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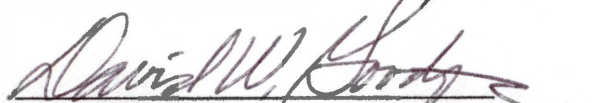


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**THE EFFECTS OF INTEGRATED EXPOSED STRANDS ON THE
BEHAVIOR OF PRESTRESSED CONCRETE PILES IN INTEGRAL
ABUTMENTS**

A Thesis

Presented for the

Master of Science

Degree

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Alan William Eichelman

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DEDICATION

I would like to dedicate this thesis to my parents and grandparents, Greg and Sarah Eichelman, William and Anne Eichelman, and William and Virginia Moreman.

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ABSTRACT

The Tennessee Department of Transportation (TDOT) has a history of designing and constructing integral abutments with prestressed concrete piles. Initially, the design of the integral abutments called for the prestressing strands to be cut along the top of the pile. The integration of the pile into the abutment was achieved by pouring the abutment around a length of the pile. This practice has been replaced. The current practice is to leave the prestressing strands intact above the pile and integrate them as well as a length of the pile in the abutment. Both types of integral abutments are still in use in Tennessee.

TDOT contracted The University of Tennessee, Knoxville (UTK) to research the behavior of concrete piles in integral abutments. This research provided an opportunity to compare abutments with and without exposed strands. Three integral abutments with prestressed concrete piles were tested. The strands were left exposed in two piles. Two theoretical models have been created to determine the effects of exposed strands. The first model determines the maximum stress that may be developed in a prestressing strand at a particular location based on the exposed strand length and the distance to the top of the pile. The second model predicts the axial load and moment capacity of the pile based on the prestressing strand stress, pile geometry, and material properties. Both models agree with the test data collected. The models show that axial load and moment capacity of the pile may be increased by changing the maximum stress in the strands. Similar increases in strand stress may be achieved by lengthening the pile embedment in the abutment or by lengthening the exposed strands, but the effects on the pile's axial load and moment capacity are not the same.

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NOMENCLATURE

A_{pile}	Cross sectional area of the pile (in ²)
A_{ps}	Cross sectional area of prestressing strand(s) (in ²)
$(A_{ps})_T$	Cross sectional area of prestressing strand(s) in tension (in ²)
C	Compressive force on the pile (kips per inch)
E_c	Concrete Modulus of Elasticity (ksi)
E_s	Steel Modulus of Elasticity (ksi)
F	Force acting on prestressing strand(s) (kips)
F_{ps}	Compressive force due to prestress on a pile under pure axial load (kips)
L	Generic term for length of prestressing strand(s) (in)
L_D	Development Length (in)
$L_{embedded}$	Embedded length of prestressing strand(s) (in)
$L_{exposed}$	Exposed length of prestressing strand(s) (in)
L_F	Flexural Length (in)
L_T	Transfer Length (in)
L_{total}	Total length of prestressing strand(s) (in)
M	Moment acting on the pile (kip-ft)
P	Total axial load on the pile (kips)
R^2	Coefficient of Determination
T	Tensile force in the prestressing strand(s) (kips)
V_c	Nominal strength of the section controlled by concrete (lb)
a	Depth of the compressive zone on the pile stress diagram (in)
a_1	Shear span (in)

b	Pile width (in)
b_1	Effective width of the compression block (in)
c	Depth of the compressive zone on the pile strain diagram (in)
d	Distance from the top of the pile to the center of gravity of the prestressing strand(s) in tension (in)
d_b	Diameter of a prestressing strand (in)
d'	Distance from the top of the pile to the center of gravity of the prestressing strand(s) in compression (in)
f	Generic term for stress (ksi)
f_c	Compressive stress in the pile due to initial prestressing (ksi)
f'_c	Concrete strength (psi)
f_{embedded}	Stress developed by the embedded strand(s) (ksi)
f_{exposed}	Stress developed by the exposed strand(s) (ksi)
f_{ps}	Strand ultimate stress based on pull out strength (ksi)
f_{pu}	Strand ultimate stress based on steel strength (ksi)
f_s	Stress in a prestressing strand (ksi)
f_{se}	Initial strand stress due to jacking, less losses (ksi)
f_{si}	Initial jacking stress (ksi)
ft	Foot
f_{total}	Stress developed by the entire length of prestressing strand(s) (ksi)
h	Total height of pile (in)
in	Inch
ksi	Kips per square inch
l_e	Embedment length (in)

m	Slope of a line in terms of $y = mx + b$ (kips per inch)
psi	Pounds per square inch
sq. in.	Square inches
β_1	Adjustment factor
β_2	0.85
Δ_{ct}	Limiting measured amount of slip (in)
$\Delta\epsilon_c$	Change in concrete strain
ϵ_c	Strain in the extreme concrete fiber
ϵ_{ce}	Compressive strain in the pile due to initial prestressing
ϵ_{cs}	Compressive strain in the extreme prestressing strand
ϵ_s	Strain in the prestressing strands or extreme steel fiber
ϵ_{se}	Initial strain in the prestressing strands
ϵ'_s	Adjusted strain in prestressing strands
σ_c	Compressive stress in the concrete (ksi)

