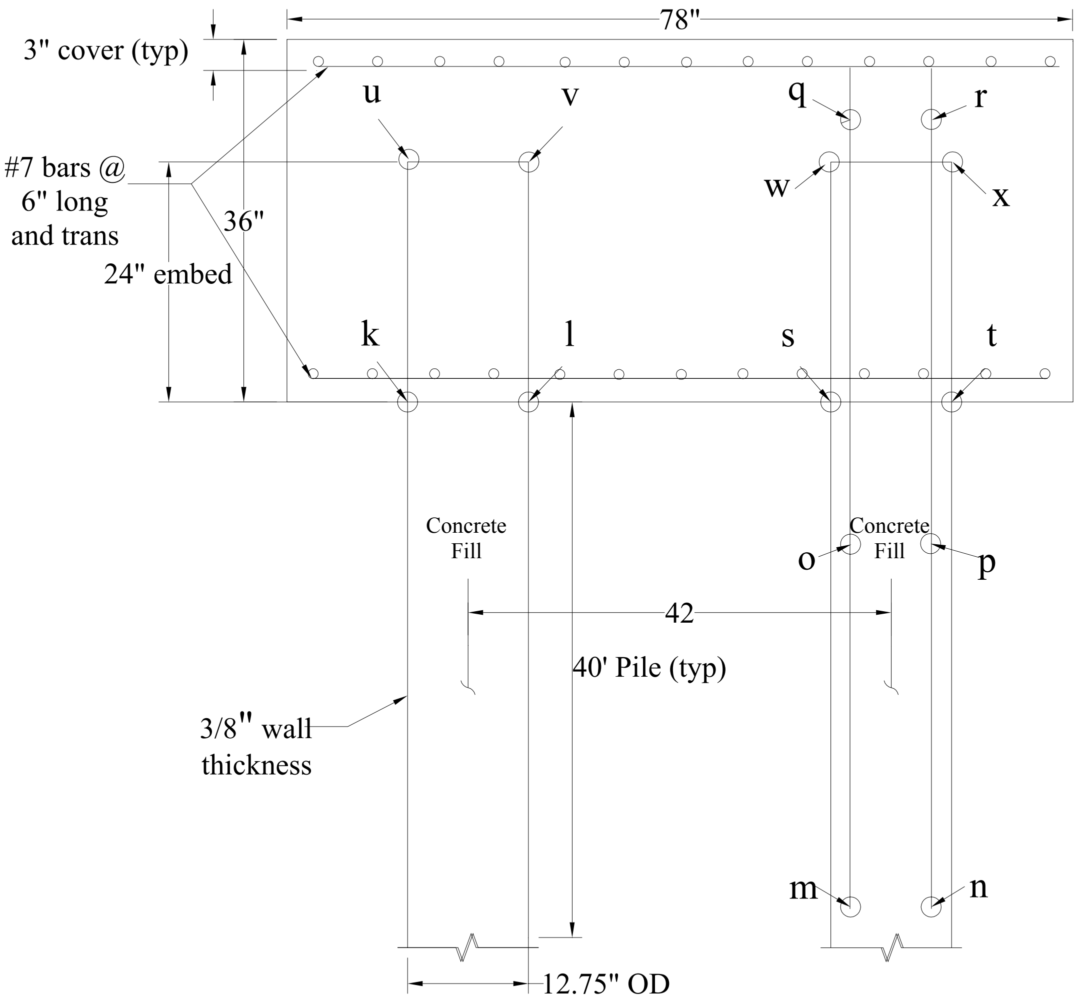


Figure 3-21 Pile Cap 4 with construction details and instrumentation layout.

Figure 3-20 shows the loading arrangement for the test of Pile Cap 4. A different hydraulic jack was used, and a strut was placed between Pile Cap 1 and the Geo pier cap to provide the reaction for the test.

4 ANALYTICAL STUDY

4.1 Introduction

Hand calculations and computer modeling were used to better understand how the pile caps would perform under lateral loads. Hand calculations were used to evaluate the potential for failure of individual elements and the computer models were used to explain the configuration as a system. Multiple failure scenarios were developed and their respective capacities determined by either hand calculations or computer modeling. In some case results were available from both methods and could be compared. There were two computer modeling programs available for calculations; LPILE 4.0, and GROUP 4.0. LPILE analyzes a single pile with user-defined soil and pile parameters. GROUP analyzes a group of piles with their respective soil and pile properties. Both programs account for group interaction effects as well as the pile head boundary conditions. Neither program considers the size, placement, or strength characteristics of the cap or the embedment length of the pile.

There were four major areas of concern: failure in the pile, failure in the cap, failure in the surrounding soil, and failure in the connection between the cap and pile. It was intended that failure would occur in the connection; therefore the pile and cap details were designed to both fit the criteria specified by UDOT and Oregon Department of Transportation as well as allow the failure modes to occur in the connection. It was predicted that even though the pile caps were to be laterally loaded that there would be large tensile forces acting throughout the pile and cap as well as large moment forces. It was therefore necessary to estimate multiple failure scenarios which will be discussed in this chapter.

4.2 Failure in the Piles

Generally, all the piles had the same material properties and geometries. The only variance was test piles for Pile Cap 3 that remained hollow while the test piles for the other pile caps were filled with concrete. Areas of concern regarding failure in the piles alone were that of excessive moments; this being the most common type of failure from testing conducted at Montana State University.

The shear strength of a hollow pile was estimated to be approximately 484 kips therefore similar calculations with concrete and/or rebar were not necessary.

According to analyses using LPILE, the hollow pile would have a 3,100 kip-in moment capacity while the concrete filled pile would have over a 3,500 kip-in moment capacity. Values obtained from equation 2-3 showed little variation with these values. GROUP estimated that the largest moment would occur at grade on the front pile, and would not exceed 2,000 kip-in for a lateral load of 130 kips. GROUP accounts for rotation effects due to the pile geometry and loading. LPILE, on the other hand, estimated that with a lateral load of 130 kips the moment would exceed

3,500 kip-in assuming that the pile was in a fixed-head condition. Although filling the piles with concrete only increases its moment capacity by 13% it is still recommended that piles be filled with concrete to delay local buckling.

4.3 Failure in the Cap

The caps themselves are also subject to moments as well as tension, compression and shear forces. Calculations using equation 4-1 estimated the cap moment capacity to be approximately 6,000 in-kips which greatly exceeded the moment to be applied. The one way shear strength of Pile Caps 1, 2, and 4 were also predicted to exceed the stresses applied during loading and to not be a concern, the one way shear strength equation is presented in equation 4-2.

$$M_u = .9 A_s f_y \left(d - \frac{A_s f_y}{1.7 f_c^{'} b} \right) \tag{4-1}$$

$$V_n = 2\phi \sqrt{f_c^{'}} A_c \tag{4-2}$$

However, questions remained as to how Pile Cap 3 would respond with the applied force acting in a direct line with the connection and with no reinforcement to hold the cap to the pile. GROUP estimated that tensile forces within the cap to reach 80 kips. The tensile capacity of the pile cap was estimated to be 192 psi by equation 4-3. In this equation the moment (M) was taken as 280 ft-kips, I as 22.8 ft^4 and y as 2.25 feet. The moment was determined conservatively by multiplying 80 kips by the pile spacing of 3.5 feet, the moment of inertia (I) was determined by considering the concrete that was in direct assistance to resist the tensile forces (3 feet wide and 4.5 feet long), and y was half of the 4.5 foot long section. Figure 4-1 is a diagram showing the forces and assumptions made in these calculations. These values should produce a conservative estimate for the tensile stress. Since this stress is considerably lower than the 400 psi tensile strength of the concrete it is expected that the cap will not fail in direct tension. A more likely scenario would be a combination of both shear and tension. For members under combined axial and shear force loading, ACI Code modifies the ultimate shear force equation 4-2 as shown in equation 4-4. With N_u equal to a negative 80 kips and A_c equal to 13.5 ft^2, equation 4-4 yields a shear strength of 169 kips; which is also below the force to be applied.

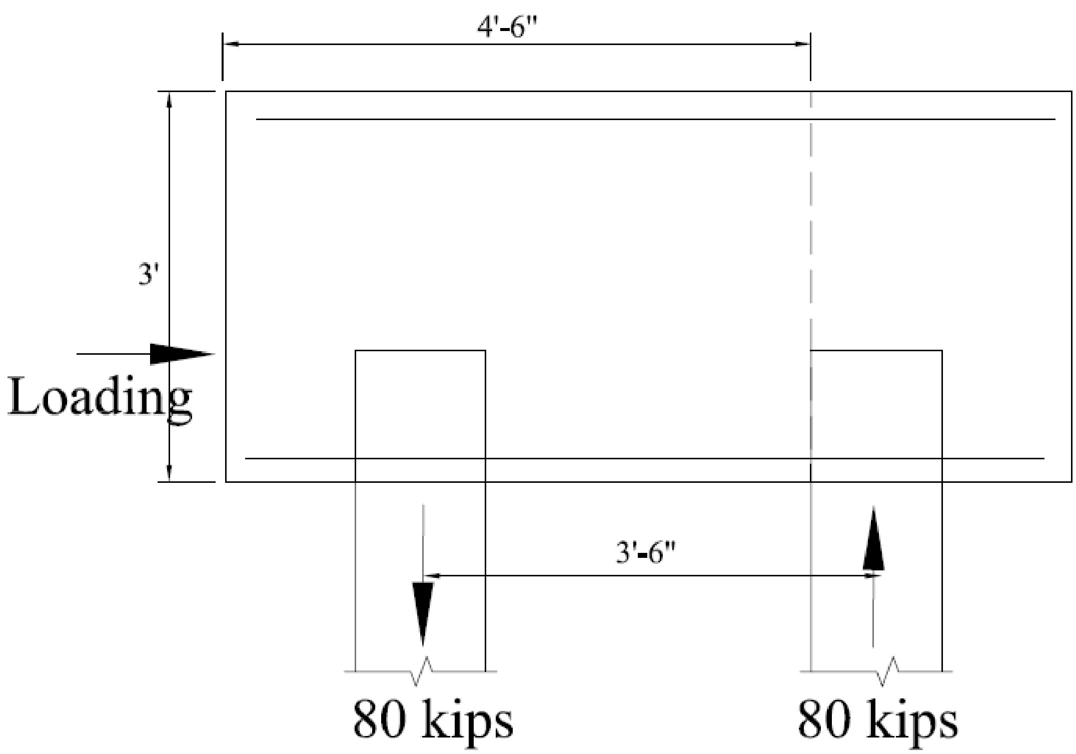


Figure 4-1 Tensile failure analysis diagram.

$$\sigma = \frac{My}{I} \quad \textbf{(4-3)}$$

$$V_n = 2\phi(1 + \frac{N_u}{500A_c})\sqrt{f_c^{'}}A_c \quad \textbf{(4-4)}$$

Although not supported on the opposite face as load is applied, the pile cap most closely resembles a deep beam. If modeled as a deep beam there is inadequate reinforcement to resist the large tensile and shear forces that may develop and a one-way shear failure is possible. There is a large amount of steel located within the cap, yet this steel including the piles are not in locations to provide direct assistance to resist a one-way shear failure.

Cracks in deep beams have been observed to occur at stresses somewhere between one-third to one-half of the ultimate strength (MacGregor and Wight 2005). To fit the criteria of a deep beam it can be assumed that the front pile is the location of the support and therefore the area of concern is only 4.5 feet long and 3 feet wide which yields a one-way shear capacity of 184 kips; one-third to one-half of this value is less then the load to be applied, and one way shear is of concern.

4.4 Failure in the Surrounding Soil

The computer modeling program GROUP proved invaluable in analyzing each test. By inputting the soil profile and each layers respective thickness and strength properties an estimation of the soil reaction vs. length along the piles was obtained. The soil profile and properties used in analyses are presented in Figure 3-1. When piles are driven at relatively close spacing the shear zones for adjacent piles overlap reducing the lateral resistance. Such group interaction effects are often accounted for using p–multipliers to reduce the soil resistance of p value. Using relationships developed by Rollins et al (2006) p-multipliers were estimated to be 0.82 and 0.61 for the front and trailing row piles, respectively. The unit side resistance along the length of the pile was estimated based on the undrained shear strength in the clay or the penetration resistance in the sand. Group analyses indicated that the trailing row pile would begin to pull-out of the ground when the lateral force reached about 80 kips. At a load of about 130 kips the pile cap would deflect significantly and the pile cap would have essentially failed at that point. Once a pile has displaced vertically more than 0.1 inch the majority of side friction is lost and additional loading would cause a magnification of both deflection and rotation. This appears to be the governing failure mode for piles 1, 2, and 4.

4.5 Failure in the Connection

It was desired that failure in the connection would occur prior to any other type of failure such that a comparison between all four of the tested connections would be possible. There were also multiple types of possible failures within the connections to be considered.

4.5.1 Tensile Failure of the Reinforcement

The tensile capacity, T, of the reinforcement is given by the equation below.

$$T = A_s f_y \tag{4-5}$$

The connection design, consisting of 4 #6 bars with a yield strength of 60 ksi would be able to resist over 106 kips of tensile force. As shown previously, the pile would pull out of the ground at an axial load of 80 to 90 kips and therefore the reinforcement design was considered adequate.

4.5.2 Reinforcement Pull-Out Failure in Pile

To develop the full tensile capacity of the reinforcing steel, the embedment length must be sufficient so that the bond strength between the concrete and the reinforcement is not exceeded. The required embedment length is known as the development length. Test Cap 1 and 2 were considered within this scope and Test Cap 3 and 4 while having no reinforcement connection were clearly not considered. According to ACI code provisions, the development length, l_d, is given by the equation below.

$$l_d = \frac{f_y d_b \alpha \beta \lambda}{25\sqrt{f_c^{'}}} \tag{4-6}$$

Utah DOT has specified a development length of 4 feet for #6 bars in their connection detail; however, calculations using the ACI equation were made requires only 29 inches of embedment. Therefore, 4 foot embedment depth specified by UDOT was used and considered more than sufficient.

4.5.3 Reinforcement Pull-Out of Cap

After determining that the reinforcement's embedment into the pile exceeded the required development length, it was then necessary to check the development length into the pile cap to ensure that this connection would also be adequate. Pile Caps 1 and 2 had both reinforced connections as well as a hook in the rebar as shown in its profile. The length of the hook, l_{dh}, provided was 12 inches and as shown in equation 4-7 below which is based on a bend of at least 12 bar diameters, only 14 inches of development length is required. In order to fully develop the reinforcement the bars must extend from the piles into the pile cap 14 inches and then hook at a 90 degree angle a distance of 12 bar diameters. The design specifications that Utah DOT provides for embedment into the cap is 27 inches from the top of the pile which excludes any type of hook, therefore once again the provided details are more than adequate.

$$l_{dh} = \frac{1200 d_b}{\sqrt{f_c^{'}}} \tag{4-7}$$

4.5.4 Concrete Pull-Out of the Pile

Another potential failure mechanism to be considered for Pile Caps 1 and 2 is if the tensile forces within the pile exceed the bond strength between the steel pipe and the concrete infill so that the reinforced concrete section pulls out of the pile. Using a bond strength of 45 psi between a steel pipe pile and concrete infill, this failure type was predicted not to be a concern provided monolithic pour of a minimum of 4.5 feet. The worst case to consider would be Test Cap 1 with only 6 inches embedment which still provides 6 inches on the exterior of the pile and 54 inches on the interior extending to the tip of the reinforcing bars. This concludes that before the concrete can be pulled out of the pile the reinforcing steel will yield.

4.5.5 Bond Strength between Exterior of Pile and Concrete

Pile Caps 3 and 4 did not have a reinforced connection detail and with such high axial loads to be considered it was necessary to calculate a possible slipping to occur between the exterior of the pile and the surrounding concrete. Pile Cap 4 had two differences compared to Pile Cap 3. First, the concrete in the pile and cap were poured monolithically providing added strength and second the embedment length was 24 inches which was twice as long as for Pile Cap 3. Using the same conservative value of 45 psi for the steel to concrete bond strength, the capacity of the interface for Pile Cap 4 was found to be 90 kips; this includes the bond strength around the perimeter of the pile. This load is near the piles side friction capacity though with the very low bond strength value of 45 psi used in the calculation this failure mode is not considered to be of high concern. These same calculations excluding the tensile strength since the piles will not be filled with concrete suggest that the interface capacity for Pile Cap 3 would be only be 50 kips. Therefore, failure at this interface could occur before the pile pulls out of the ground. However, these calculations did not account for the influence from the bottom reinforcing grid which includes the pile cap longitudinal bars extending into the piles through 2 inch holes; this is expected to provide additional resistance.

4.5.6 Bearing at Connection Interface

The calculations influenced by the embedment length were that of excessive bearing at the embedment interface. As shown in the literature review, the majority of pile cap tests showed extensive failure in this region. Figure 4-2 is presented to show how many of the tests previously conducted have failed in the connection area due to concrete crushing. Tests conducted in this study are different in that the pile cap was free to translate and rotate while laboratory tests, such as those conducted by Montana State University involved pile caps which were restrained against translation and could not rotate. Other steel to concrete connection tests performed by Marcakis and Mitchell (1980), and Mattock and Gaafar (1982) also fixed the embedment region while applying the force at some distance away from the connection such that a large moment could be developed at the interface. Mattock and Gaafar (1982) presented the equation below.

$$V_u = 54\sqrt{f_c^{'}}\left(\frac{b^{'}}{b}\right)^{.66} \beta_1 b L_e \left[\frac{.58-.22\beta_1}{.88+\dfrac{a}{L_e-c}}\right] \tag{4-8}$$

Using equations 2-1, 2-2, and 4-8 charts have been developed and are presented in Figure 4-3 and Figure 4-4 which compare the required embedment lengths as a function of concrete cracking. As shown in Figure 4-3 and Figure 4-4; Pile Cap 1 having only a 6 inch embedment length had a high possibility of failure in the connection interface area for applied loads greater than about 60 kips, while Pile Cap 2 and Pile Cap 3, both with 12 inches of embedment length also pose a threat of failure. However since the tests in this study involve a pile cap that is able to displace and rotate, the applicability of these equations are suspect.

Figure 4-2 Failure of pile caps tested at Montana State University.

As noted previously, the equations used to develop Figure 4-3 and Figure 4-4 were developed through a series of tests in which the embedded steel received the force while the concrete in which the steel was embedded remained fixed. This would cause a notable difference in the embedment length required to achieve full moment capacity of the desired connection. The problem in directly applying these equations is that in the current study the lateral force is applied at the elevation of the connection region rather than at a distance away. Therefore, it was necessary to use the computer modeling programs to determine moments as a function of depth below the pile cap produced by the applied lateral load. Using the location at which the moment was equal to zero and its corresponding shear force the values in the three equations were calculated.

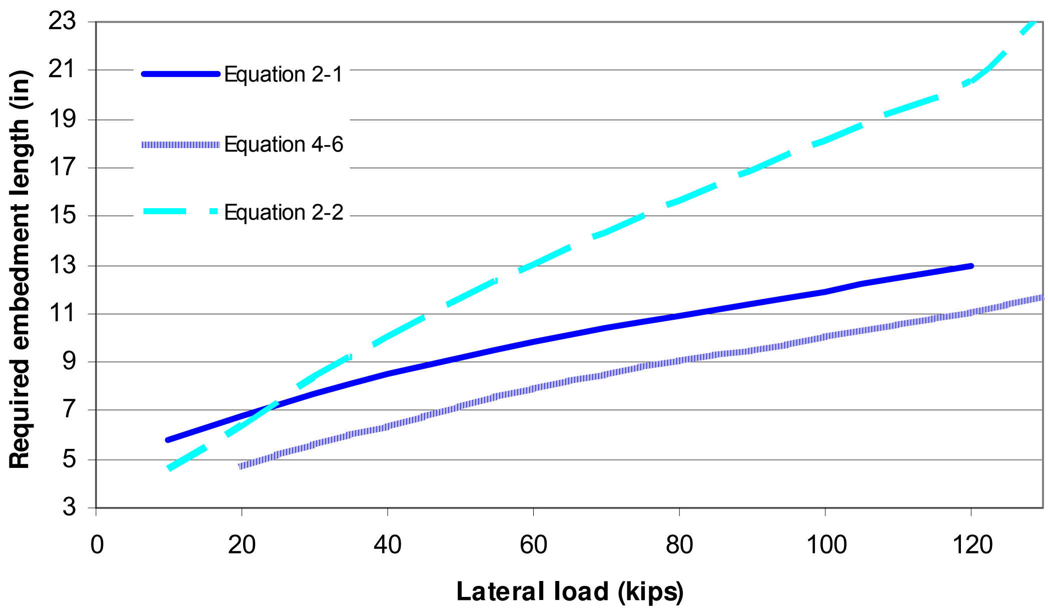


Figure 4-3 Required embedment using moment and load from LPILE.

4.6 Rotational Restraint

In general, it is desirable to determine accurately the boundary conditions of the connection in order to properly design and/or analyze the connection between the pile and the cap. In terms of stiffness, it is desirable to achieve a fixed head condition such that zero pile head rotation occurs, yet this is seldom achievable in practical cases. In contrast, a free-head or pinned connection which allows full pile head rotation is seldom seen in practice and assuming this boundary condition could result in a very costly over design. On the other hand, assuming a completely fixed condition when it is really not the case, could have the opposite effect which would lead to under design and hence a high potential for failure. As indicated in the literature review, Mokwa and Duncan (2003) developed a method to calculate the rotation spring stiffness of a pile head for pile caps with are intermediate between fixed-head and free-head boundary conditions. Using Mokwa and Duncan's' method, a value $K_{M\theta}$ of 90,859 kip-ft was determined and was used as an input in the computer modeling program GROUP. This value, as will be seen further on in this paper, was found to produce results which were very similar to a fixed head condition and also very accurate regarding deflection and rotation compared to the data observed while testing.