Abbreviations

ACI	American Concrete Institute
LVDT	Linear Variable Displacement Transformers
PCI	Precast/Prestressed Concrete Institute
TDOT	Tennessee Department of Transportation
UTK	The University of Tennessee, Knoxville

CHAPTER 1

INTRODUCTION

The Tennessee Department of Transportation (TDOT) contracted with The University of Tennessee, Knoxville's Transportation Center to research the lateral load capacity of integral abutments. The contract specified two phases. Phase I researched integral abutments constructed with steel piles. Phase I began in 1996 and was completed in 1999 (1). Phase II of the contract involved prestressed concrete piles and began in 2000.

An integral abutment is an abutment designed to sustain both axial loads and horizontal loads. One benefit gained by designing a bridge with integral abutments is a reduction or complete elimination of expansion joints. The expansion joints are no longer required to relieve the horizontal loads created as the bridge undergoes thermal changes. Expansion joints account for a substantial portion of the total cost of building and maintaining a bridge. Therefore, the use of integral abutments is often an attractive design alternative. The state of Tennessee has been a leader in the use of integral abutments to create bridges with longer continuous spans (2). However, the length of continuous spans is limited. An integral abutment sustains horizontal and vertical loads as well as the moment induced by these loads. The magnitude of these loads is determined by the type of bridge, load conditions, and length of continuous bridge. Therefore, the length of the bridge with no joints is directly related to the abutment capacity.

TDOT has a long history of using prestressed concrete piles in integral abutments. Over the course of that history, two design techniques have been utilized to integrate the concrete pile and the abutment. In the past, when the pile was cut, the prestressing strands were cut as well. In this situation, a portion of the pile was integrated into the abutment. This design practice has been replaced. TDOT currently leaves the prestressing strands exposed above the top of the pile. The strands and the pile are then both integrated in the abutment. Both types of abutment are currently in use in Tennessee.

The objective of this thesis is to develop a model capable of predicting the behavior of an integral abutment based on the use of or lack of exposed, integrated strands. If a strand is capable of being fully developed, the use or lack of exposed strands becomes irrelevant to the pile's behavior. Therefore, this thesis is concerned with determining the location of the region of the pile where the strands are not fully developed and what the pile behavior is in this region. The model may then be used to determine what if any benefits arise from the use of exposed strands.

CHAPTER 2

LITERATURE REVIEW

2.1 Project History

The University of Tennessee, Knoxville (UTK) under a contract with TDOT, conducted research on the lateral load carrying capabilities of integral abutments. The first phase of the contract researched the behavior of integral abutments constructed with HP 10x42 steel piles driven approximately 38 feet on site. Five pile/abutment assemblies were tested over the course of three years. Three of the piles were driven in virgin soil and two in compacted fill. Four piles were embedded one foot in the abutment, and one foot of clear space was left between the abutment and the ground. The fifth pile was embedded 2 feet in the abutment. All of the piles were instrumented with strain gages along their length. The current practice in the state of Tennessee at the time of testing was to design integral abutments for 1 inch of lateral deflection. Consequently, great care was taken to examine the pile and abutment behavior in the region of 1 inch of deflection. Previous laboratory research had suggested that the concrete abutment would begin to fail before the entire assembly reached 1 inch of deflection, contrary to field experience. The test results gathered by UTK suggested that the pile and abutment assembly did not experience significant damage at deflections as great as 1.5 inches. The tests also demonstrated that the extra foot of pile embedment in the fifth test significantly increased the entire assembly's capacity. When pulled far enough, the abutment cracked and showed signs of failure. Specifically, diagonal cracks began to grow from the pile

flanges and limited the assembly's moment capacity. The cracks suggested a mechanism by which the pile and abutment interface could fail (1).

The second phase of research by UTK was concerned with the use of prestressed concrete piles in integral abutments. A point of particular interest was the behavior of the pile and abutment interface. The questions created by Phase I were whether or not the same type of damage would occur and what moment capacity could be developed at the pile and abutment interface.

Phase II tested four prestressed concrete piles and their abutments. The initial tests on Pile 1 revealed several areas for improvement in the instrumentation and test set-up. The piles were driven approximately 36 feet on site. As in Phase I, the abutments were cast one foot above grade with one foot of pile embedded in the abutment. The prestressing strands in the pile were left exposed 3 feet above the pile top and embedded in the abutment in three of the assemblies. The piles in Phase II were not instrumented along their length as in Phase I. Instead, strain gages were used only on the 1 foot of exposed pile above grade and below the abutment. Phase I used a reaction beam to prevent abutment rotation. Phase II used hold down beams to prevent abutment rotation. However, initially the moment induced by the eccentricity of the beams with relation to the pile was not countered, causing a moment on the pile at all times. The solution was to align the ballast on top of the abutment to create zero moment in the pile when tests were not in progress. Data for Phase II was collected with Linear Variable Displacement

Transformers, strain gages, DEMEC gages, and load cells. All of the electronic data was collected with a MEGADAC and Test Control Software (3, pp. 7-29).

Three different configurations were used for the hold-down assembly. The purpose of the assembly was to simulate the stiffness of a bridge deck and limit pile rotation. Two configurations were tried during the testing of Pile 1. A third configuration that most closely simulated actual bridge stiffness was used for Pile 2 and the remainder of the tests (4), and is discussed in chapter 4.

2.2 Research History

Currently, the use of integral abutments in bridge design is considered a viable solution to the problems associated with the use of expansion joints. The reduction in maintenance required for integral abutments as opposed to expansion joints provides a substantial cost saving over the course of a structure's life. Expansion joints complicate future renovations or additions to structures and do not always perform as designed. Studies have found that bridges designed with expansion joints still generate horizontal loads capable of causing measurable deflections on their abutments. The majority of state Departments of Transportation design a portion of their abutments as integral abutments. TDOT designs both semi-integral and integral abutments but prefers to use integral abutments (2).

The University of South Carolina has researched the ability of prestressed concrete piles embedded in concrete abutments to resist horizontal loads. In particular,

the focus of the research was the ability of “bare” concrete piles to develop full moment capacity when embedded in an abutment. The researchers defined “bare” pile as a pile that has not been altered in any way to enhance mechanical or chemical bonding. The researchers also defined full moment capacity as the maximum moment the pile may develop at a particular location along the pile. Therefore, by definition a pile may be able to develop its full moment capacity based on the embedment length in the abutment but only a portion of the pile’s maximum moment capacity when the prestressing strands are fully developed. The researchers concluded that “bare” piles embedded in an abutment could develop the piles’ full capacity. The required embedment length may be defined by either the equations for the embedment length of steel haunches in the Precast/Prestressed Concrete Institute (PCI) manual or as the width of the pile top. Twelve inches is suggested as an absolute minimum (5). PCI defines the embedment length of steel haunches with equation 6.9.1:

$$V_c = \frac{0.85 * f'_c * b_1 * l_e}{1 + \frac{3.6 * e}{l_e}} \quad (1)$$

$$e = a_1 + \frac{l_e}{2} \quad (2)$$

In equation 6.9.1, f'_c is defined as the concrete strength, b_1 as the effective width of the compression block, l_e as the embedment length of the haunch, and a_1 as the shear span. When reinforcement ties are spaced less than 3 inches apart, it is suggested that b_1 be defined by the smaller of the distance from outside to outside of the ties or 2.5 times the width of the steel section (6, pp. 6-27).

Several tests concerned with the ability of prestressed concrete piles and their pile caps to resist horizontal loads have been performed in New Zealand. In particular, six tests were performed by Joen and Park using octagonal prestressed concrete piles. The research focused on the pile behavior and the capacity of the pile and pile cap connection. Two of the piles (1, 2) were roughened to facilitate mechanical bonding and embedded 31.5 inches in the abutment. Three piles (3, 4, 5) were embedded 2 inches in the abutment, but 23.6 inches of prestressing strand and spiral reinforcing were also integrated in the abutment. The final pile (6) utilized four 0.79 inch diameter dowels placed along the top of the pile and anchored by 90 degree standard hooks in the abutment. The piles were then tested under horizontal loads in conjunction with an axial load of $0.2 \cdot f'_c \cdot A_g$, where f'_c is the concrete compressive strength and A_g is the gross area of the pile. The maximum flexure sustained by the pile was compared to four theoretical models. Two of the models were based on American Concrete Institute (ACI) provisions. Model A assumed fully developed strands. Model B accounted for the possibility of non-fully developed strands. Models C and D used a concrete stress versus strain diagram specific to spirally reinforced concrete. Model C assumed fully developed strands, and Model D accounted for the possibility of non-fully developed strands. The conclusions reached were that all of the models were valid and that either embedment of the pile or exposed strands were valid connection details. The test results exceeded the theoretical results by the percentages listed in Table 1 (7).

Table 1: Joen and Park Test Results

		Model	
		A	B
Test	1	20%	22%
	2	21%	22%
	3	19%	19%
	4	21%	21%
	5	27%	27%

2.3 Prestressing Strand Capacity

Chong Key Chew conducted extensive research into the adequacy of ACI 318-83 relative to transfer and development lengths of prestressing strands. In particular, he addressed the question of bond development of seven wire prestressing strands. Based on his research, he was able to provide guidelines for the determination of strand end slip during testing and when the strands were suddenly cut. Several factors affected the strand's maximum load capacity before slipping began. They included strand diameter and surface condition and the concrete strength. The determination of when a strand would slip during testing was at best an approximation as was the magnitude of the slip. Instead, slip in the strands was identified by close inspection of P- Δ curves. An equation for approximating the strand end slip when cut was provided.