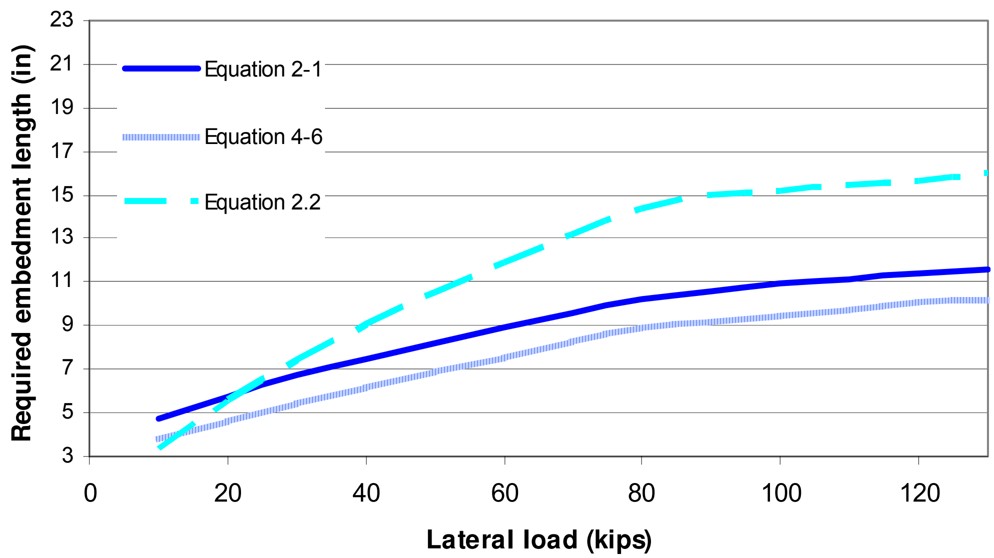


Figure 4-4 Required embedment using moment and load from GROUP.

4.7 Summary of Predictions

After conducting this analytical review, predictions have been made regarding each test. The connections in both Test Cap 1 and Test Cap 2 are expected to perform well with the system failing in pullout. Test Cap 3 is more of a challenge to predict with three potential failure modes governing: the bond strength between the back pile and concrete in the cap slips and the back pile is detached from the cap, the cap acts as a deep beam and shear failure occurs along the top of the piles, or the connection remains intact and the system fails in pullout as expected for Test Cap 1 and Test Cap 2. Test Cap 4 is also expected to fail in pullout with a small possibility that the bond strength between the back pile and concrete in the cap causes an excessive amount of slip.

5 Test Results

5.1 General Remarks

The majority of tests previously conducted have laboratory tests where most contingencies can be estimated and corrected quickly. In-situ testing is more challenging, yet can potentially produce more valuable data because the test conditions are closer to actual conditions. Each test conducted utilized a hydraulic ram to act as the lateral force which pushed the pile cap at a predetermined location one foot above grade. Five load cycles were applied at each deflection increment with slight variances that are noted. Three types of instrumentation were used for gathering data: string potentiometers to measure the amount of displacement, strain gauges to measure the amount of strain, and a load cell to determine the applied force. The tests lasted an average of 90 minutes due to the loading sequence as well as inspections of equipment and the pile cap.

5.2 Pile Cap Test Results

Test Caps 1, 2, and 4 all failed as expected with the piles losing their friction with the surrounding soil and the back pile being pulled out of the ground. Their respective connections appeared to be adequate and no sign of failure was noticed. In contrast, Test Cap 3 experienced connection failure while both piles remained in the ground with no noticeable movement. During the loading sequence of Test Cap 1 a shear zone in the surrounding soil was noticed while the other tests showed no sign of shear failure in the soil. The testing confirmed the hypothesis that either a proper reinforcement connection is needed or an embedment length long enough to act as a fixed head connection. A detailed summary of the results from each of the four tests is provided in the subsequent sections of this report. In addition, the results are compared with previous analyses to provide a better understanding of the significance of each test.

5.2.1 Test Cap 1

The connection detail for Test Cap 1 consisted of a pile embedded 6 inches into the cap along with a reinforcing cage which extended to the top of the cap. Full details and specs for pile cap 1 are provided in Chapter 3. It was expected that the connection design for this cap would perform well under the loading sequence. As shown in the load versus time plot shown in Figure 5-1, the load test was performed using nine load increments. At each increment, five cycles of load were applied and then reduced to zero load. Prior to loading to the next desired increment, the pile cap was brought back, as close to its initial position as possible without causing an excessive amount of tension on the connections. This general procedure was continued on all four tests. All of the charts presented in this paper will compare the observed test results with the estimated values predicted by computer analyses. These predicted values are the same on each chart presented; LPILE does not consider a pile head fixity condition while GROUP requires the user to input a fixity boundary condition as either pinned, fixed, or elastically restrained. The elastically restrained condition allows the user to input a spring constant, as will be shown in the current analysis this boundary condition is very close to the fixed head condition. Also; the observed values during testing were gathered from each initial and final load event of its corresponding cycle.

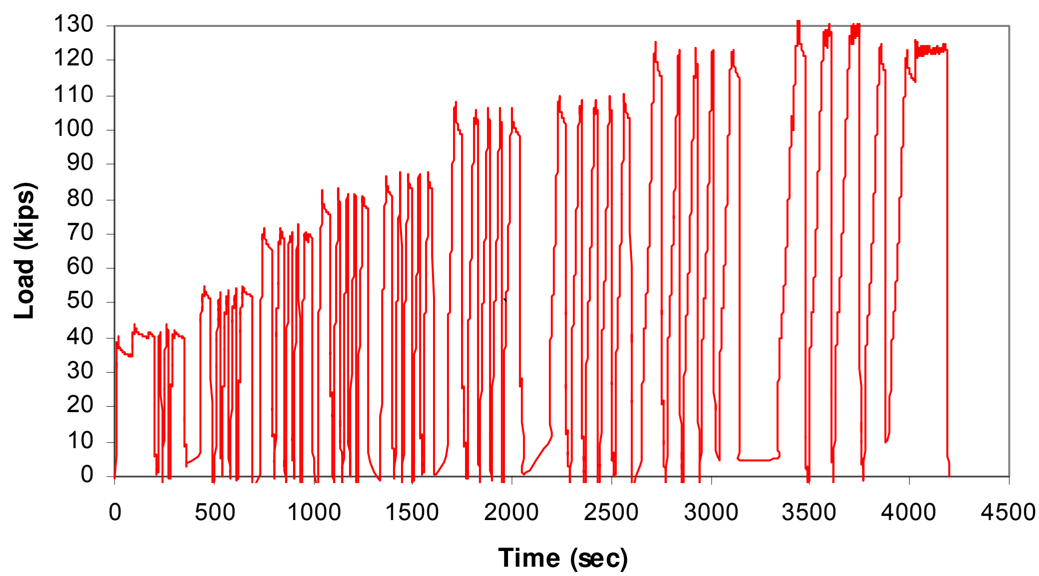


Figure 5-1 Test Cap 1 Loading Sequence.

In the first test as the pile cap was reloaded to a given load within a cycle an excessive amount of creep displacement was observed, this became more significant as the load increased causing the deflection during cycling to exceed the desired target deflection for the next load increment. Therefore, it was decided that the remaining tests should be loaded to an incremental displacement control value instead of a load control value. Figure 5-2 shows a plot of the complete load-deflection curve during the load testing. This significant creep is shown in Figure 5-2.

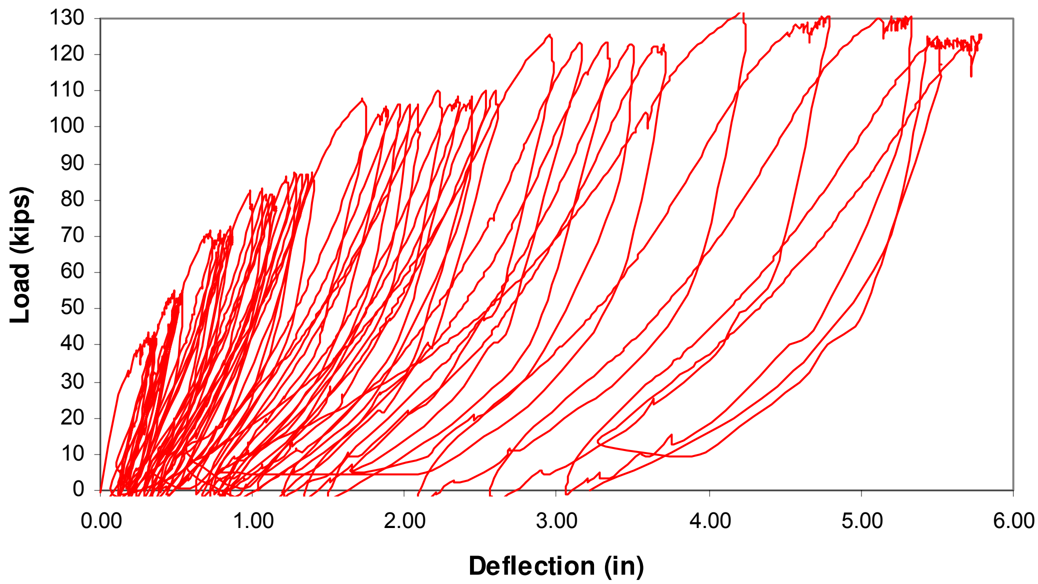


Figure 5-2 Cyclic Loading of Test Cap 1.

The peak load versus deflection curve for the first and fifth cycle of loading is presented in Figure 5-4 while the rotation versus load curve is plotted in Figure 5-5. The rotation was calculated using the deflection measured by the string potentiometers located strategically on both the top face and front face of the cap. Unfortunately, string potentiometer #11 malfunctioned, which was not noticed until all four tests were completed. Therefore, wherever string potentiometer #11 was installed, the corresponding data had to be discarded. During testing of Cap 1 this string potentiometer was positioned on the top face of the cap. Therefore, rotation (Θ) was computed using the deflection data on the front face of the cap using the equation below.

$$\theta = \arctan(\frac{X_1 - X_2}{L - \delta_v}) \quad \textbf{(5-1)}$$

In equation 5.1, $L - \delta_v$ is used rather than L to correct for string pot displacements caused by vertical translation. This is shown in the diagram below. The rotation versus load curve shown in Figure 5-5 includes data points of the observed rotations and the calculated values from GROUP.

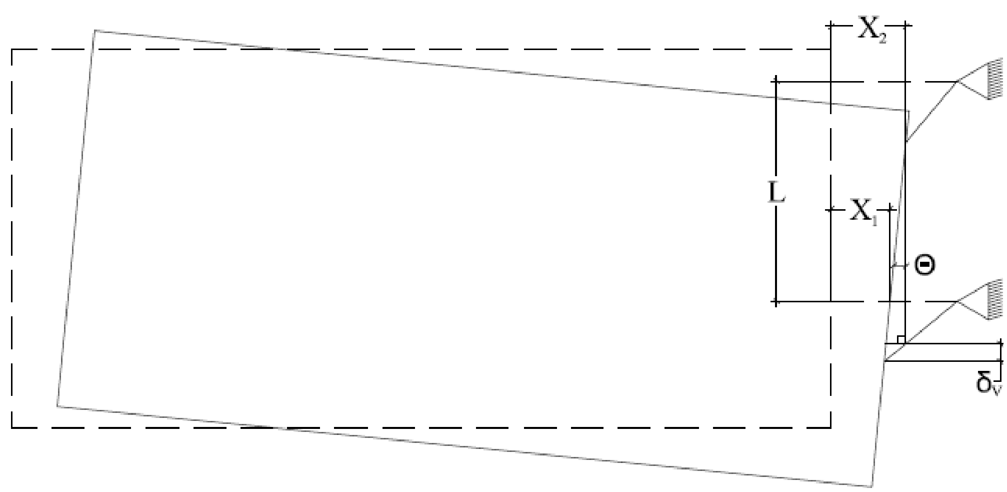


Figure 5-3 Pile cap rotation analysis.

Figure 5-4 includes 5 different data sets; fixed head, elastically restrained, and pinned which represent outputted values from GROUP for their respective boundary condition, the other two are the measured readings with their respective data points for the 1st and 5th load cycles. Both the load versus deflection and the rotation versus load curves remained approximately linear until a load of about 80 kips was reached and a shear crack was observed radiating outward at about a 45 degree angle from the back pile. At this point the uplift force on the back piles apparently began to exceed the side resistance between the pile and the soil. Soon after this, increased rotation and deflection were observed as shown in Figure 5-4 and Figure 5-5. The percent error in load for a given deflection for Test Cap 1 was typically less than about 5%. The discrepancy between the measured and computed load-deflection curves appears to increase at the higher load levels (>100 kips).

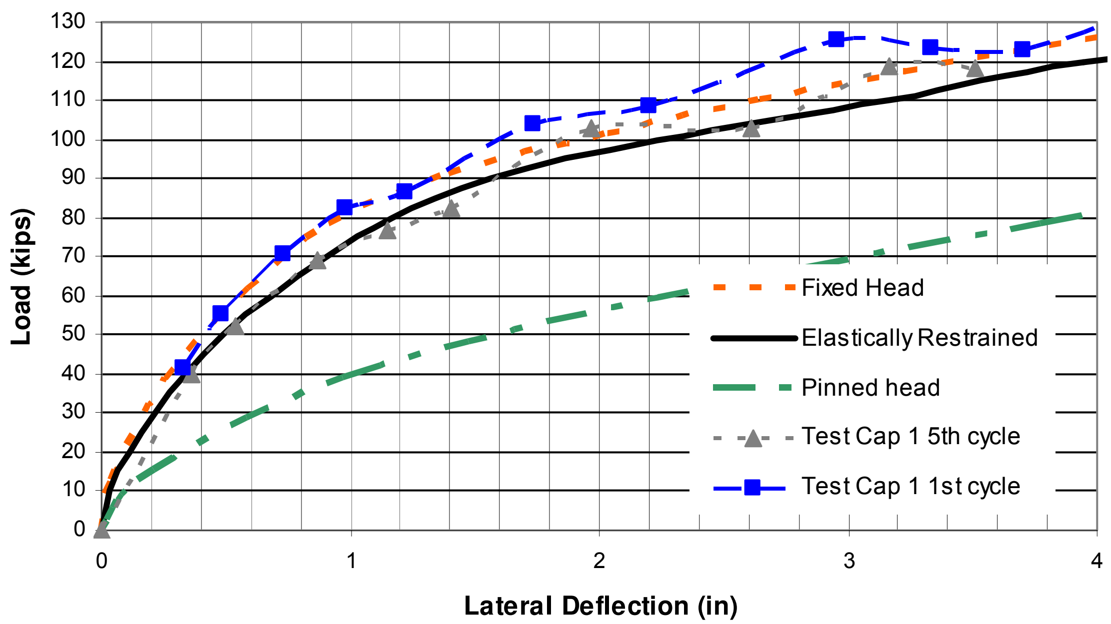


Figure 5-4 Test Cap 1 and GROUP deflection curves.

The load-rotation curve computed prior to testing using GROUP is also plotted in Figure 5-5 and it compares favorably with the measured curve. As GROUP predicted, pile cap rotation increased significantly at a lateral load of 80 kips as was observed in the experimental data. According to the strain gauge readings plotted in Figure 5-6 the tensile strain in the back pile also increased significantly as the lateral load increased to 100 kips. This behavior is also consistent with an increase in axial pile force which led to pile pullout. Although the strain level is far below the tensile capacity of the bars, Figure 5-6 confirms the need for some type of connection between the pile and cap. It explains that as the load is transferred the reinforcing bars pull the piles out of the ground. As will be shown later in this report, if a proper connection is not provided the pile cap rotates while the pile remains in the ground with little or no disturbance to the piles.

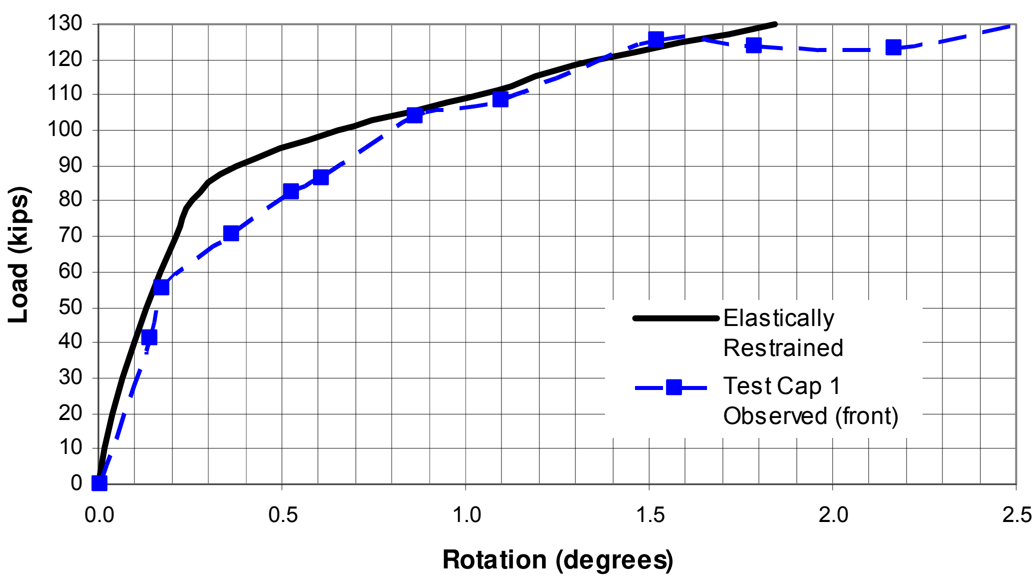


Figure 5-5 Pile Cap 1 observed vs. computed rotation.

While the pile cap and connection remained essentially elastic throughout the loading sequence, the failure was considered to fall under the category of excessive deflection. This was due to the piles pulling out of the ground. As shown in Figure 5-4 GROUP was used to produce load-deflection curves assuming three connection boundary head conditions; completely fixed, elastically restrained, and pinned. The value for the rotational stiffness in the elastically restrained case was calculated using equations developed from Mowka, and Duncan (2003) and was considered to be the most accurate for analysis. However, in this case, the load-deflection curve for the fixed-head condition were nearly identical to that for the elastically restrained condition. GROUP was observed to predict very accurately the test results with regards to rotation and deflection as shown in Figure 5-4 and Figure 5-5. It was thus concluded that for the testing performed that the connection detail was adequate.

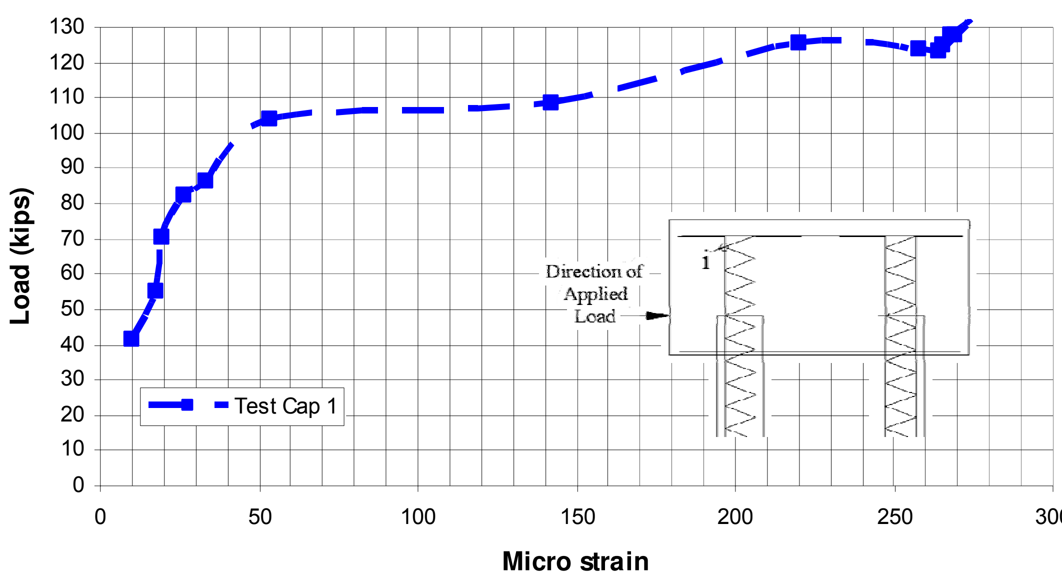


Figure 5-6 Observed micro strain at location i on vertical reinforcement.

Figure 5-7 Photograph of Test Cap 1 at failure.

5.2.2 Test Cap 2

The design of Test Cap 2 varied from Test Cap 1 only in the embedment length of the piles. The piles were embedded 12 inches into the cap rather than 6 inches as in Test Cap 1 (refer to chapter 3 for Test Cap 2 details and specs.) It was expected that Test Cap 2 might experience smaller deflections and rotations. However this was not the case. Slightly larger deflections were observed relative to Test Cap 1 yet they were within 5% of each other. Therefore the differences could've been a combination of other scenarios ranging from imperfections in construction, slightly different soil parameters or the different loading scenario.

The measured load-deflection curve from the test on Test Cap 2 is shown in Figure 5-8. Also shown is the predicted load-deflection computed by GROUP. The same general trends are observed as with Test Cap 1. Initially, the response is relatively stiff and linear and the deflections are small. However, at a load of about 80 kips there is a significant change in slope as the back pile begins to pull out of the ground at which point the lateral deflections increased. The load-deflection curve observed during the test on Test Cap 1 was slightly higher than predicted by GROUP while the curve for the test on Test Cap 2 was slightly lower. However, the percent difference from GROUP in load for a given deflection for Test Cap 2 was typically less than about 5% as was shown with Test Cap 1.

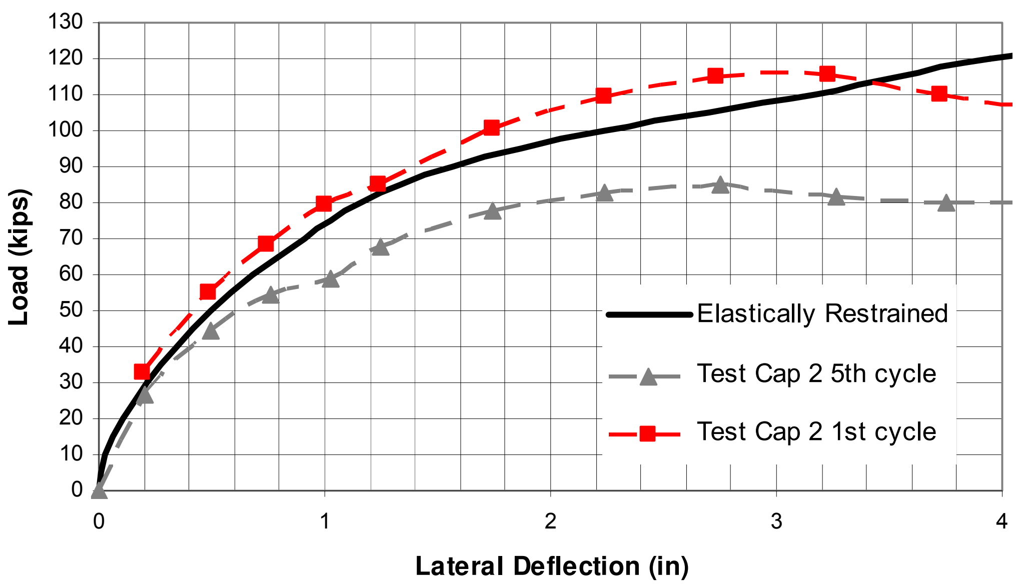


Figure 5-8 Observed vs. estimated deflection Test Cap 2.

Figure 5-9 shows the observed rotation of Test Cap 2 as a function of load. Rotation was computed using two string pots on the top face and two string pots along the front face. Figure 5-9 shows plots of load versus rotation from both sets of string pots. Small rotations were again observed until 80 kips at which point the piles began to lose side friction and the amount of rotation was magnified. The measured load versus rotation curves again compare favorably with the curve predicted by GROUP It is important to note how similar all three of these curves are to each other and that they are slightly lower than what was observed with Test Cap 1.

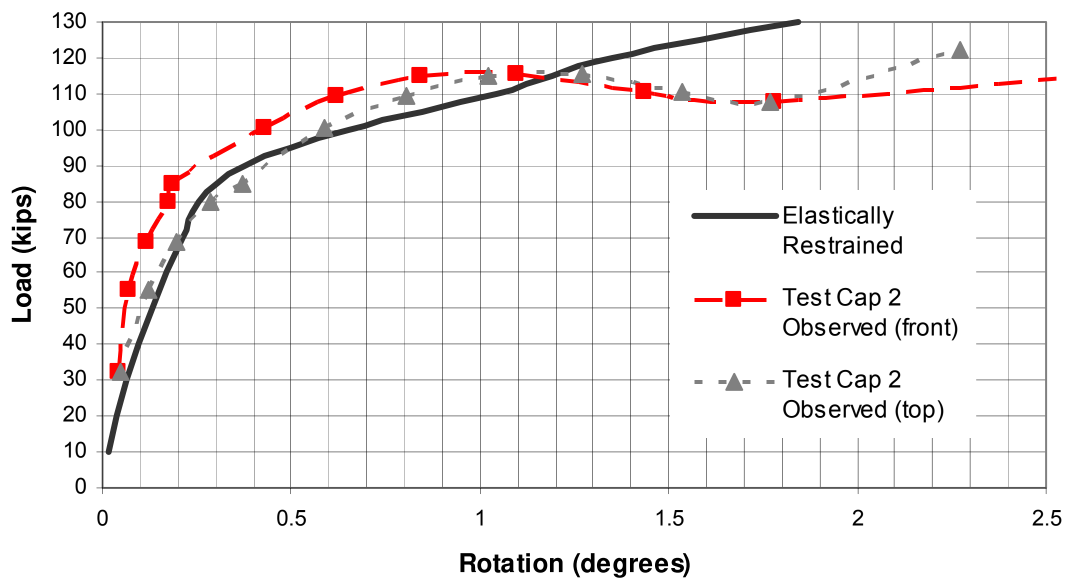


Figure 5-9 Test Cap 2 observed vs. estimated rotation.

Plots of maximum negative moment versus load at the base of Test Cap 2 are presented in Figure 5-10. Figure 5-10 shows five different curves and one horizontal line that represents the piles ultimate capacity; the two solid curves represent the maximum negative moment predicted by GROUP for the front and back row piles, the two dashed lines represent the observed moments from Test Cap 2 derived from the strain gauges, and the black dashed line represents what was predicted by LPILE. It should be noted that the front pile was both predicted and observed to develop a larger moment for a given load. The agreement between measured and predicted moment is reasonably good until the load of reaches about 80 to 90 kips. This is the load at which GROUP predicted uplift to begin to cause failure. LPILE on the other hand doesn't predict failure even when the moment capacity is exceeded by the applied moment; it continues to predict an approximate linear curve. It is noted that LPILE only analyzes a single pile and in order to determine the moments shown in Figure 5-10 for a group of two piles, equivalent lateral forces were determined from GROUP and these respective forces applied to a single pile model in the LPILE program.

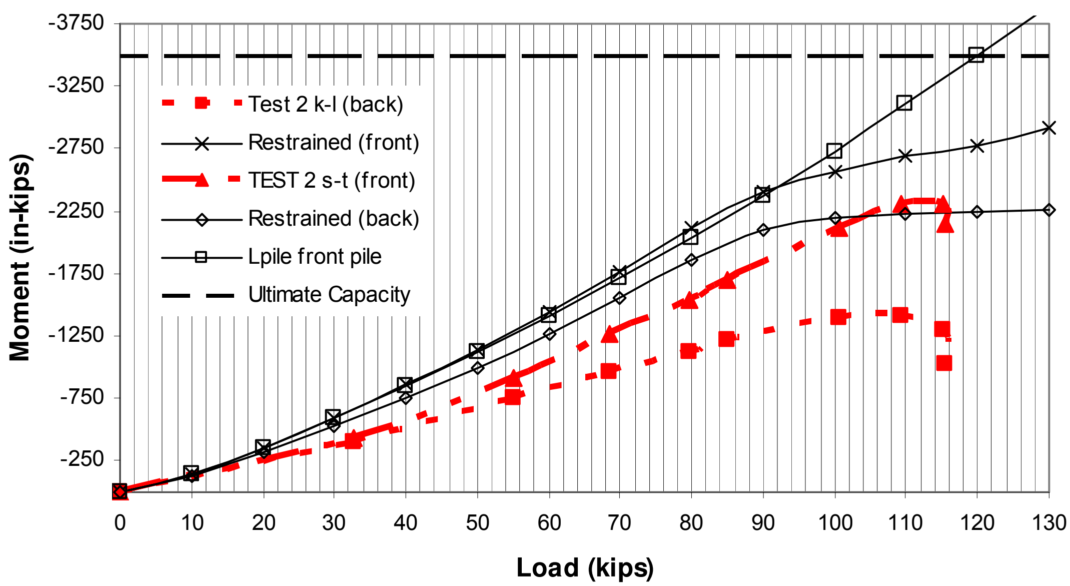


Figure 5-10 Maximum negative moment at grade.

As shown in Figure 5-10 the observed moments varied quite significantly from the prediction by GROUP once the back pile began to lift up and cause the cap to rotate. As shown, LPILE does not predict this sort of failure since it only analyzes a single pile with no specified pile cap and therefore rotation is not estimated resulting in the relatively linear curve shown. GROUP does predict this rotation and failure scenario and as shown predicts that once the piles began to uplift and rotate significantly that the moments only gradually increase rather than continue this linear relationship as was observed.

Figure 5-11 Test Cap 2 failure in pullout.

There was some question on the mode of failure. While it was considered that the piles pulled out of the ground, another possibility was that the piles remained in the ground and the concrete lost its friction with the pile and slipped. Figure 5-11 shows a close up of the back pile for Test Cap 2. It is seen here that a cavity was formed around the pile and that it did indeed pull out of the ground. There is no evidence of slippage between the pile and the cap. It was observed during both the loading and unloading parts of each cycle that the pile and cap remained completely connected with one another.

5.2.3 Test Cap 3

As explained in Chapter 3; Test Cap 3 and Test Cap 4 did not have a reinforcing bar connection detail; rather, their connection capacities were dependent upon their respective pile embedment lengths. For Test Cap 3 the pile was embedded 1 foot into the cap with for Test Cap 4 the pile was embedded 2 feet. From a construction and economics standpoint, it would be desirable if these two connection details could provide the same or similar capacities to that measured for the connection details involving reinforcing cages. In the field, construction is much simpler and less expensive if the reinforcing cage connection is left out and only a minimum embedment length is provided.

Another important fact to reiterate is that Test Cap 3 not only lacked the reinforced connection detail but also the piles remained hollow (refer to Chapter 3 for Test Cap 3 specs). Although filling the pile with concrete would improve its moment capacity, the pile was left hollow to simulate typical practice by the Oregon DOT which does not fill the piles with concrete.

The same loading sequence used for the tests on Cap 1 and Cap 2 was followed for Test Cap 3 with one exception. As cracking developed at the elevation at which the force was applied, a significant amount of concern arose. It was feared that the connection of the loading equipment to the pile cap would fail if the pile cap were pulled back to its' initial position prior to loading to the next deflection target. Therefore, after a zero load was registered the applied load was increased to the next target deflection.

As indicated previously, less data was collected from this test than for the other caps because without a reinforcement detail only a few strain gauges were able to be installed and half of them failed either during construction or during loading. In addition, some electronic data was lost due to problems with the data acquisition system. Nevertheless, sufficient basic information was obtained to help understand the behavior of the test cap.

During the first push of the 5th load increment corresponding to 1.25 inches deflection and 80 kips of lateral force, a loud popping sound was heard. Observations indicated that a crack had developed along the front of the pile cap and approximately 1 foot above grade as shown in Figure 5-12. The combined shear and tensile forces developed within the cap exceeded the concrete capacity in the absence of vertical reinforcing steel. Without this vertical steel, the stresses resulted in a shear crack.

During the next load increment, at approximately 90 kips, another popping sound was heard and additional cracks were noticed. These cracks began near the top of the back pile and propagated across the cap at the same elevation as the top of the embedded piles. The crack then joined the previous crack near the front of the pile cap. The new crack on the back and side of the cap can be seen in Figure 5-13. Again it appears that the applied force exerted by the hydraulic ram was not transferred to the pile as it was during the other tests due to the lack of vertical reinforcement with the center section of the pile cap. The cracks shown in Figure 5-13 continued to propagate until failure occurred as shown in Figure 5-14.

Figure 5-12 Initial cracking on the front face and size of Pile Cap 3.

The load versus deflection curve observed during Test Cap 3 is shown in Figure 5-15. Despite the cracking and shear failure exhibited in this test, the load-deflection curve is surprisingly similar to that for the other tests. With the excessive amount of cracking it appeared that the back pile did little in resisting deflection, yet either the front pile compensated or the remaining concrete was sufficient to transfer load between the two piles. The cracking occurred at the location where the front face string pots were located and that their respective data may not be entirely accurate.

Figure 5-13 Cracking at a 90 kip load on the back and side of Test Cap 3.

Figure 5-14 Test Cap 3 at failure.

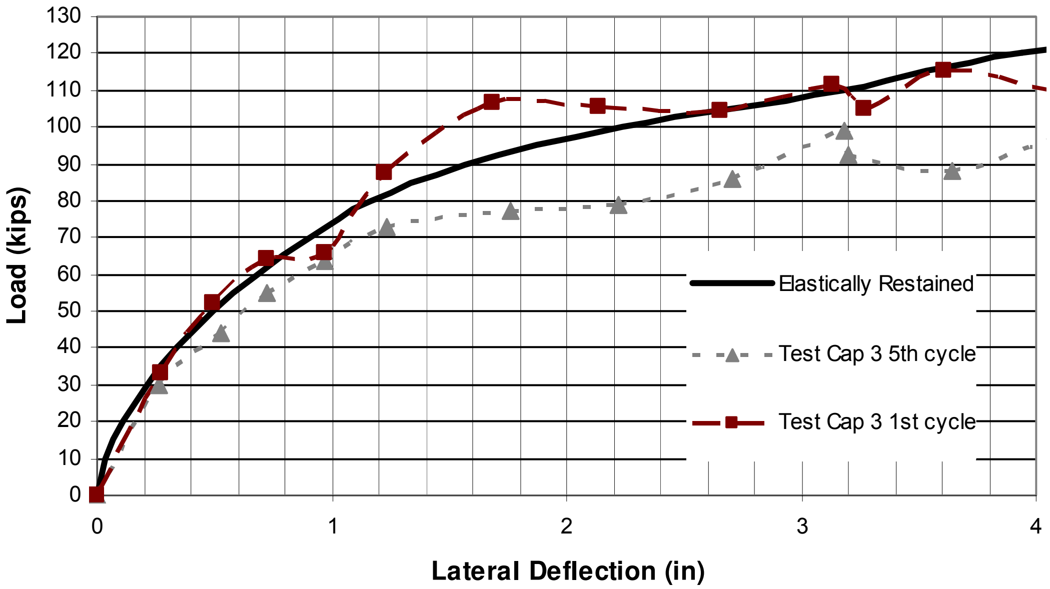


Figure 5-15 Observed vs. estimated deflection Test Cap 3.

5.2.4 Test Cap 4

As explained in section 3 of this report Test Cap 4 included a pile embedment length of 24 inches which is twice as long as the prior two tests, it also did not include a reinforcing steel connection detail. The test was conducted to determine if the embedment length would be long enough to act as a reinforced connection such that the full capacity of the piles could be developed. Because the first two tests both had adequate connections and the back piles pulled out of the ground, it was concluded that if the connection were adequate Pile Cap 4 would also fail by having the back pile pull out of the ground.

The loading sequence followed the same pattern as was followed in the prior two tests. The measured load versus deflection and load versus rotation curves are presented in Figure 5-16 and Figure 5-17, respectively. Once again the measured curves are compared with the curves predicted by GROUP. In general, the measured load versus deflection curve is 5 to 10% higher than the curve predicted by GROUP. However, the measured curve is still very similar to that obtained for Test Caps 1 and 2 where vertical reinforcing steel was used in the connection detail.

Similar to Test Cap 2 two rotation values were computed using two string pots on the top face of the cap and two string pots of the front face. The top face values are thought to be more accurate because they span a distance of 6 feet which is much greater than the front face string pots which span roughly 1.83 feet. The greater span should lead to a lower chance of error in the rotation computation; however, as the pile caps rotate and translate simultaneously it becomes difficult to estimate the actual rotation from the top string pots. The rotation computed from the front face string pots is consistently higher than that computed from the top face string pots for a given load. The percent difference between the two rotations becomes smaller at higher load levels, but a significant error is apparent at lower load levels. The computed load versus rotation curve is in good agreement with the measured curve, based on the top face string pots; at loads less than about 90 kips but then overestimates the measured rotation at higher loads.

Based on the test results, the 2 foot pile embedment length used in Test Cap 4 was sufficient to provide tensile capacity such that the piles and cap remained in complete connection and the back pile pulled out of the ground. This resulted in a significant increase in deflection and rotation as shown in Figure 5-16 and Figure

5-17, respectively. Figure 5-18 shows how the back pile lifted up and out of the ground causing the deflection and rotation.

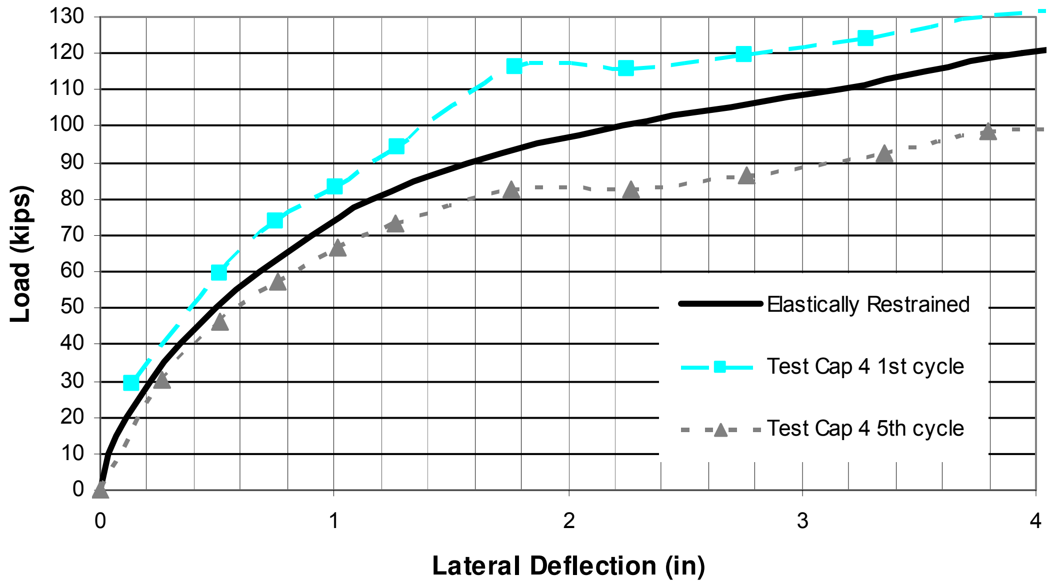


Figure 5-16 Observed vs. estimated load vs. deflection curves for Test Cap 4.

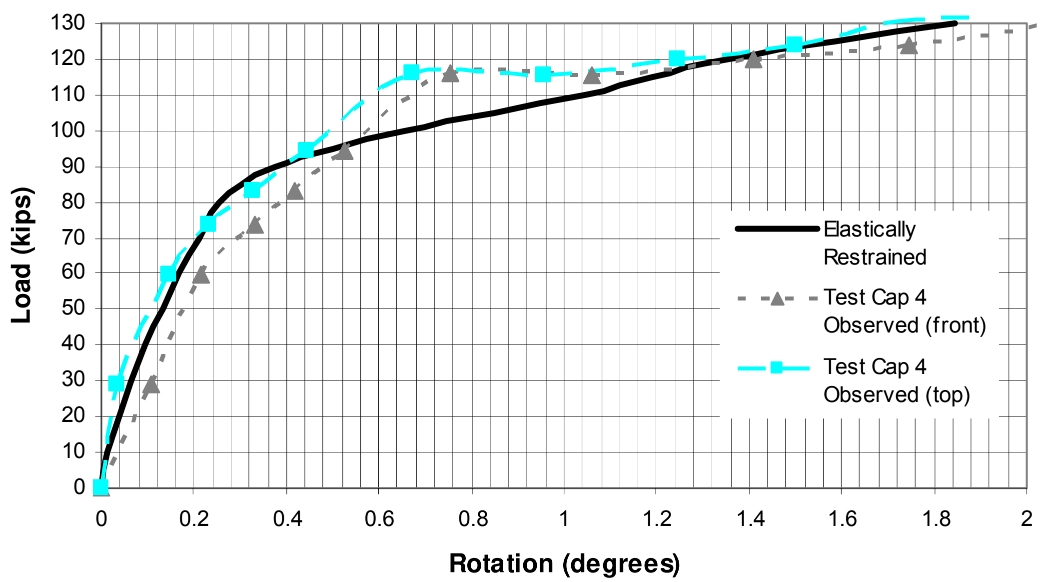


Figure 5-17 Observed vs. estimated load vs. rotation curves for Test Cap 4.

Figure 5-18 Test Cap 4 at failure.

5.3 Analysis of Longer Piles

The connections failed to reach their ultimate moment capacity and a proper analysis of them was not achieved. It was therefore necessary to analyze longer piles such that pullout was less of an issue and the connection could reach its full capacity. The figures below show three configurations that were analyzed in GROUP; the 40 foot pile is the same as was tested while the 60 foot and 80 foot piles were analyzed assuming that the additional lengths consisted of the same soil as was present in the last layer of the soil profile.

As shown in Figure 5-21 a 60 foot pile would have been ideal for this testing. The longer pile would have had more resistance against pull out and allowed the capacity of the connection to be fully utilized. It also would have allowed more load to be applied to the cap with lower amounts of rotation and deflection.