$$\Delta_{ct} = \frac{f_{si} * d_b}{950} \quad (3)$$

This equation is only an approximation and based on the initial jacking stress, f_{si} and the strand diameter, d_b . However, test data supported the conclusion that the strands which slipped the most during cutting were the first to slip during testing. Sudden release of the prestressing force in a strand also had the potential to increase the transfer length of the strand. Chew found that in flexure the strands might slip by relatively small amounts

without lessening the pile's capacity, provided the strands did not slip simultaneously. Simultaneous slips when the pile was in flexure led to a complete loss of load carrying capability (8, pp. 27-29)

Sudden cutting of prestressing strands and other factors have been shown to affect the transfer length of the strands. The question then arises as to whether the transfer length equations provided by ACI are adequate. Research conducted in Florida suggested that large abutments cast around a prestressed concrete pile provided a "clamping" or confinement force on the pile. This force negated the need for any multipliers to be applied to the existing transfer length equations. When a pile was confined by a cast in place abutment, the ACI code provisions were adequate. A suggested rule of thumb was 100 times the diameter of the strand of development length was necessary to fully develop a strand (9 and 10).

Extensive research was carried out at UTK to determine the pull out strength of unstressed, prestressing strands. The testing found that the relationship between strand embedment depth and developed stress in the strand was linear regardless of strand diameter. The study determined that strand surface condition was very influential in the strand's maximum stress as was concrete strength to a lesser degree. The study also found that sudden release of the prestressing force in a strand might increase the transfer length (11, pp. 32-33)

CHAPTER 3

THEORY

3.1 Introduction

Piles subjected to both horizontal and vertical loading can develop large moments. Depending on the magnitude of the loads, the moments might be large enough to cause cracking or failure in a prestressed concrete pile. Three such piles, Piles 1, 2, and 3, tested by UTK for TDOT, developed cracks and, in the case of Pile 2, ultimately failed at a large deflection when subjected to horizontal loads and their associated moments. The piles developed horizontal cracks and crushing failure immediately below the abutment. Pile 2 also developed cracking and a failure several feet below the bottom of the abutment (Appendix 7). In order to understand how prestressed concrete piles resist moments and what happens as they crack and eventually fail, a theoretical understanding of pile behavior is required. This thesis explores the pile's behavior based on a model of the pile and its associated interaction diagram. The interaction diagram provides a graphical relationship between the maximum axial load and maximum moment the pile is capable of sustaining. It requires knowledge of the pile geometry, material characteristics, and the stresses created in the prestressing strands as a moment is applied. It is assumed that pile geometry and material characteristics will be known in any situation. Therefore, a model must deal primarily with the stresses created in the prestressing strands as a moment is applied. In particular, a model must look at the manner in which different strand arrangements develop stress. The model then can

utilize these stresses and the pile characteristics to generate an interaction diagram which will allow test results to be compared with theoretical predictions.

3.2 Definitions

Before a theoretical model can be developed, several terms that will be used in the discussion must be defined. The pile and abutment interface is the horizontal plane created by the bottom of the abutment, which passes through the pile. The embedded length of the prestressing strand is the length of strand between the pile top and the horizontal plane of the pile being analyzed. The exposed length of the prestressing strand is the length of strand left uncut above the pile. The sum of the embedded and exposed length is referred to as the total strand length. The differentiation between exposed and embedded strand is important, because the strand develops stress in two stages that correspond to the two different lengths. Any steel discussed in this thesis behaves according to the stress versus strain relationship in Figure 1.

3.3 Embedded Strands

Prestressing strands develop stress in two distinct stages. The first stage is controlled by the embedded length of strand. The embedded strand develops its maximum allowable stress regardless of the exposed strand length, because the exposed strand is not placed under stress until the embedded strand pulls on the exposed strand's end. As long as the maximum stress at any point along the embedded strand is equal to or less than the maximum stress capable of being developed by the embedded strand