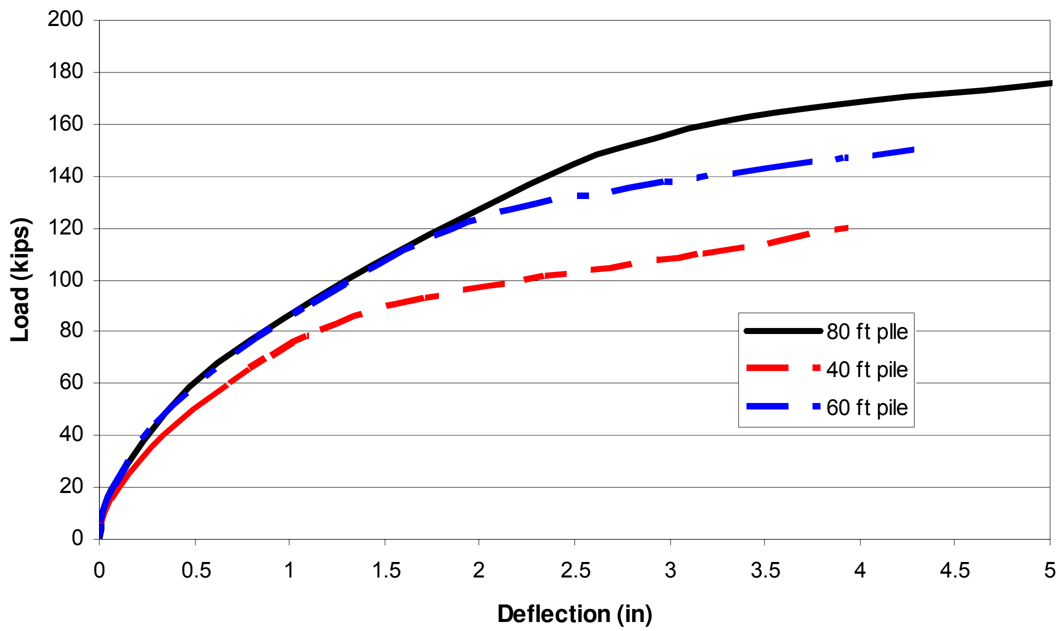


Figure 5-19 Predicted deflection of longer piles.

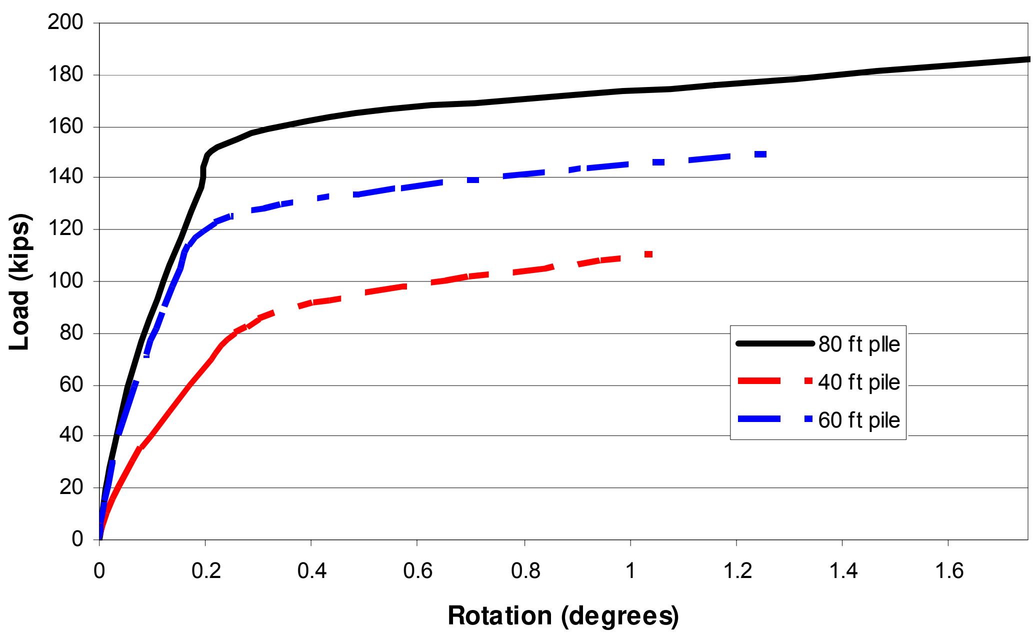


Figure 5-20 Predicted rotation of longer piles.

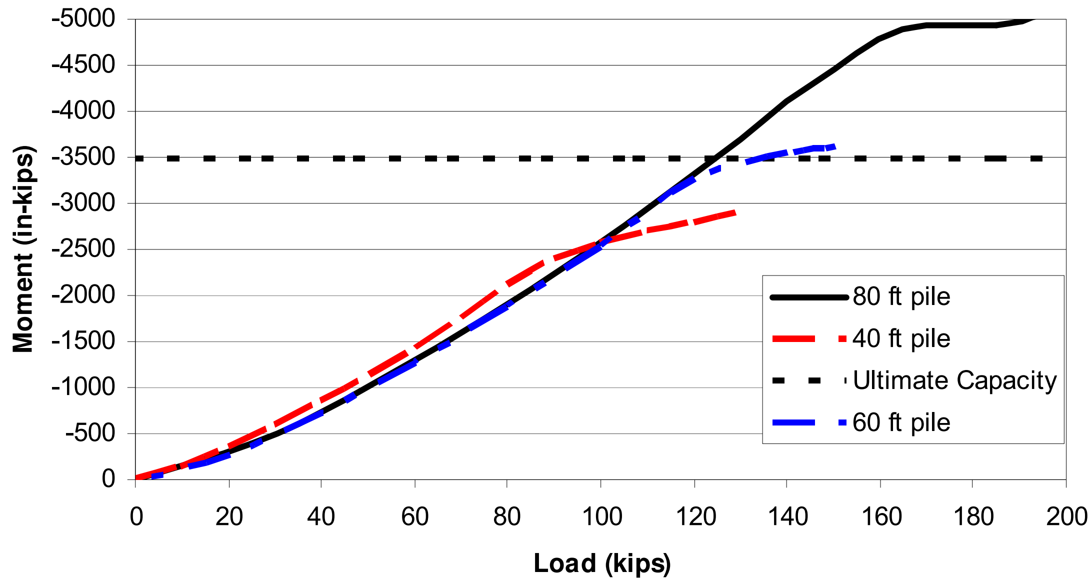


Figure 5-21 Predicted moment of longer piles.

5.4 Comparison of Observed Strain

A portion of the strain gauges produced either insignificant data or failed during loading; nevertheless, a proper comparison of those that functioned was conducted to better understand the nature of force transfer within the pile and pile cap. Shown in Figure 5-22 through Figure 5-28 are strain gauge values with applied load on the abscissa and micro strain on the ordinate. The location of each of these gauges is mentioned in the figure caption and located on the pile at grade. In addition, a drawing of the location of the strain gauge location on the pile cap is inserted in each figure. Please refer to section 3.5 for the exact locations of these gauges. The charts in figures below show the similarities within the tests and provide a good understanding of the forces within the pile cap system. As expected, the strain gauges located on the side closest to the force were in compression while the gauges on the opposite side of the pile were in tension. If the piles were in pure bending, the tensile and compressive strains would be equal but opposite in sign. In cases such as this, where both axial forces and bending moments are present, the strain values will be different. In this case, the axial force is proportional to the average strain, whereas the bending moment is proportional to the difference in strain.

The strain readings allowed a moment to be computed and these moments are presented subsequently in this report when relevant. As the strain versus load curves generally show, the strain on each pile face increased until the back piles began pulling out and the pile cap started rotating and deflecting a large amount. At this point, the strain gauges reached a maximum and then began to decrease towards zero. The strains on opposite faces of the back pile are much higher in tension than in compression suggest that there is significant moment plus a tensile force at the pile cap-ground interface. However, the difference in strain on the front and back faces of the front pile is relatively small, while the average strain level is lower than on the back pile. These observations suggest that the bending moment is higher on the front pile but that the axial force is smaller than on the back pile.

Strain gauges located along the reinforcement 4 feet below grade (locations e, f, m, and n) yielded very small strains which were similar to what was estimated by GROUP and LPILE; therefore their respective strain charts are not presented. The strain gauges at locations g, h, o, and p, which were approximately one foot below grade, measured the largest strains of the strain gauges located on the reinforcement; yet lower than those gauges located on the piles themselves at the base of the pile cap.

Figure 5-26 and Figure 5-27 show the strain readings near the middle of the vertical reinforcing bars approximately one foot below grade and on both sides of the front pile. Similar to strain gauges on the front piles at grade, the gauges opposite sides of the pile develop close to equal and opposite suggesting the pile is dominated by bending stresses.

The gauges located at the top of the reinforcement at locations i, j, q, and r yielded very similar results with respect to each other. Each showed a small amount of strain until the lateral load increased enough that the cap began to rotate and then the strain increased dramatically. This is shown in

Figure 5-28, and the other strain gauges at these locations measured very similar strain levels. This observation suggests that tension is developing in the piles after the cap begins to rotate. This tension is developed as the reinforcing steel acts to hold the pile and the pile cap together.

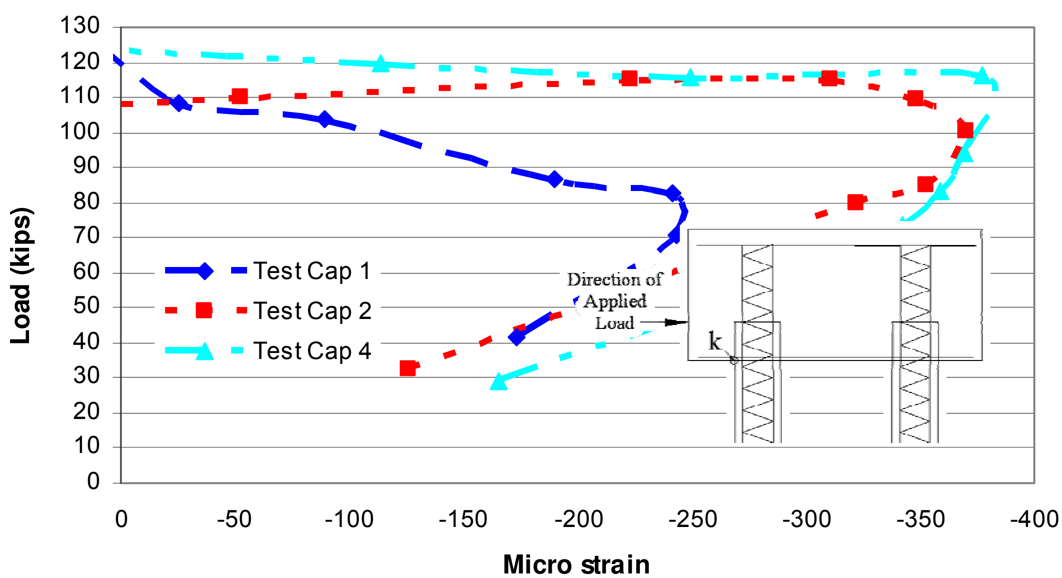


Figure 5-22 Strain gauge readings location k.

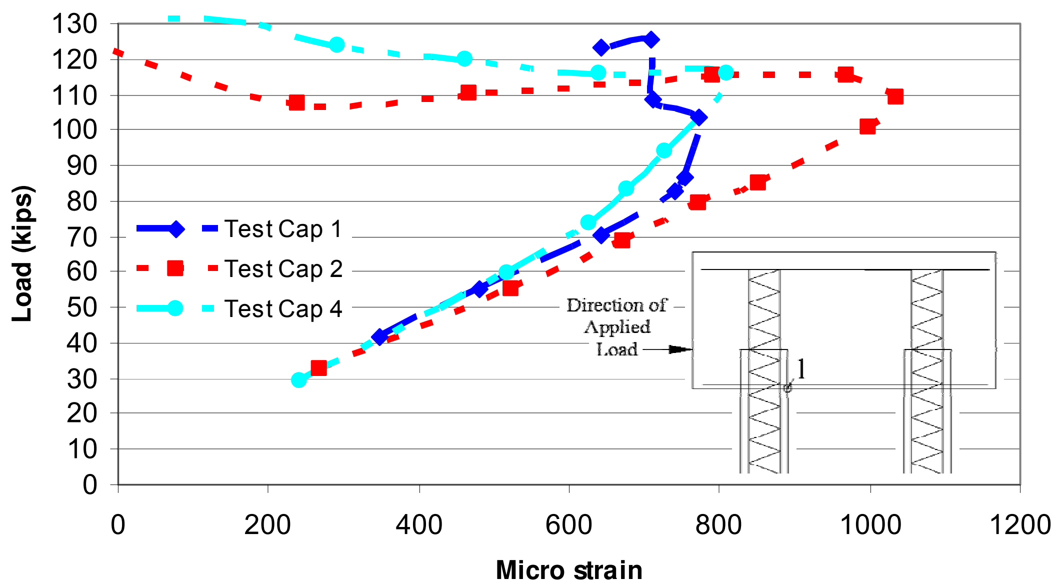


Figure 5-23 Strain gauge readings location l.

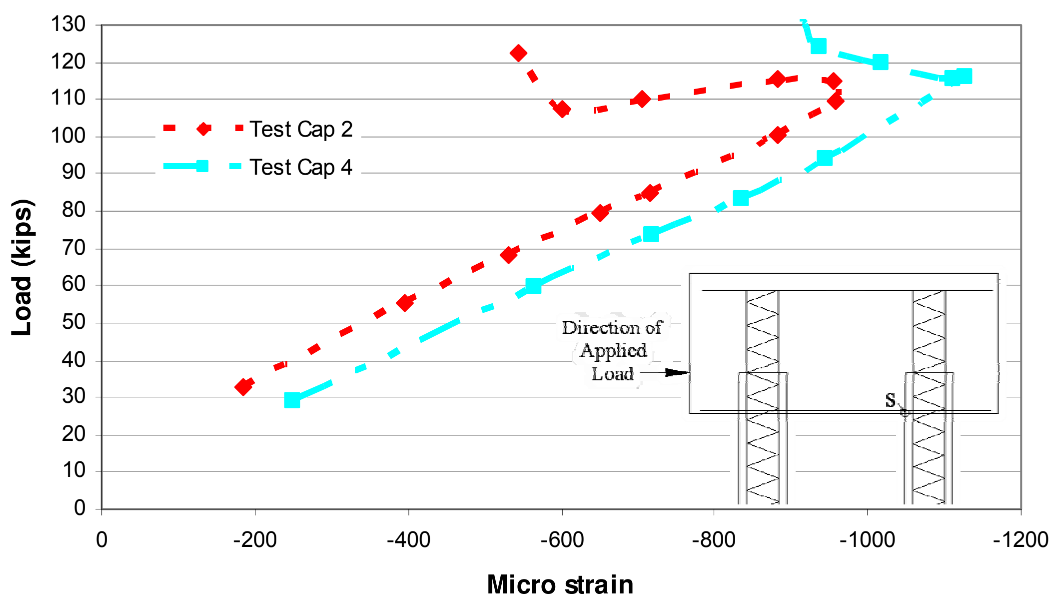


Figure 5-24 Strain gauge readings location s.

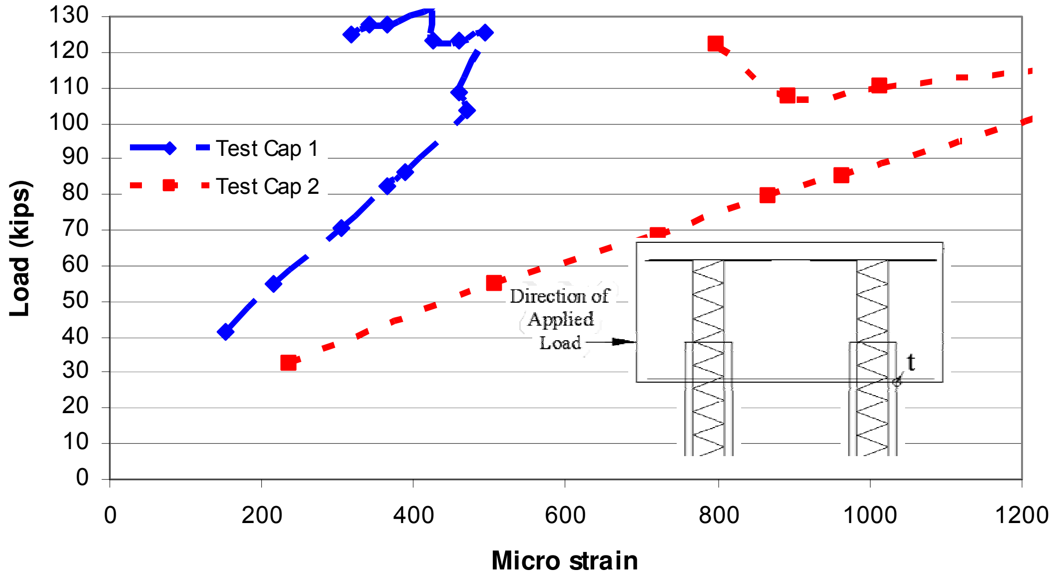


Figure 5-25 Strain gauge readings location t.

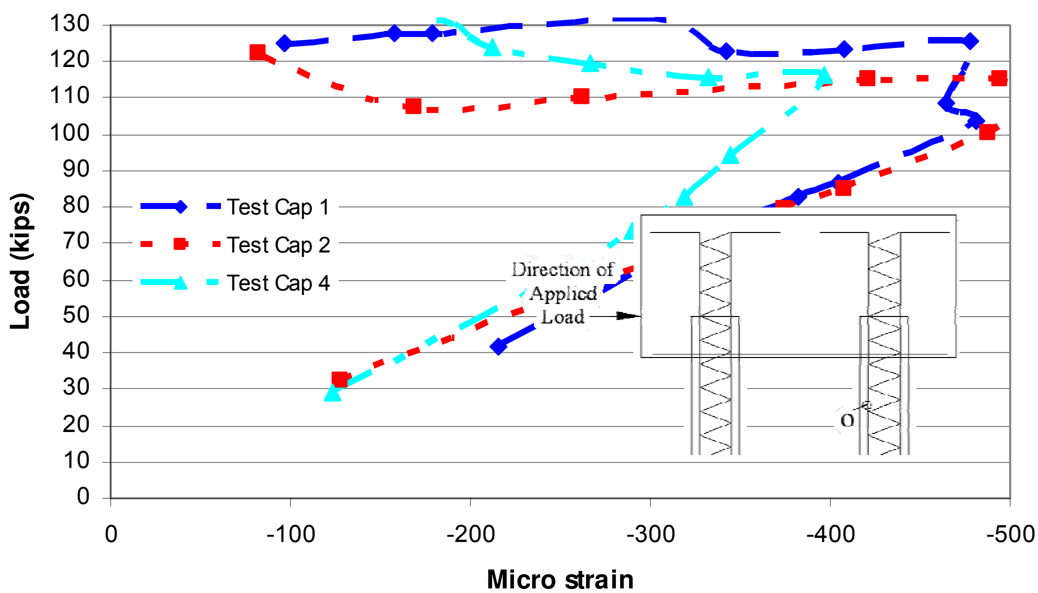


Figure 5-26 Strain gauge readings location o.

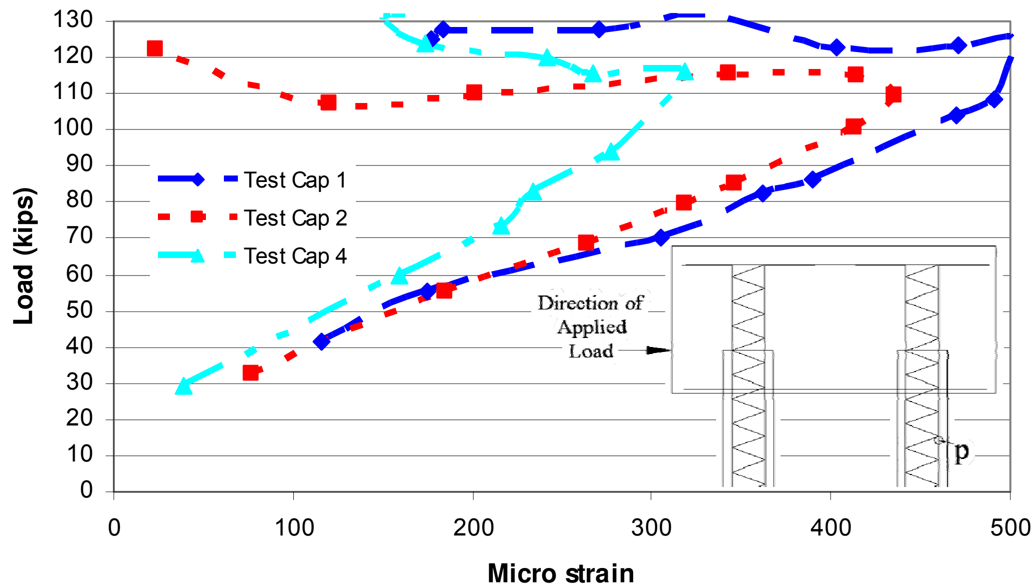


Figure 5-27 Strain gauge readings location p.

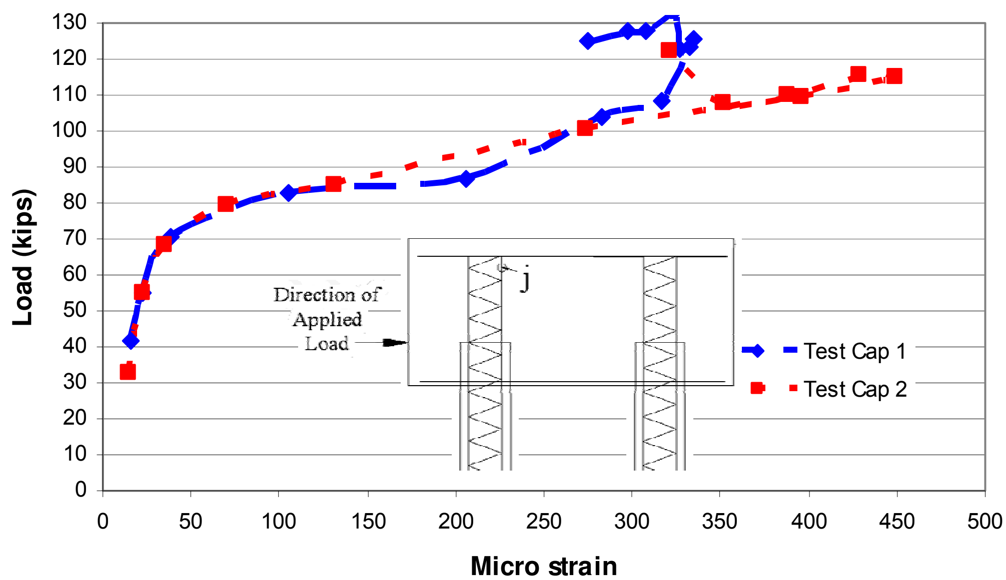


Figure 5-28 Strain gauge readings location j.