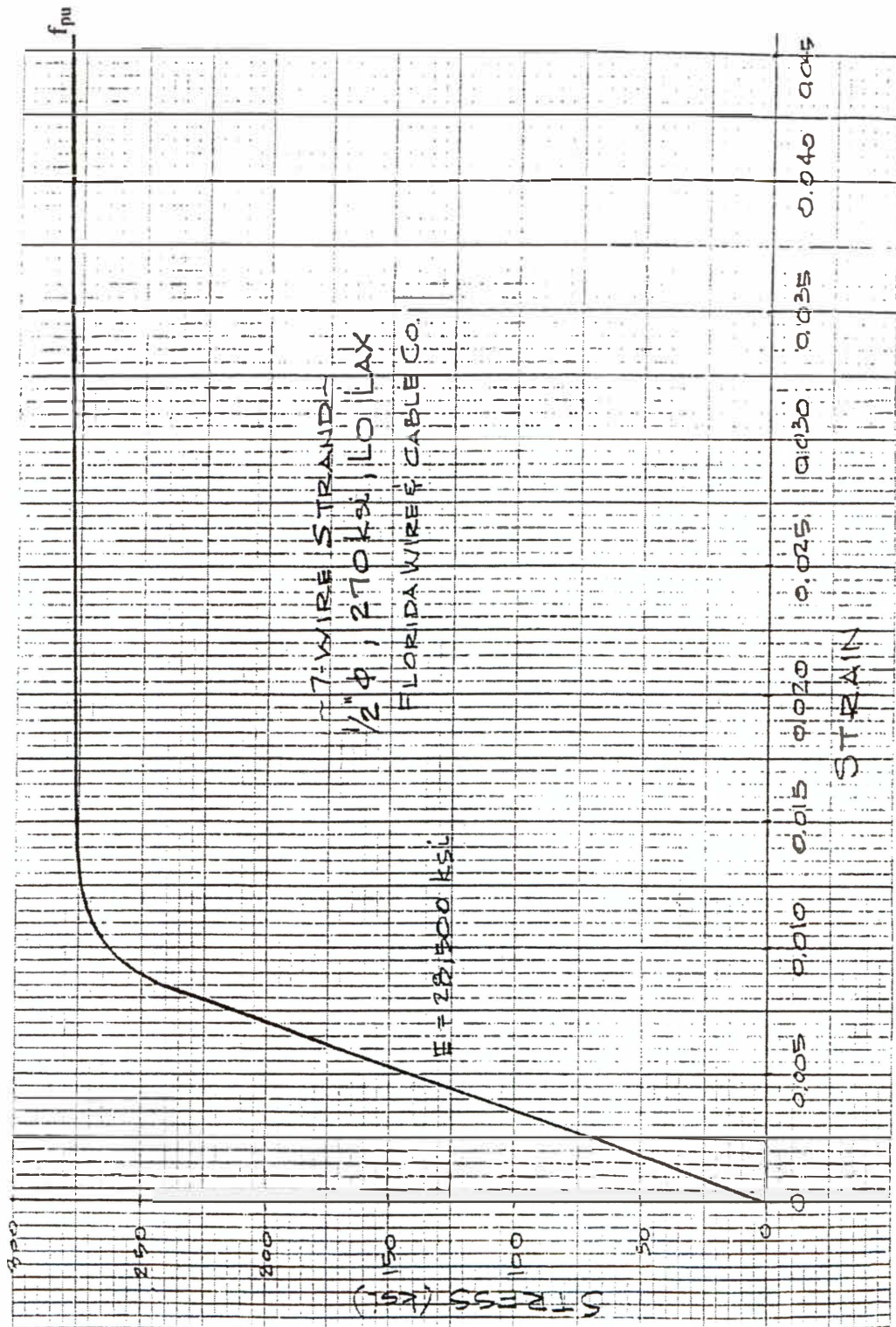


Figure 1: Steel Stress versus Strain Curve (14)

alone, the stress and therefore pull at the end of the embedded strand and beginning of the exposed strand is zero (Figure 2). By modeling the exposed strand stress as zero until the embedded strand is fully developed, the pile may be idealized as having an exposed strand length of zero. ACI and PCI both agree on a method for determining embedded strand stress if the exposed length of strand is zero. The maximum stress developable in the embedded strand is based on the distance from the free end. The free end is defined as the top of the pile. Figure 2 is based on both ACI and PCI. It shows that as the embedded length of strand increases, the strand stress will increase in two stages. The first stage is a linear increase to the strand stress after losses (f_{se}) over the Transfer Length (L_T) (6, p. 4-27 and 12, p. 192). f_{se} is defined for prestressed members as the prestress induced by jacking less losses. Losses result from creep, relaxation, and shrinkage (13). L_T is the length of strand, measured from the free end, necessary to develop f_{se} . ACI and PCI provide the following equation for L_T (6, p. 4-27 and 12, p. 192).

$$L_T = \left(\frac{f_{se}}{3} \right) * d_b \quad (4)$$

f_{se} is determined solely by the embedded strands. f_{se} is not reduced or increased by the presence of exposed strands, because f_{se} is determined by the ability of the concrete pile to sustain the prestress in the strand (13). The pile requires L_T of embedded strand in order for the strand to develop f_{se} . When the pile is cut or crushed in order to reveal the exposed strands and create a new pile top, the maximum f_{se} developable in the pile at any distance from the pile top less than L_T is redefined (6, p. 4-27 and 12, p. 192).