The strain gauges located along the bottom reinforced grid at locations a, b, c, and d yielded quite different yet small strains from one another. While it was not possible to develop any consistent patterns from the measurements, it does appear that the tensile force in the bottom reinforcing grid was relatively small.

5.5 Comparison of Observed Moments

The observed moments developed within the pile were calculated using the equation below.

$$M = \frac{EI(\varepsilon_c - \varepsilon_t)}{h} \tag{5-2}$$

The composite EI before cracking was determined to be 12,195,440 kip-in^2 and the ultimate moment capacity for the section was determined to be 3500 in-kips. GROUP produced a moment vs. depth chart presented in Figure 5-30 for loads of 80 to 120 kips. This plot indicates that the maximum negative moment occurs at the interface between the pile cap and the ground. The measured moment charts below confirm this occurrence because the largest observed negative moments occur at location s-t which is at grade on the front pile. The observed moment is very close to the piles moment capacity calculated by LPILE assuming non-linear behavior. Below the interface, the moment then very quickly decreases to a zero value at around 3 feet below grade. The strain gauges that were located 4 feet below the ground and attached to the vertical reinforcing bars generally confirm this pattern as the measured moments were much smaller. Below a depth of 3 feet, the moment increases to its largest positive value which occurs at a depth near 10 feet below grade. The depth to maximum moment cannot be confirmed by the strain data because the reinforcing cage did not extend to this depth.

The measured moments in figures below show a general increase with load until a maximum moment is reached at a load level of about 100 to 110 kips. This load level generally corresponds to the load level at which the back pile pulls out and the pile cap begins to rotate. Figure 5-33 shows the moments at a depth of 14 inches below grade at locations o-p and g-h. These locations as presented in section 3 are at the same location but on opposite piles; this shows that larger moments were observed on the front piles as predicted by GROUP.

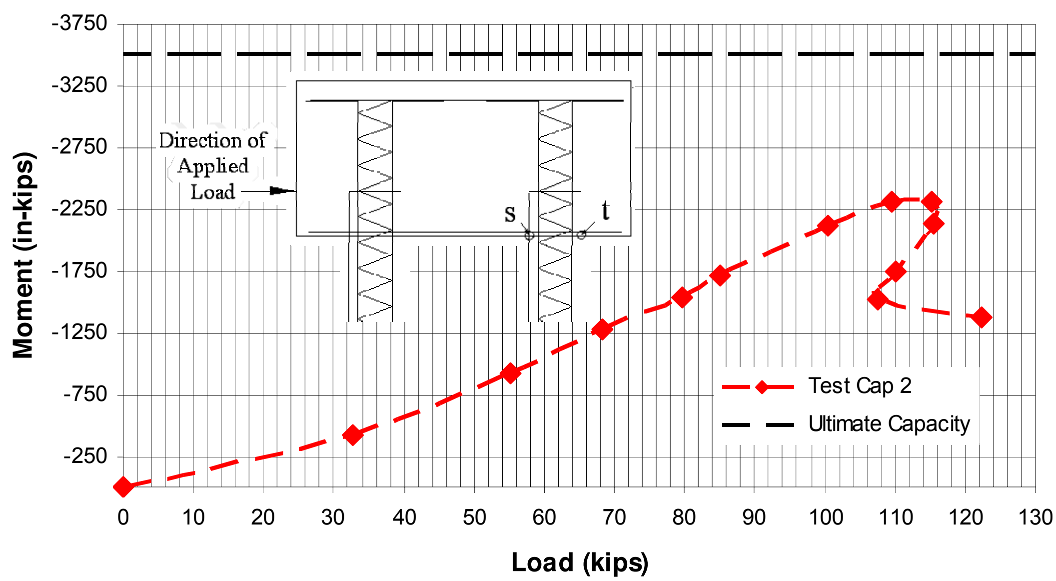


Figure 5-29 Observed moments at location s-t.

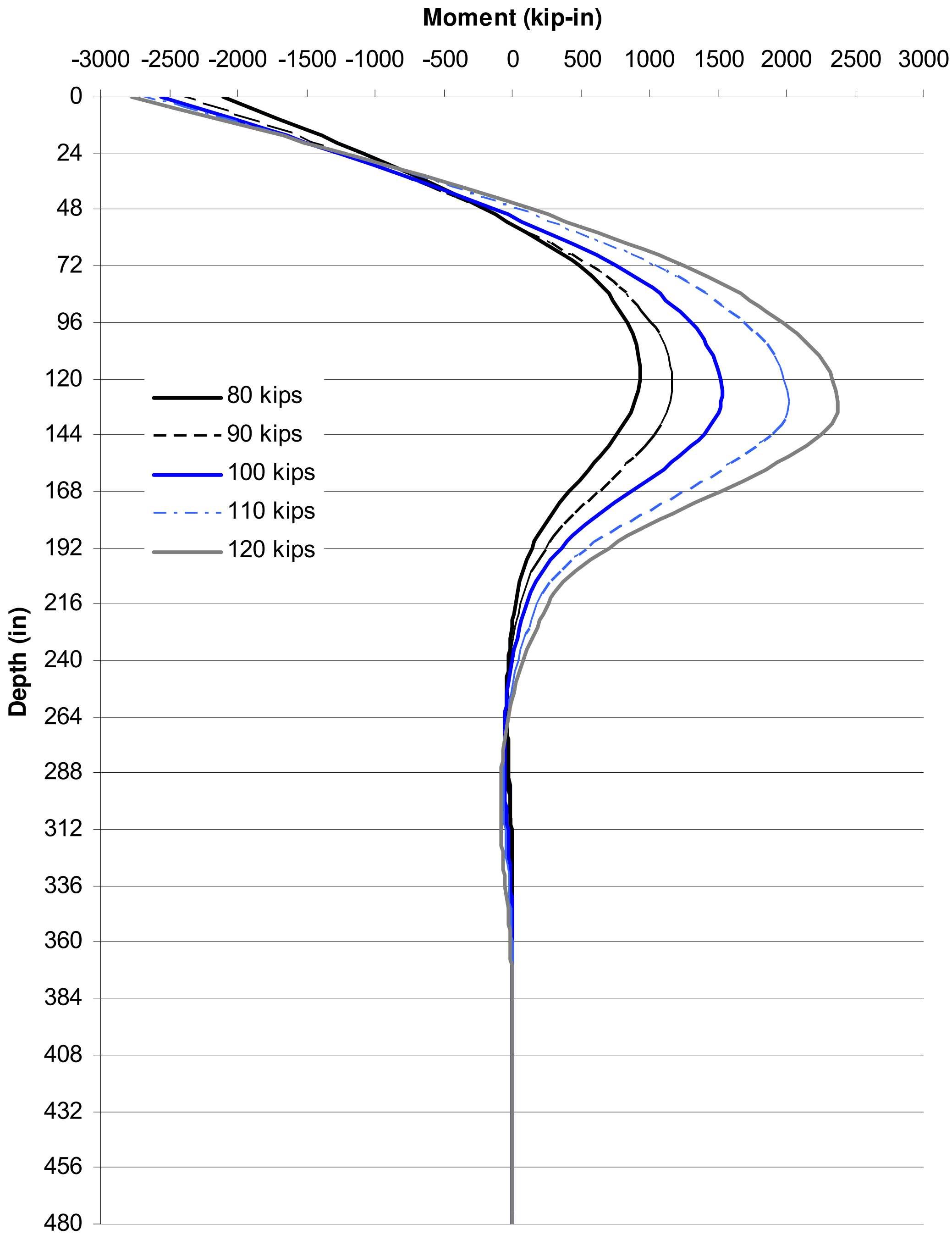


Figure 5-30 Moment vs. depth chart by GROUP.

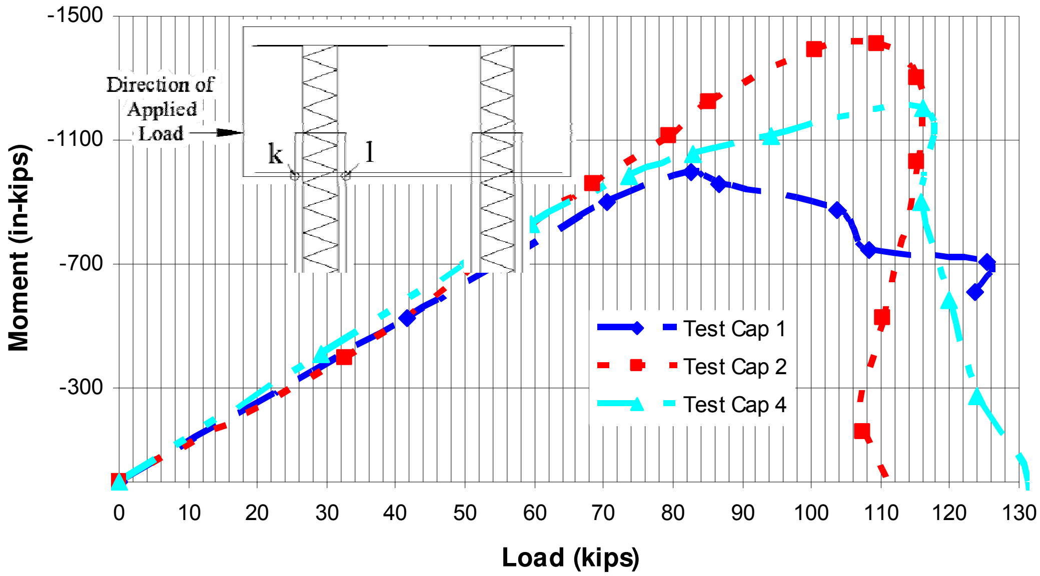


Figure 5-31 Observed moments at location k-l.

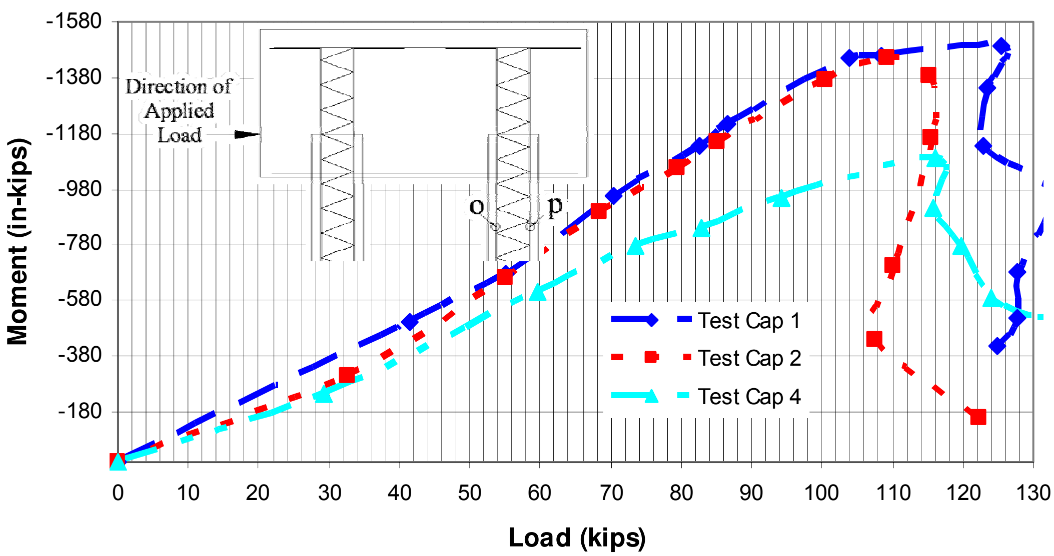


Figure 5-32 Observed moments at location o-p.

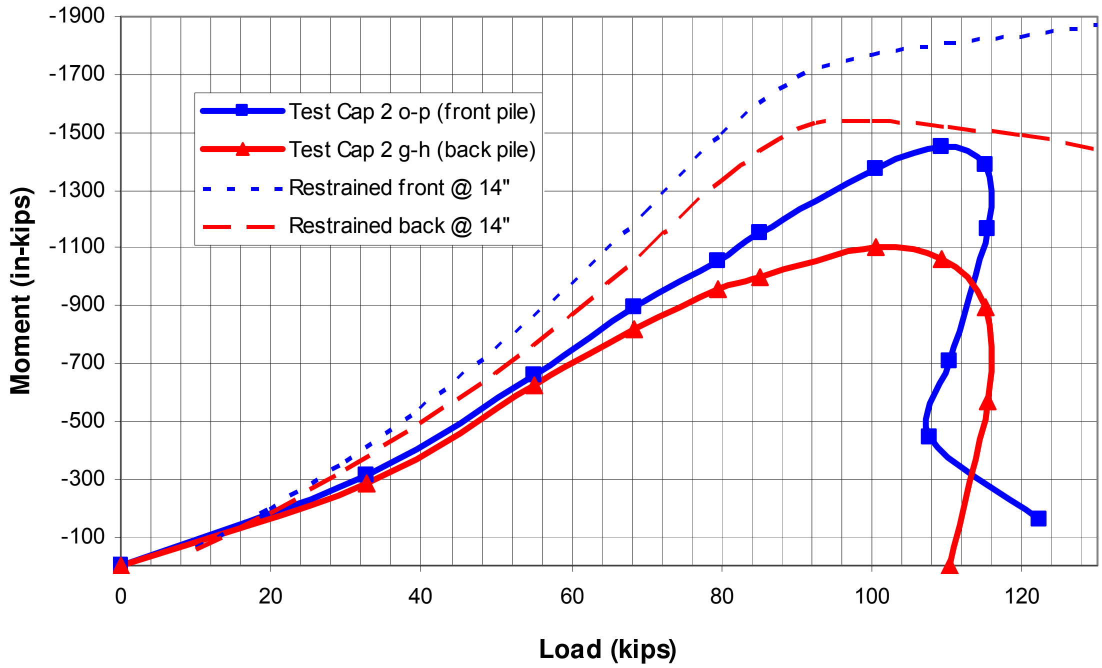


Figure 5-33 Observed and predicted moment vs. load at 14" below grade.

5.6 Comparison of Test Results

The four tests performed were each designed such that a proper comparison would be beneficial for future design. In current design with steel pipe piles it is typical to both embed the piles a sufficient depth into the pile cap as well as provide a reinforcing cage extending from the top of the pile cap and into the piles. Figure 5-34 and Figure 5-35 plot load-deflection and load-rotation curves, respectively for all four lateral pile cap load tests to facilitate comparisons.

Comparing the performance of Test Cap 1 with Test Cap 2 shows that the additional embedment length may not be necessary for applied lateral loads. The reinforcement did an adequate job in connecting the piles to the cap even when the embedment was only 6 inches.

Comparing the connection designs of Test Cap 2 with Test Cap 3 also proved to be valuable. Although the piles for both test caps were embedded 1 foot into the caps, the connection of Test Cap 2 performed very well while Test Cap 3 failed in the connection region. The connection for Test Cap 2 included a reinforcing cage extending from the pile through the pile cap while Test Cap 3 did not include any connection other than the pile embedment itself. This shows the importance of providing an adequate connection. As presented in section 4.3; Test Cap 3 was determined to be able to resist the tensile and shear forces, yet the cap still failed. The only mechanism which seems to account for the observed behavior is the deep beam failure approach; however, the application of this model to the pile cap geometry is tenuous. Although the actual mode of failure is somewhat uncertain, it is clear that the connection was not adequate.

Perhaps the most important comparison is the performance of Test Cap 4 with the other three test caps. Test Cap 4 performed very well, yielding lower deflections and rotations for a given load than any of the other three caps as shown in Figure 5-34 and Figure 5-35. The observed rotation from the front face string pots is shown in Figure 5-35 and the top face string pot data which was only gathered from Test Cap 2 and 4 is shown in Figure 5-36. The largest variance between observed rotations occurs with the front face string pots of Test Cap 2 which at low loads yield very small rotations; this could be misleading data since it varies significantly from the other tests rotations from both the front and top string pots.

The observed data leads to the conclusion that this simple 2 foot embedment connection, which was 2/3 the cap height and about 2 times the piles diameter, is an adequate design and possibly the most favorable connection presented. However, this somewhat better performance might be a result of slightly different soil parameters or variances in construction.

The load-deflection and load-rotation curves computed by GROUP assuming elastically restrained conditions were in reasonable agreement with all the all of the test results. The computed stiffness for the two pile group was about 80 kips/inch and this value was essentially the same as the measured stiffness for all four test caps.

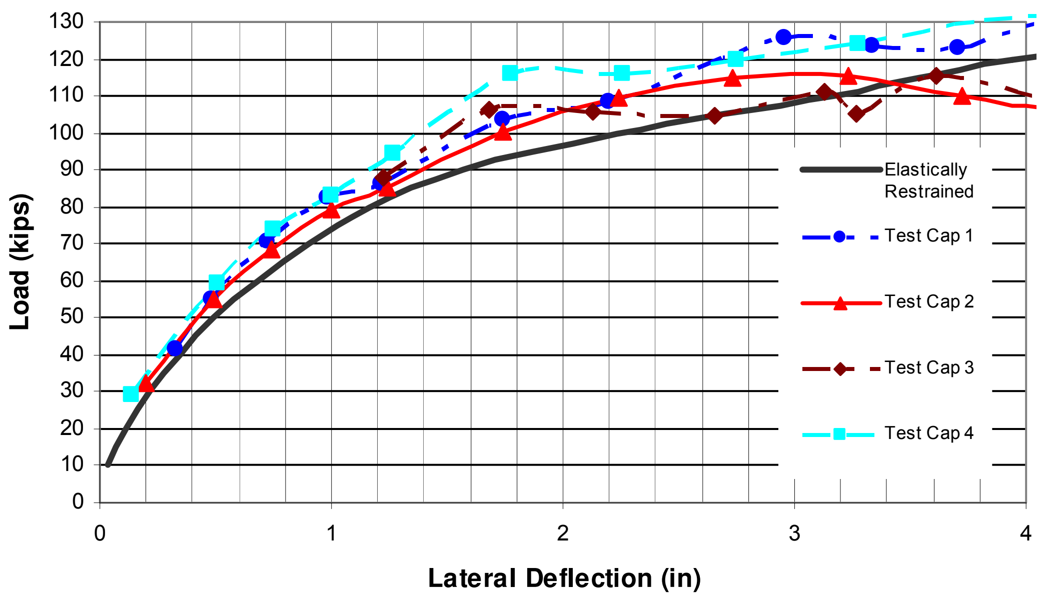


Figure 5-34 Deflection comparisons of all tests.

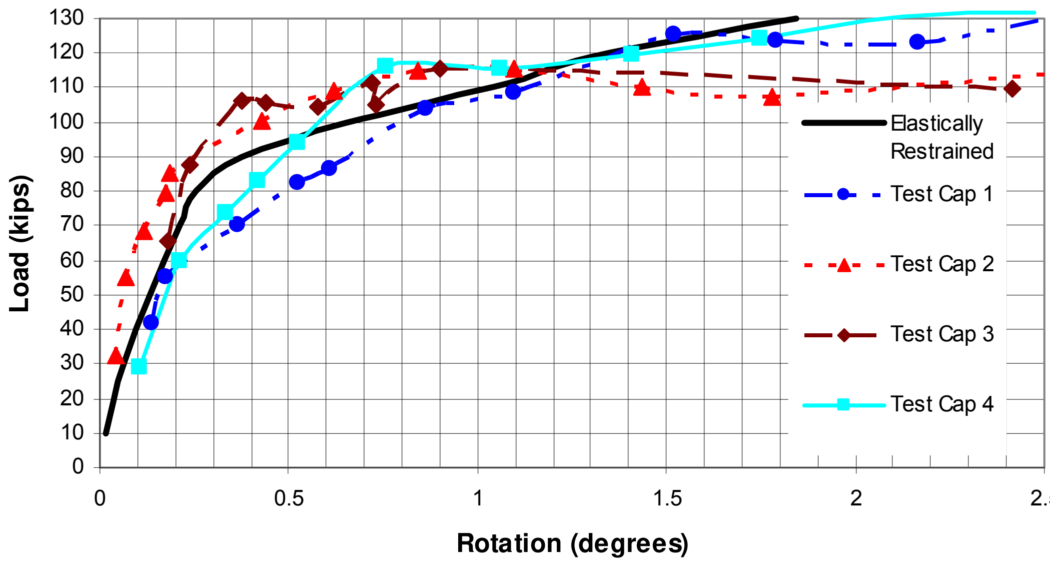


Figure 5-35 Rotation comparisons of all tests (front face).