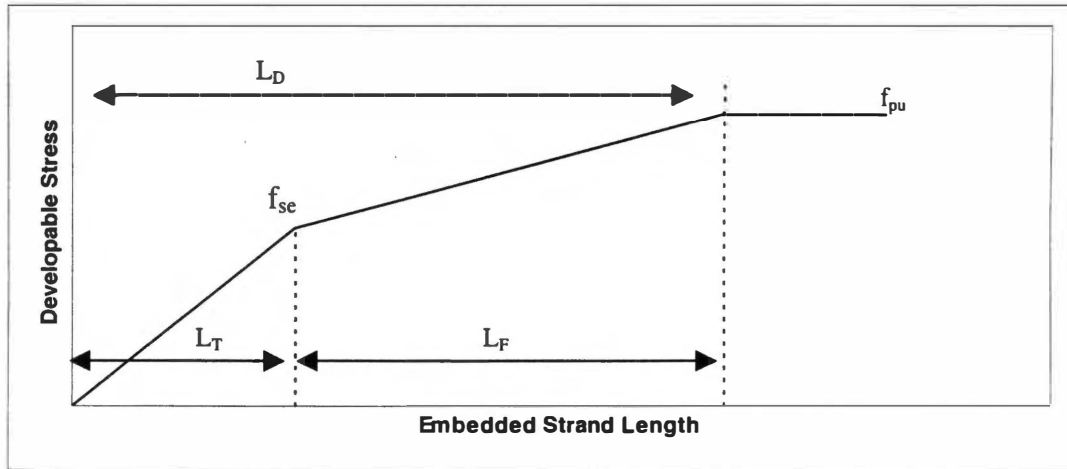


Figure 2: Strand Development Diagram (6, p. 4-27 and 12, p. 192)

The second stage is a nonlinear increase from f_{se} to the strand's ultimate stress (f_{pu}) over the flexural length (L_F). L_F is defined as the length of strand beyond L_T necessary for f_{pu} to be developed. ACI and PCI provide the following equation for L_F .

$$L_F = (f_{pu} - f_{se}) * d_b \quad (5)$$

The sum of L_F and L_T is the Development Length (L_D). Any embedded strand longer than L_D develops f_{pu} (6, p. 4-27 and 12, p. 192). For the purposes of this model, the second increase in stress over L_F is idealized as linear, and the strand stress beyond L_D is considered constant (Figure 2). The tri-linear curve allows the use of a mathematical model which is capable of defining the stress developed at any point in the embedded length of strand. The equations are conservative. The assumption of constant stress beyond L_D assumes that the steel is “flat-top” steel. This means that the steel reaches a maximum stress at the yield point and maintains that stress as the strain is increased until failure.

$$L \leq L_T \quad f = L * \frac{f_{se}}{\frac{f_{se}}{3} * d_b} \quad (6)$$

$$L_T < L < L_D \quad f = f_{se} + \left[(L - L_T) * \frac{f_{pu} - f_{se}}{(f_{pu} - f_{se}) * d_b} \right] \quad (7)$$

$$L \geq L_D \quad f = f_{pu} \quad (8)$$

Equations 6 through 8 define the stress in the embedded length of strand, where L is measured from the free end of the pile. If L is insufficient to develop the necessary stress in the prestressing strand to sustain the applied moment, then a failure occurs unless a

second means of carrying load is available. If exposed strands are present, the second stage of strand development will occur and allow the larger loads to be carried.

3.4 Exposed Strands

The second stage of strand development is applicable when the embedded length of prestressing strand has developed its maximum stress, but a larger stress is necessary to resist the applied moment. Figure 3 is an example of a free-body diagram of a pile with both embedded and exposed strands and an applied moment. The moment is represented by the tensile force in the strand and the compressive force in the concrete necessary to create an equilibrium couple. When the tensile stress due to the moment is greater than the maximum embedded strand stress, the difference is resisted by the exposed length of strand. The exposed length of strand develops stress differently than the embedded length. While the embedded length of strand is subjected to a maximum stress of f_{se} when the applied moment is zero, the exposed length of strand only develops a stress when necessary to maintain equilibrium in the system. If the moment is small enough that the embedded length of the strand may hold, the exposed length of strand will not develop stress. If the embedded length of strand is not strong enough, the exposed strand will develop the necessary stress up to its maximum capacity. The capacity of the exposed strand is defined by Griffin and dependent primarily on the length of strand. The relationship between exposed strand length (L) and maximum stress before failure (f) is linear (11, p. 27).