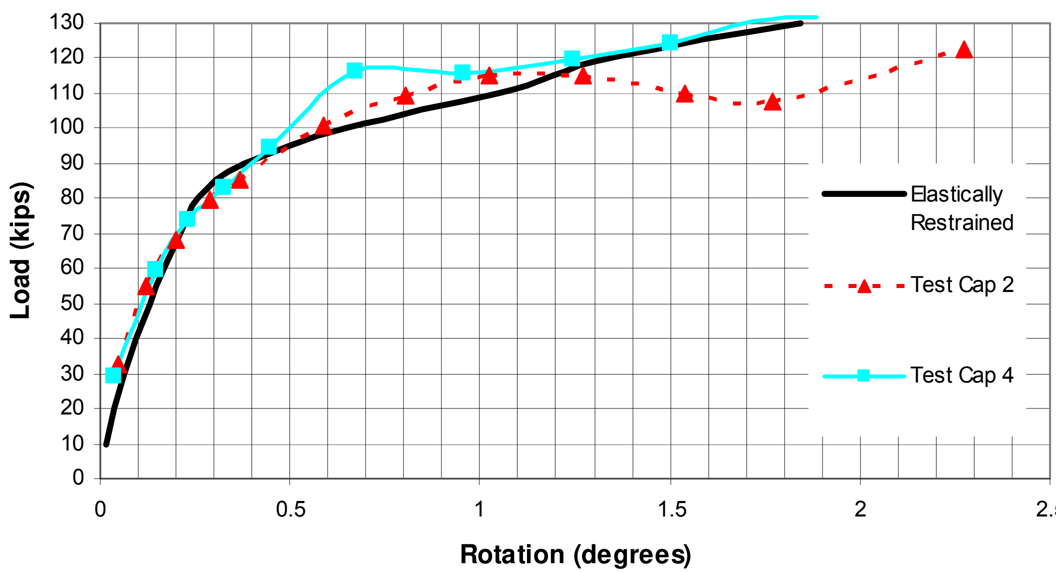


Figure 5-36 Rotation comparisons of all tests (top face).

6 Conclusions

6.1 Summary

To better understand the connection details involved with full-scale piles and pile caps four pile cap configurations were built, analyzed and then tested. All of the tests consisted of the same cap details and the only variations were that of the connection detail. Each cap was 6 ½ feet long, 3 feet wide, and 3 feet tall, with two circular steel piles driven to a depth of 40 feet and spaced at 3½ feet on centers. Reinforcing grids with #7 bars spaced at 6 inches were placed in the longitudinal and transverse directions both top and bottom. There are two variations with the connection presented in this paper; the length of the pile extending into the cap (embedment length), and the amount of rebar extending down into the pile and into the cap.

Test Cap 1 included a 6 inch pile embedment length and (4) 7 foot #6 bars extending to the top of the cap and 4 feet below grade. Test Cap 2 included a 12 inch pile embedment length and the same rebar detail as Test Cap 1. Test Cap 3 included only a 12 inch pile embedment length with no reinforcement and a steel plate at the top of the pile. The pile was not filled with concrete. Finally, Test Cap 4 included only a 24 inch pile embedment length with no reinforcing cage. All piles were filled with concrete with the exception of Test Cap 3 which remained hollow in accordance with Oregon DOT practice.

String potentiometers, strain gauges and a load cell were attached to each pile cap during testing to measure deflection, rotation, strain, and applied force. This data has been collected and when relevant presented as graphs in this paper. The pile caps were analyzed by hand calculations and two computer modeling programs; GROUP and LPILE. When appropriate the results from these programs are presented and compared with measured response. In general, GROUP yielded estimations very similar to the observed data and therefore most of the data presented in this paper has been compared to those estimations.

Testing produced results very similar to what was expected. Test Caps 1, 2, and 4 all failed at a lateral load near 100 kips with each tests respective back pile pulling out of the ground causing an excessive amount of deflection and rotation. Test Cap 4 yielded slightly lower deflections and rotations while Test Cap 2 yielded the largest deflections and rotations of the three tests. Test Cap 3 developed large shear cracks that propagated until failure while the piles remained in place with little noticeable disturbance.

6.2 Conclusions

The testing conducted supported the question of the importance of a proper connection to resist seismic events. The conclusions are:

a. a pile embedded a sufficient depth into the cap could produce a connection with an equivalent capacity of those with a reinforced detail. Pile Cap 4 which relied solely upon its embedment length to provide an adequate connection preformed as well or better than the comparable caps which relied on a reinforced connection.

b. pile caps that lack an adequate amount of reinforcement can result in an early seismic failure due to large shear and tensile forces. Pile Cap 3 lacked both an adequate embedment length and reinforcement resulting in early failure due to large shear and tensile cracks. Had the cap itself been reinforced with a cage type of configuration the deep beam failure may have been delayed if not avoided (at least for the applied loads).

c. when a reinforced connection detail is provided the length of embedment provided is not critical to its performance. Pile Caps 1 and 2 both included a reinforced connection detail; Pile Cap 1 with only six inches of embedment performed essentially the same as did Pile Cap 2 which included a twelve inch embedment.

d. programs such as GROUP and LPILE are quite accurate when predicting deflections and rotations. As shown in the graphs presented in this thesis the observed readings were nearly identical to that predicted by GROUP and LPILE though near failure their predictions proved less accurate.

6.3 Recommendation for Future Research

Based on the experience from conducting these tests, a few recommendations for future research efforts can be made. While these tests were made possible because the piles themselves were already in the ground, if new piles are driven for subsequent testing it is recommended that strain gauges be installed along the length of the pile and protected such that the data presented from the computer modeling programs can be more accurately confirmed. Ideally, if new piles are driven for subsequent testing, it would be ideal if they could extend deep enough so that failure occurred at the pile cap connection region prior to the pile pulling-out from the ground.

As an earthquake occurs, large vertical loads from the superstructure are supported by a pile cap and lateral forces are distributed through the cap. If it is assumed that these forces act at the center of mass of the cap, then the pile embedment length must either be above the center of mass such that the lateral forces don't cause a shearing of the top half of the cap or the reinforcing cage must extend above through this zone. It is recommended that future tests confirm this by applying cyclic forces to the top of a pile cap and comparing the results of embedment lengths that are both above and below the center of mass for the cap.

6.4 Implementation of Results

As shown in the testing preformed there is definite need for a proper connection detail to properly connect the piles to the cap and fully develop the piles capacity. As current common practice around the world requires a reinforced connection it is believed that with a proper embedment length that the reinforced detail can be omitted. The applied force in the current testing acted one foot above grade, if this force would have been applied a higher position it is believed that the same type of failure would have occurred with Test Cap 3. In a seismic event the force could be said to be positioned in the center of mass of the cap and therefore it is believed that an embedment length that extends a development length above the caps center of gravity should be provided. It is also important in future design not to embed the piles too far into the cap such that proper axial forces cannot be developed within the piles.

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Appendix A. Complete Test Results

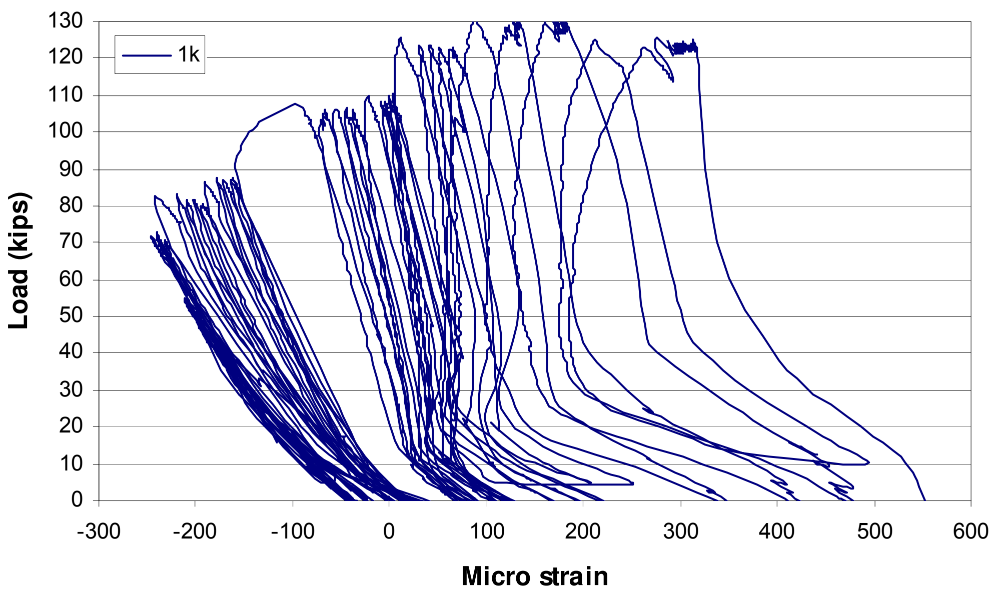

Figure A-1 Observed strain Test Cap 1 location k.

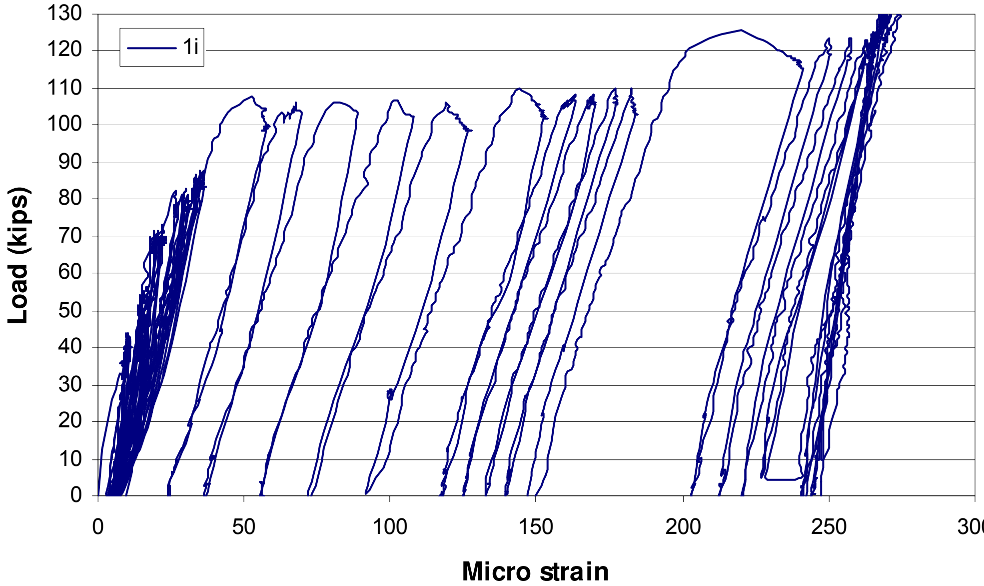

Figure A-2 Observed strain Test Cap 1 location i.

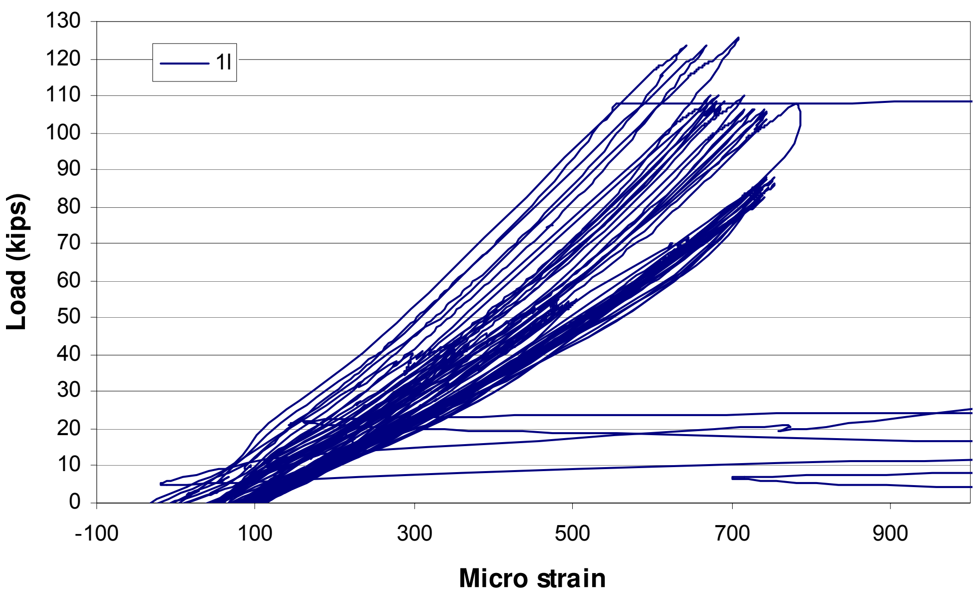

Figure A-3 Observed strain Test Cap 1 location l.

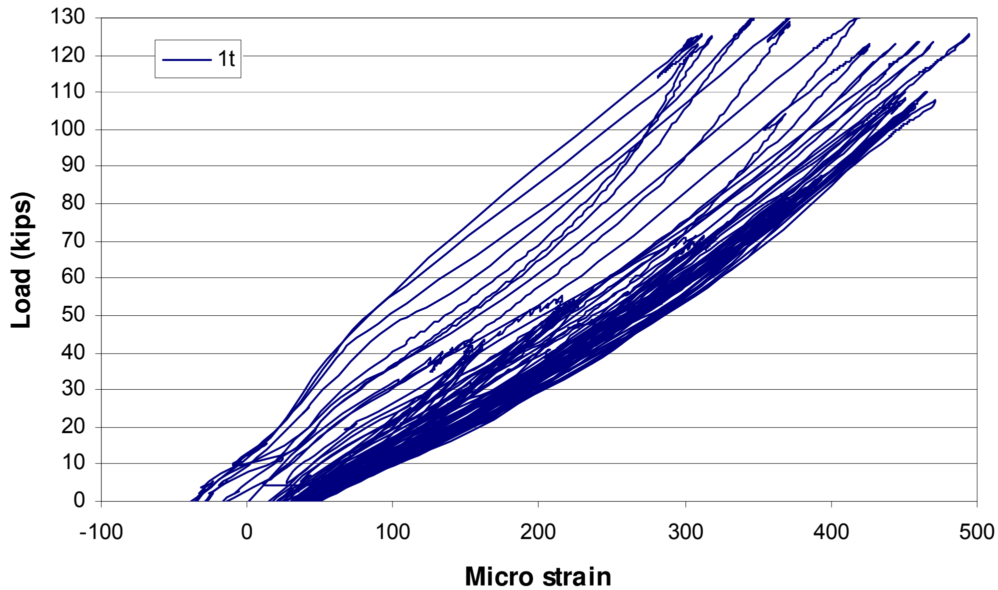

Figure A-4 Observed strain Test Cap 1 location t.

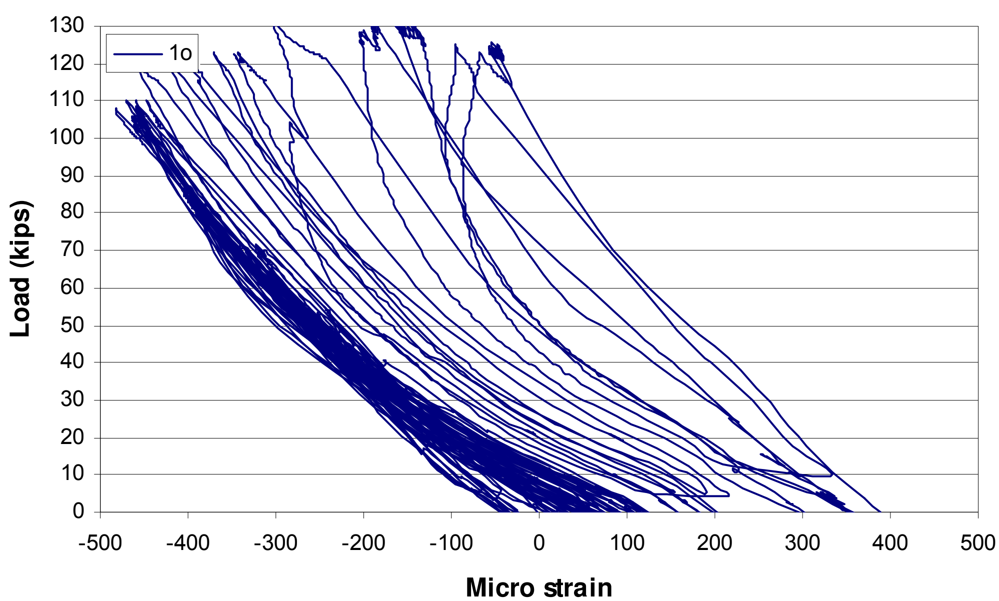

Figure A-5 Observed strain Test Cap 1 location o.

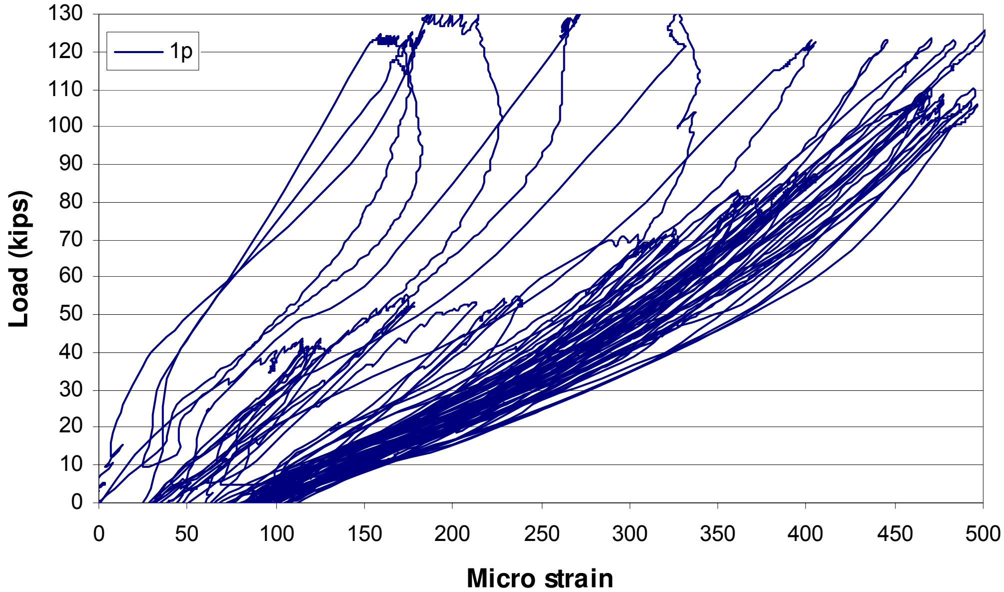

Figure A-6 Observed strain Test Cap 1 location p.

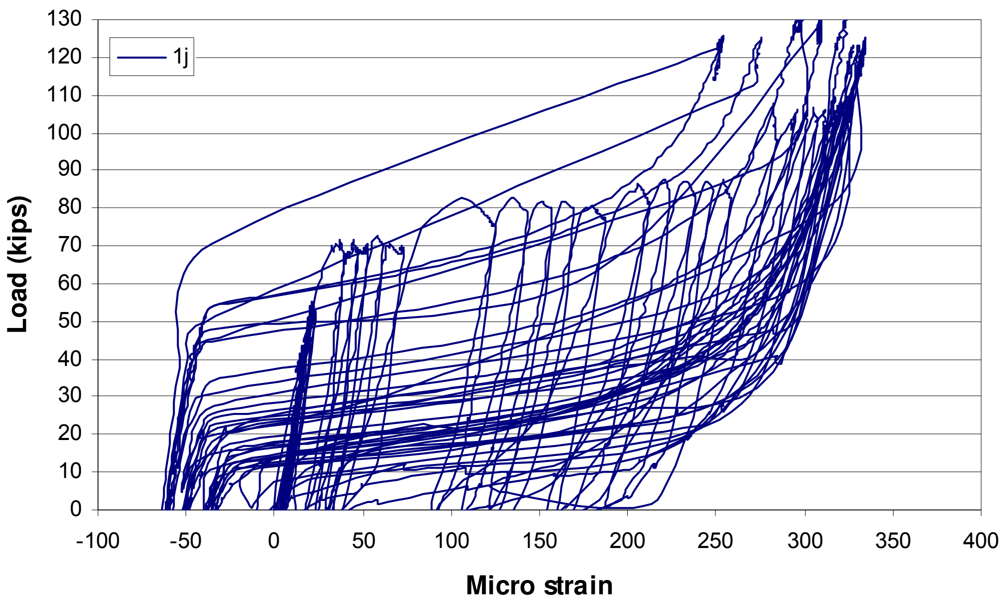

Figure A-7 Observed strain Test Cap 1 location j.

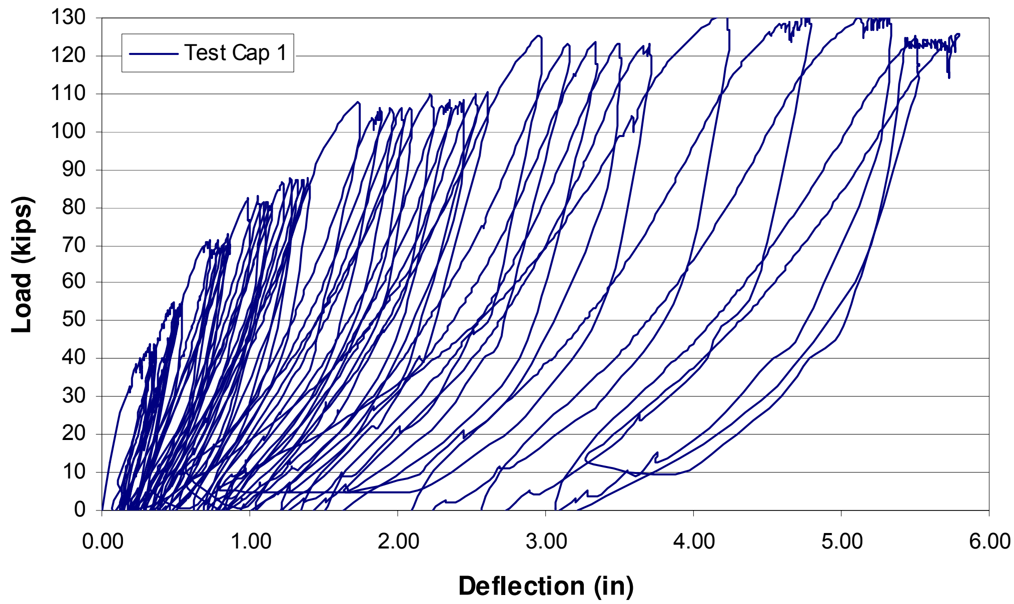

Figure A-8 Observed deflection Test Cap 1.