$$f = \frac{L - 2.26}{0.198} \quad (9)$$

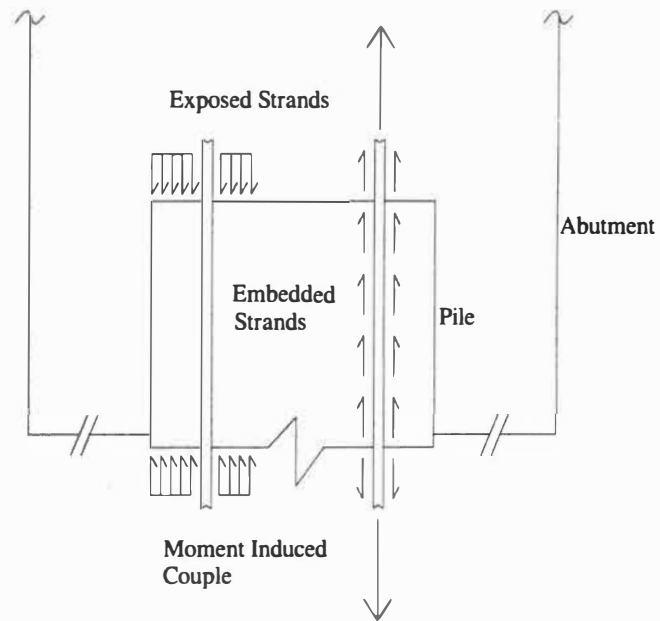


Figure 3: Pile and Abutment Interface Free Body Diagram

When the exposed length of strand is under stress, the net stress developed at any point in a strand is the sum of the embedded strand stress and the stress being developed in the exposed length. In this situation, L_T and L_F must be used with care. The stress in the embedded strand is developed as if the exposed length was zero. The exposed strands then develop stress. In conjunction with the exposed strands, the stress in the embedded length of strand may reach f_{se} or f_{pu} before L_T or even L_D is reached. However, the maximum stress at any point in the prestressing strand is always f_{pu} (Figure 4).

3.5 Interaction Diagram Assumptions

When the maximum developable stress in the strands is known, an interaction diagram can be created for a pile. The interaction diagram is based on several assumptions illustrated in Figures 5 and 6. In bending, plane cross-sections of the pile remain plane. This allows the assumption of a linear strain relationship in the pile cross section (Figure 5). The stress versus strain relationship for the concrete in an assumed elastic range may be approximated by a single Modulus of Elasticity (E_c), and all concrete fibers have the same E_c . For the purpose of moment calculations, the compression zone in the pile is assumed to be a uniform stress (Figure 5) (14 and 15). These assumptions are necessary regardless of whether or not the pile is prestressed. Because the model is specific to prestressed piles, the method used to create the interaction diagram requires further refinement. While nonprestressed reinforcement in a pile may carry compressive forces, prestressing tendons will never

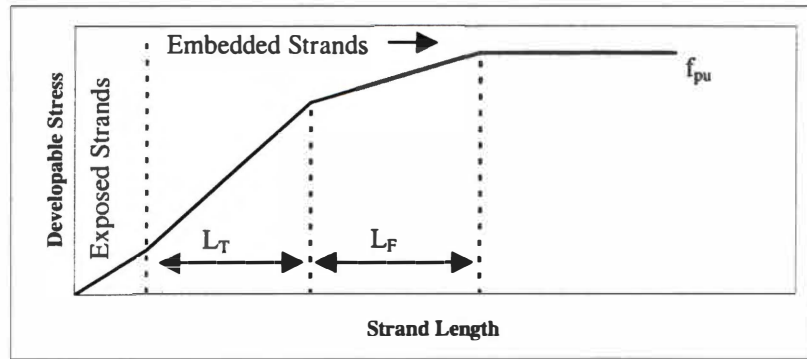


Figure 4: Theoretical Strand Development Diagram

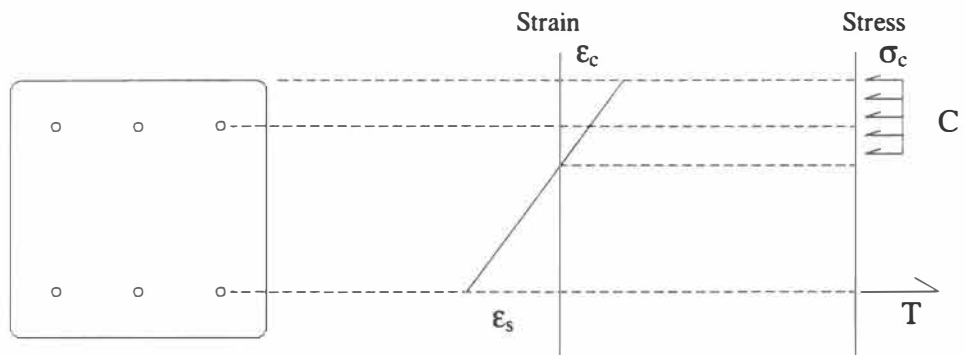


Figure 5: Pile Stress versus Strain Relationship (14)

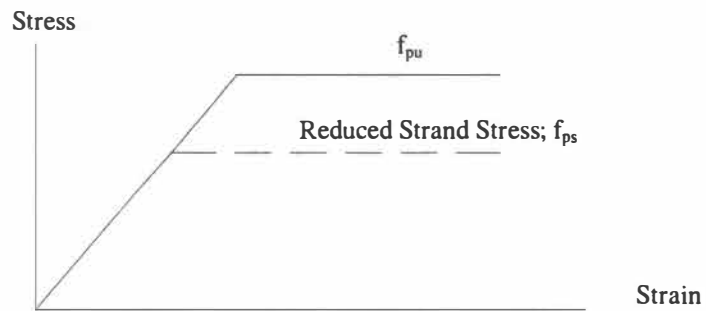


Figure 6: Strand Stress versus Strain Relationship

carry a compressive force. Instead, as a compressive load is applied, the resulting strain is removed from the strain in the tendon created by f_{se} . If the reduction in strain is greater than the initial strain in the strand, the strand's strain is reduced to zero and no farther. All steel is assumed to have a "flat-top" stress versus strain diagram once the elastic limit is reached (Figure 6) (14 and 15). This assumption does not allow the steel's maximum stress to continue to increase after the yield point is reached and is conservative. As mentioned earlier, it simplifies the mathematical model associated with the interaction diagram. However, this assumption must be tested against each new type of steel used because each steel's stress versus strain diagram is different.