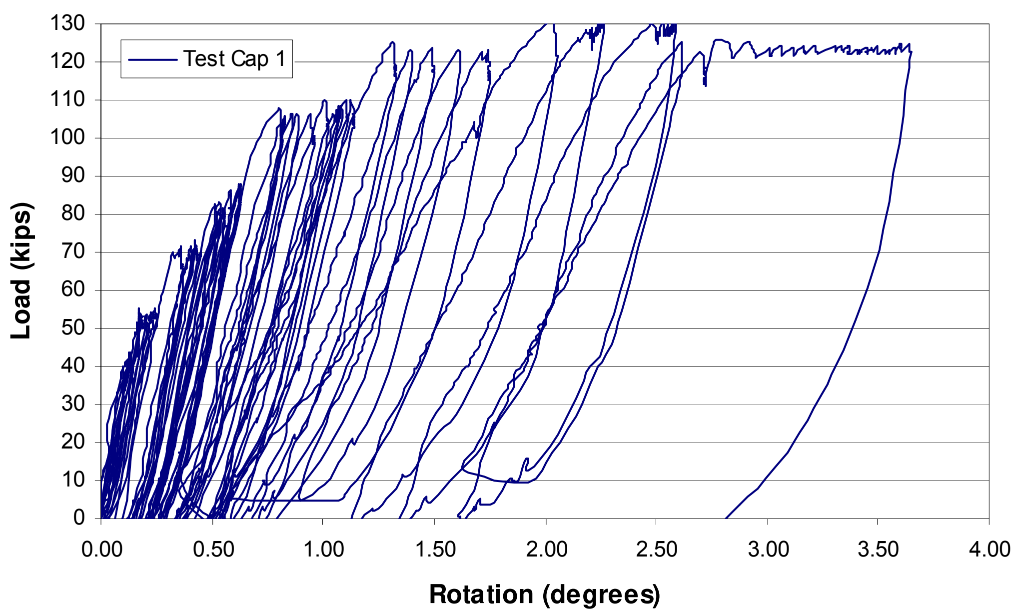

Figure A-9 Observed rotation Test Cap 1.

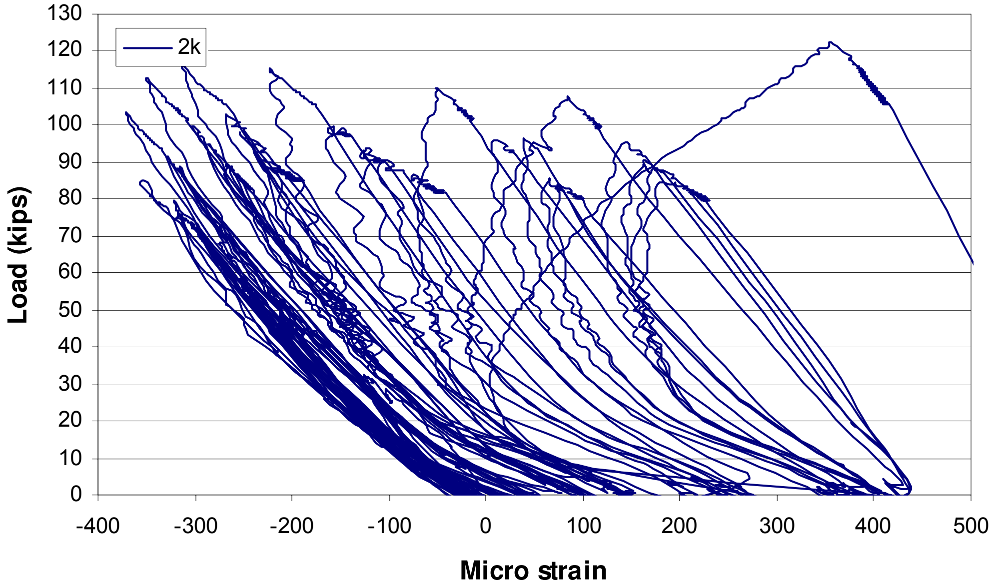

Figure A-10 Observed strain Test Cap 2 location k.

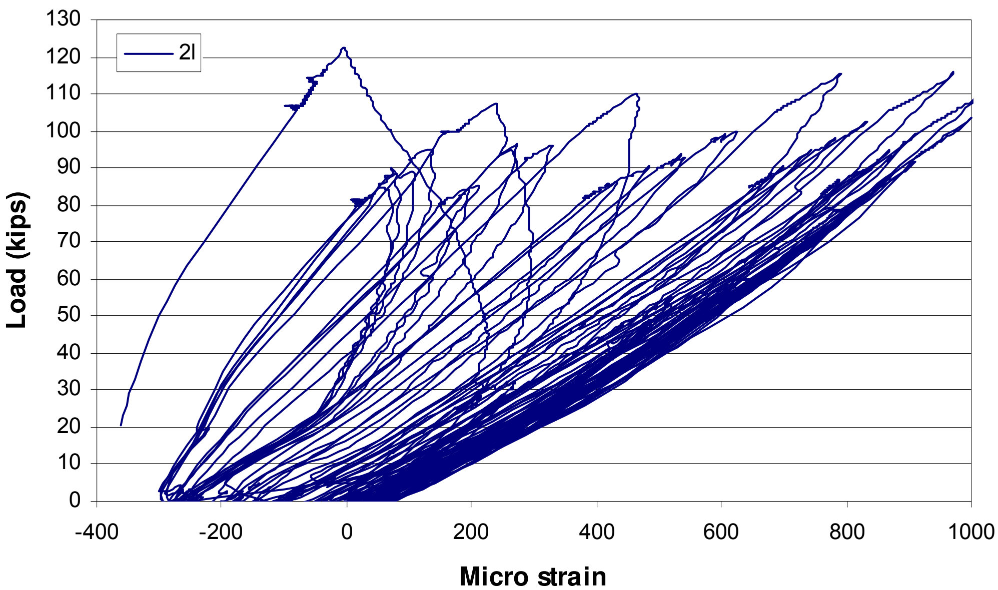

Figure A-11 Observed strain Test Cap 2 location l.

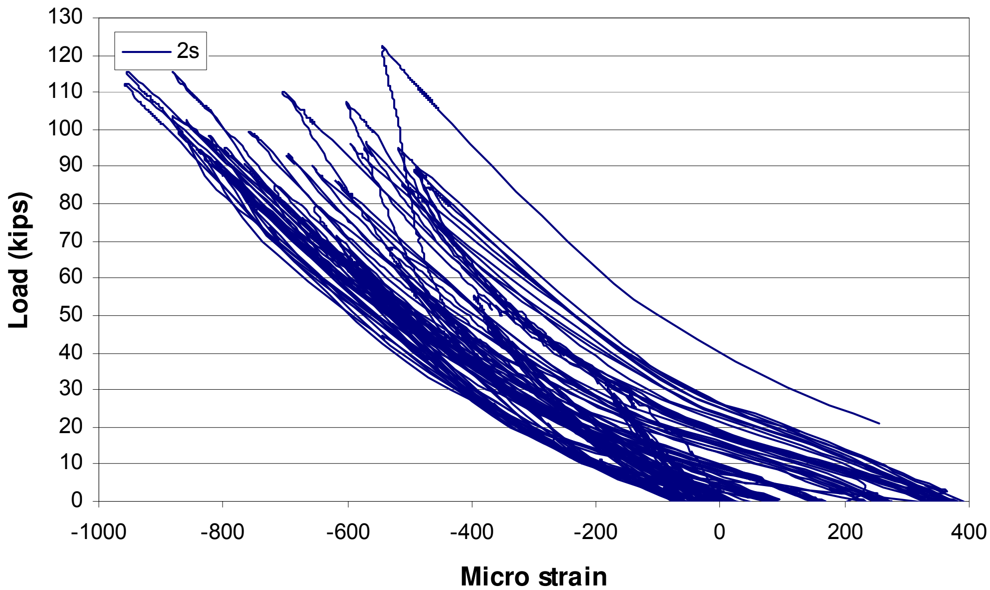

Figure A-12 Observed strain Test Cap 2 location s.

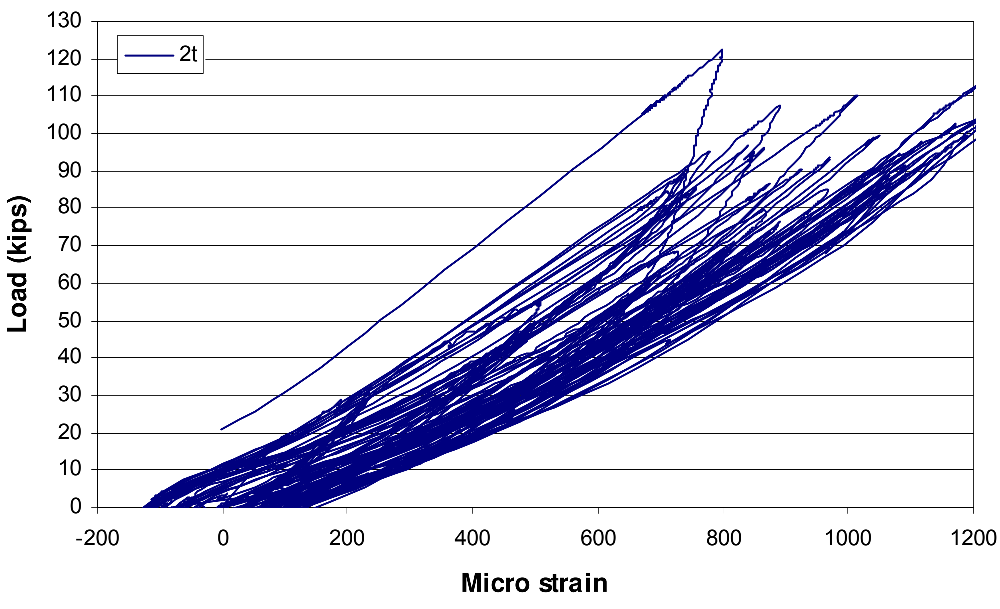

Figure A-13 Observed strain Test Cap 2 location t.

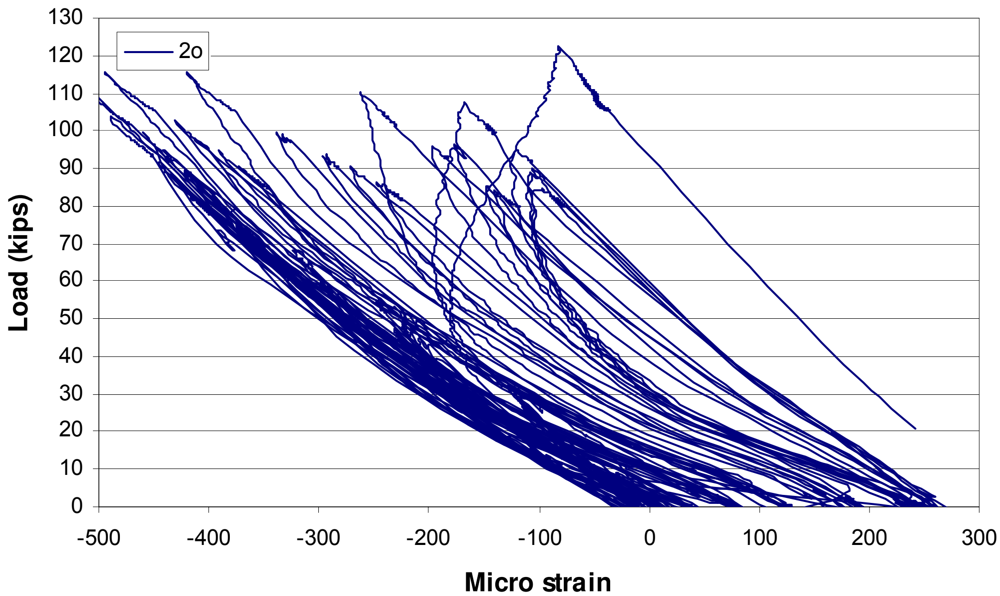

Figure A-14 Observed strain Test Cap 2 location o.

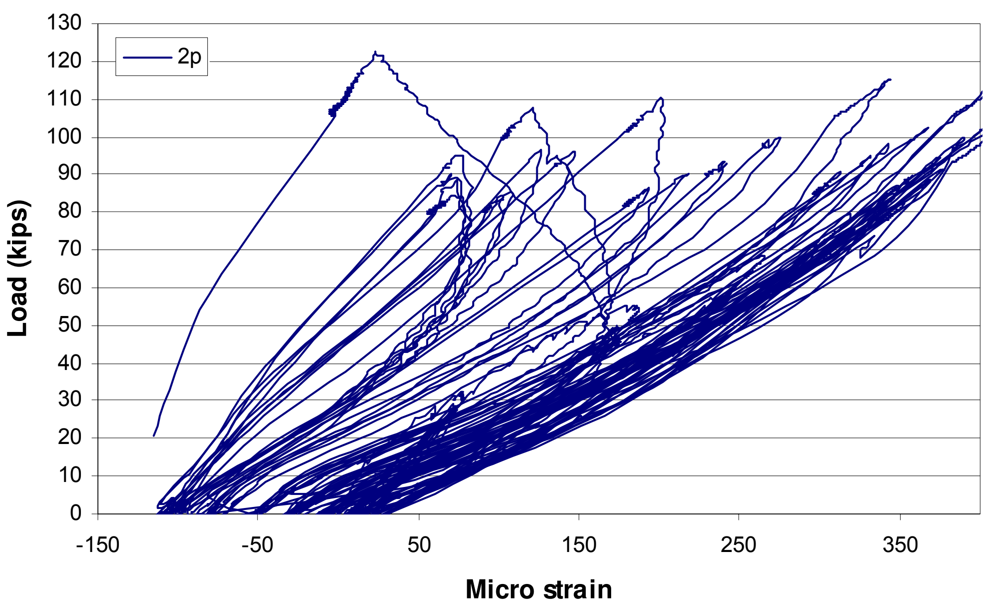

Figure A-15 Observed strain Test Cap 2 location p.

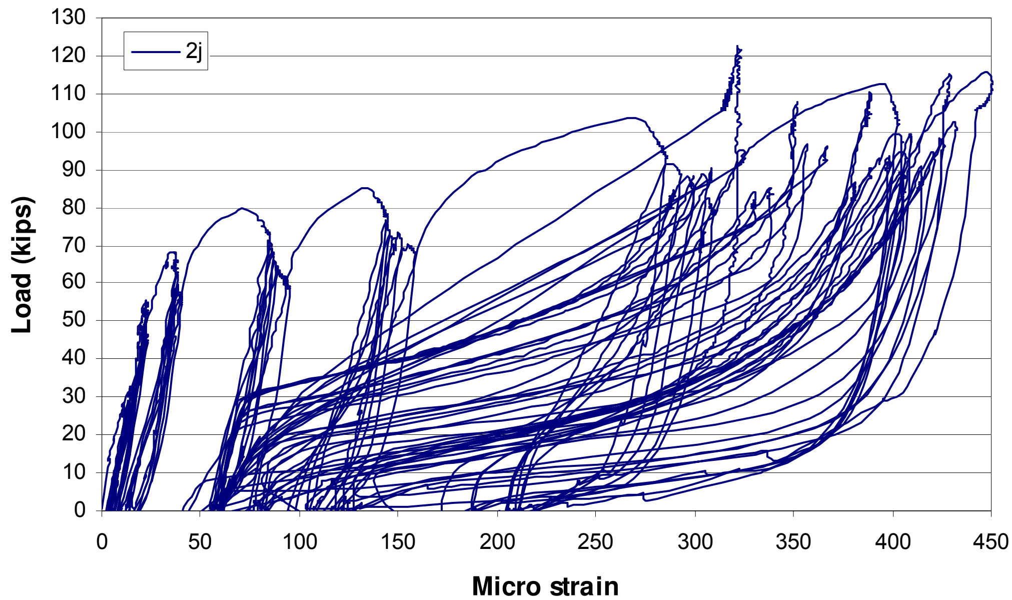

Figure A-16 Observed strain Test Cap 2 location j.

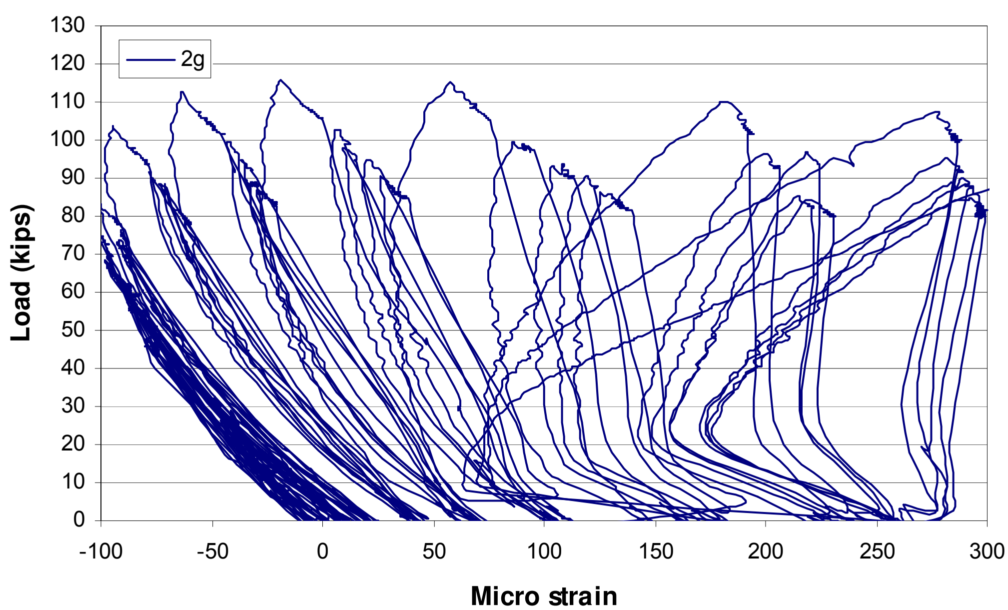

Figure A-17 Observed strain Test Cap 2 location g.

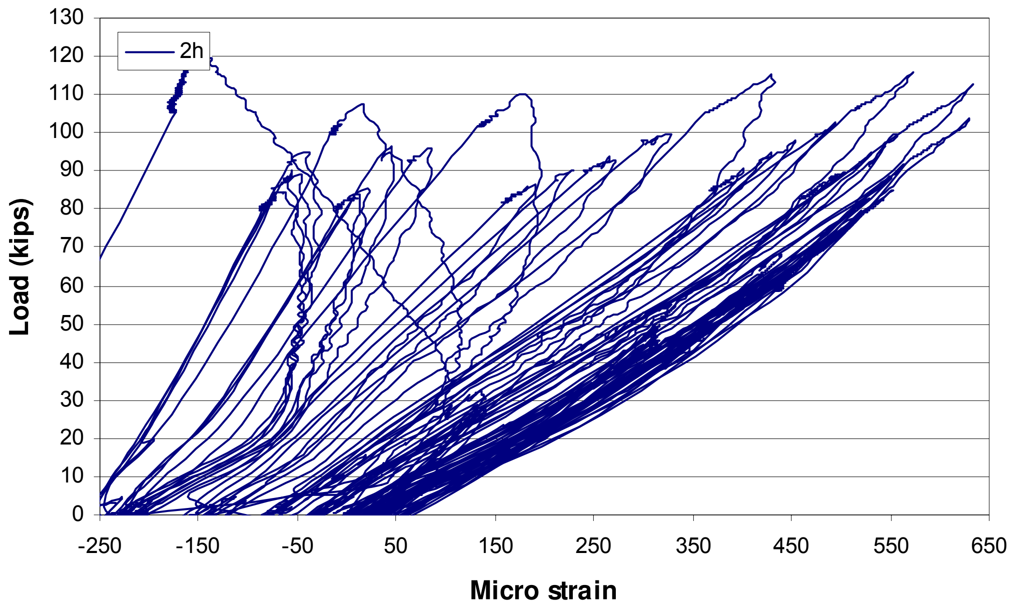

Figure A-18 Observed strain Test Cap 2 location h.

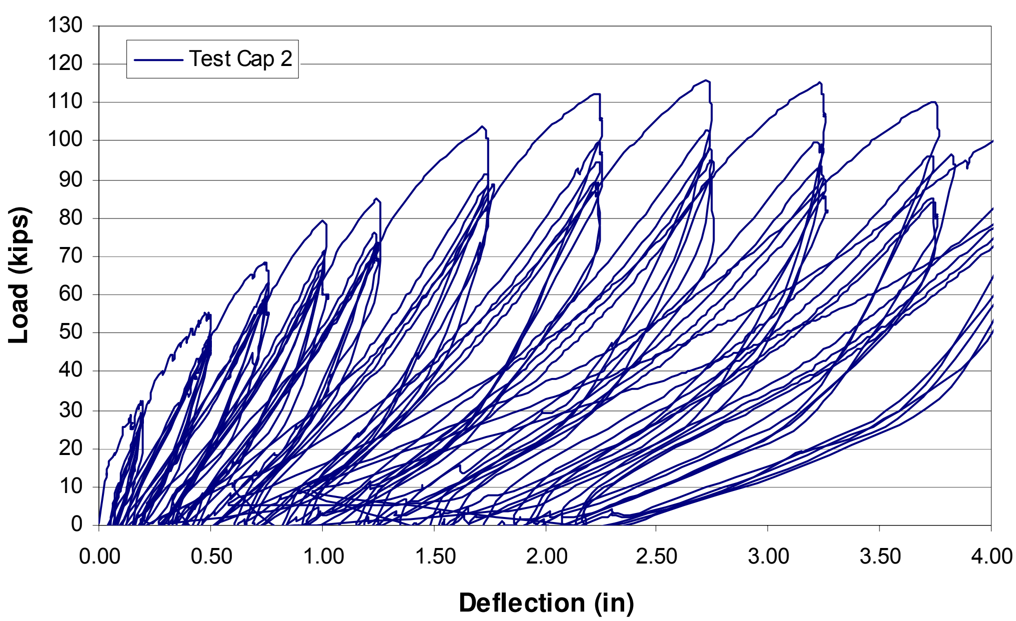

Figure A-19 Observed deflection Test Cap 2.

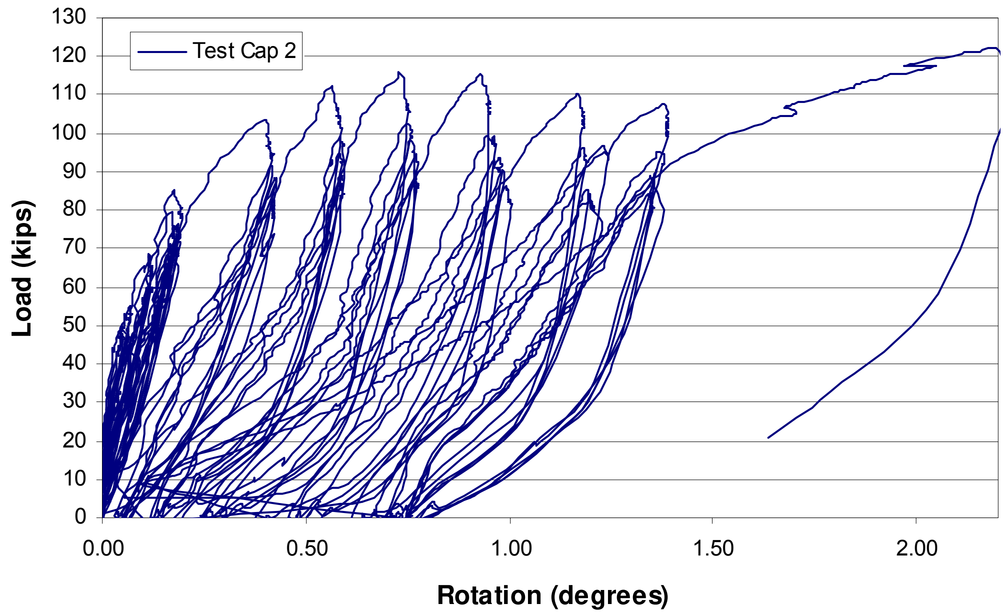

Figure A-20 Observed rotation Test Cap 2.

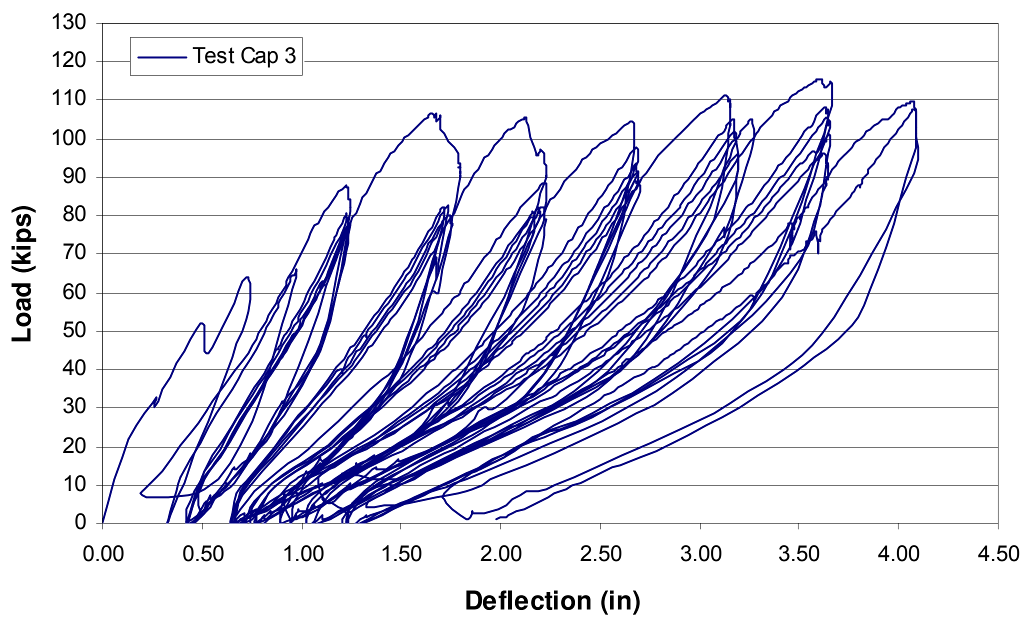

Figure A-21 Observed deflection Test Cap 3.

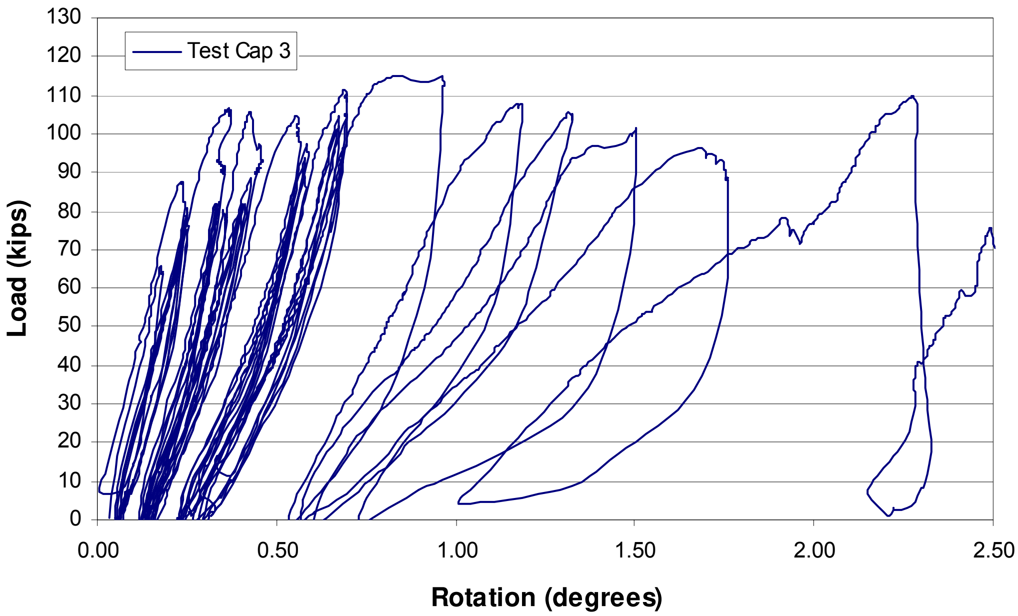

Figure A-22 Observed rotation Test Cap 3.

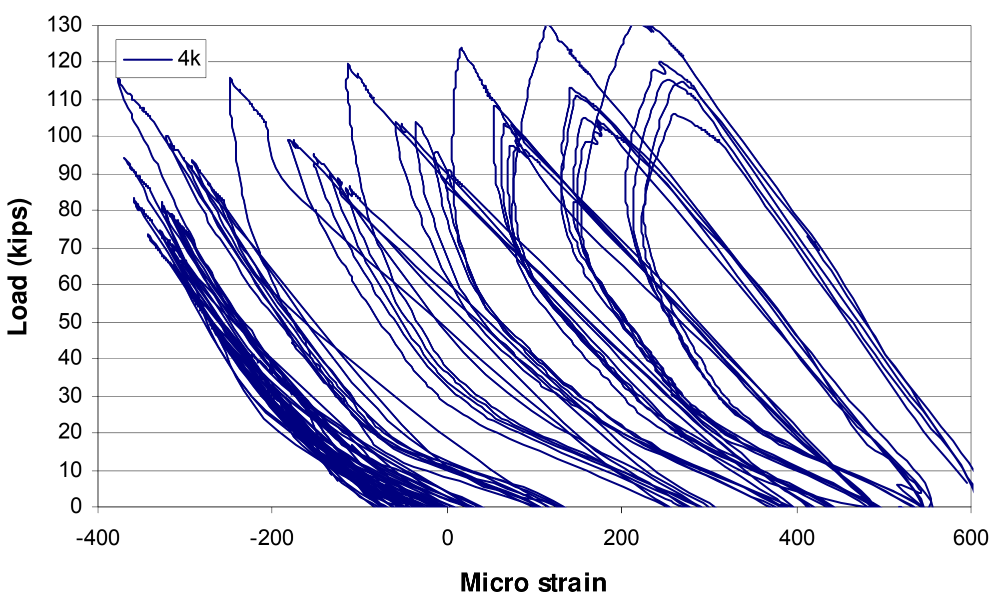

Figure A-23 Observed strain Test Cap 4 location k.

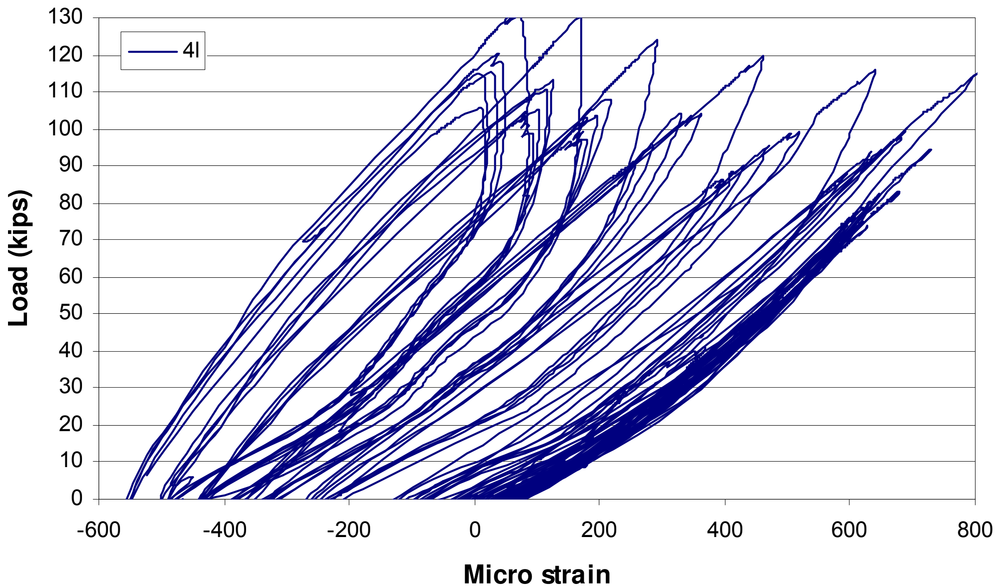

Figure A-24 Observed strain Test Cap 4 location l.

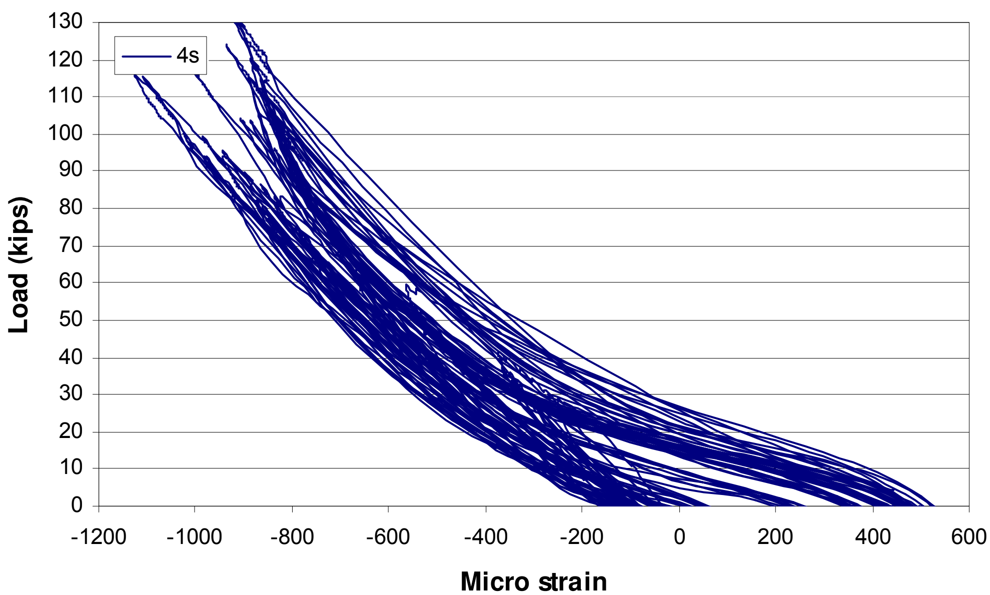

Figure A-25 Observed strain Test Cap 4 location s.

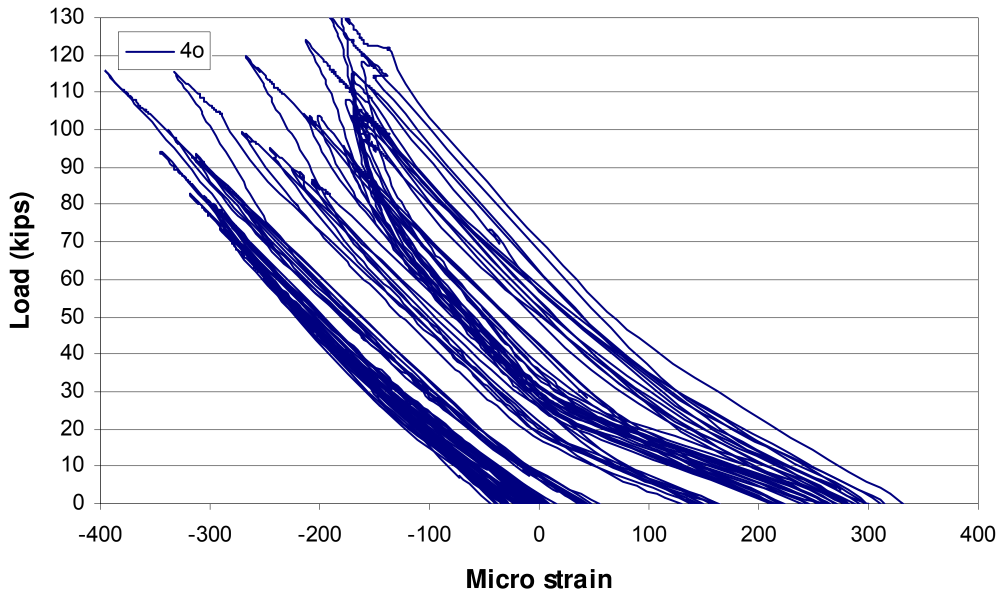

Figure A-26 Observed strain Test Cap 4 location o.

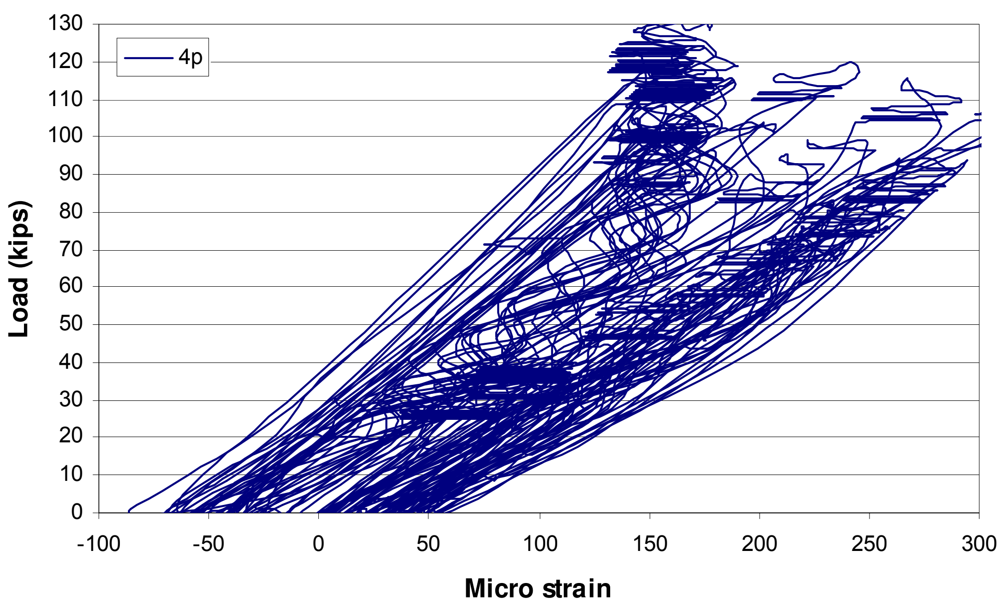

Figure A-27 Observed strain Test Cap 4 location p.

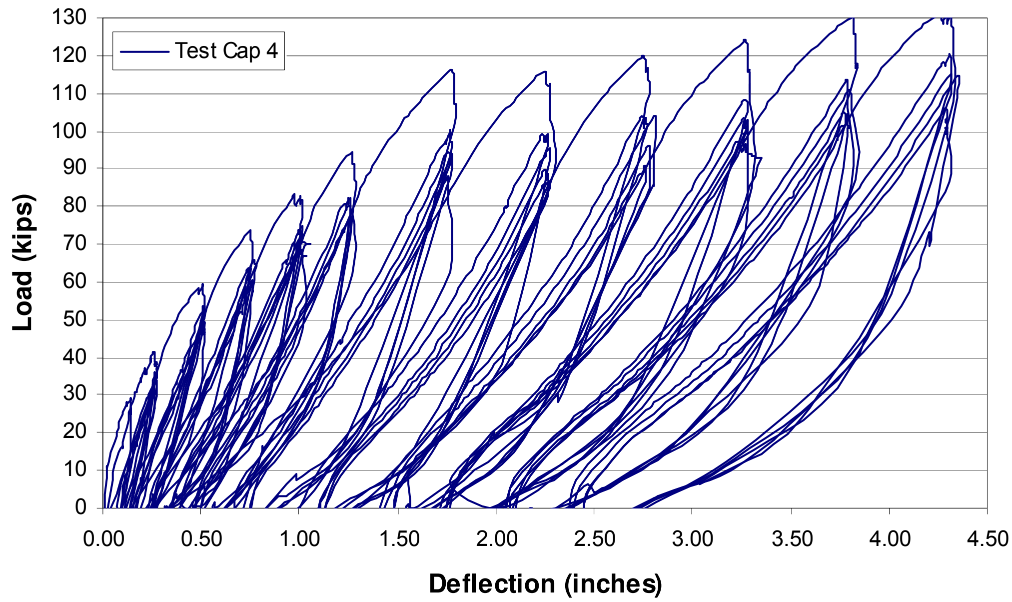

Figure A-28 Observed deflection Test Cap 4.

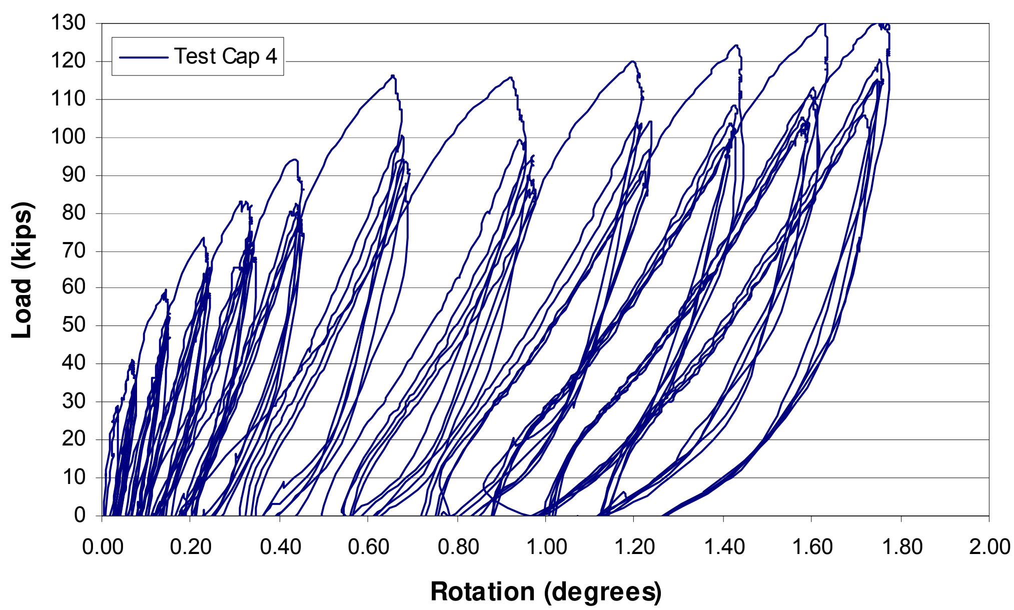

Figure A-29 Observed rotation Test Cap 4.