When a strand is unable to develop f_{pu} , the reduced maximum stress capable of being developed (f_{ps}) is considered to represent a new yield point and "flat-top" on the stress versus strain diagram (Figure 6). The strand's Modulus of Elasticity (E_s) remains constant, but the maximum stress and hence the "flat-top" is adjusted accordingly. These assumptions and refinements allow an accurate interaction diagram to be created.

3.6 Interaction Diagram Development

The pile interaction diagram is based on an analysis of the stresses and strains acting on a single horizontal plane of the pile. The surface is representative of the pile geometry and not associated with a particular position (14 and 15). However, the maximum strand stress is associated with a position in the pile. Using the pile geometry, material properties, and maximum strand stress, an analysis of the pile's moment and axial load carrying capabilities is performed. The analysis is based on the strain in the

concrete and prestressing strands created by a particular moment and axial load. The concrete compressive strain at the extreme fiber (ϵ_c) is set at 0.003, which is the maximum strain the concrete may experience before flexural failure according to the ACI code. The ACI code uses 0.003 based on experimental stress versus strain curves and the fact that 0.003 allows the largest moment in the member (12, p. 108). In order to determine the axial capacity of the pile at different moments, the tensile strain in the extreme prestressing strands (ϵ_s) is altered (14). Due to assumptions already made, the strain in the pile between the extreme concrete fiber in compression and the extreme prestressed fiber in tension is linear (Figure 5). New compressive or tensile strains in a prestressing strand are respectively subtracted from or added to the initial prestressing strain in the strand. The new strains in the strands created by the strain are limited by the maximum stress developable before failure (f_{ps}). The compressive strain in the concrete is converted to a rectangular block of constant stress according to ACI practice (Figure 5). The moment in the pile due to the forces is the maximum moment at a specific axial load. The axial load is the sum of the loads (14). When the data for several steel strains are plotted, the interaction diagram is developed. In a nonprestressed pile, the interaction diagram is valid for any section of the pile. However, in this model, the pile is prestressed, and the maximum developed stress in the strand is dependent on its location in the pile. Therefore, the interaction diagram is specific to a unique location in the pile. For instance, the interaction diagram for a position in the pile at which the strands may only develop half of f_{pu} is significantly different from that of a position where the strands may develop full f_{pu} . Unless the interaction diagram is based on fully developed strands, the diagrams are unique to a location in the pile.

3.7 Strand Slip

The model does not account for strand slippage. The slip when the strands are cut along the pile top may be approximated by the following equation (8, p. 27).

$$\Delta_{ct} = \frac{f_{si} * d_b}{950} \quad (3)$$

Equation 3 defines a maximum possible value for slip during a sudden strand cut. Not all of the strands will slip equally. The amount of slip is controlled by several factors that are not considered in this model. They include mechanical and bond strength of the strands, strand surface conditions, concrete strength, and quantity of concrete slurry around the strands. The strand that slips the most is expected to be the first to fail (8, p. 27). Chew also discusses strand slip during testing. He defines two types of slip, small and large slip. A relatively large slip under flexural load cannot be accommodated by the pile. The slip causes the pile's compressive zone to shrink, leading to a compressive failure. The magnitude of slip necessary for this scenario is dependent on the pile and not defined by Chew. The second type of slip is small or "slight" slip. When the concrete bonded to the strand maintains a high stress, creep can occur. This allows a small slip. These slips quickly reach a new equilibrium with only a minor loss in the load-carrying capability of the pile. As long as the slips are not simultaneous, the load capacity of the pile is relatively unchanged. However, if the slips do occur simultaneously, the load capacity of the pile begins to fall until ultimate failure occurs (8, pp. 85-87). A quantitative method of analyzing the slips and their effects is not provided. Without such a method, the interaction diagrams will not account for slip.

CHAPTER 4

TEST SET-UP

The second phase of research undertaken for TDOT was concerned with prestressed concrete piles. The piles were 14 inches square with six 0.5 inch diameter, 7 strand, grade 270, low relaxation prestressing strands. The strands were grouped by three in line on opposite sides of the pile such that during tests they lay along the tension and compression faces. Each strand had a minimum of 2.5 inches of cover to the center of the strand (Figure 7). The piles were 40 feet long, composed of 6000 psi concrete, and driven in “virgin” soil approximately 36 feet at the research site. One foot above grade, a single abutment was poured on each pile (Figure 8). The abutment was 3 feet wide in the direction of horizontal loading, 10 feet long, and 2 feet tall. The pile was embedded 1 foot into the abutment. In some cases, 3 feet of prestressing strand were exposed above the pile cut. Above the abutment, a concrete slab was poured. The slab was 2.5 feet thick, 10 feet wide, and 14 feet long in the direction of horizontal loading.

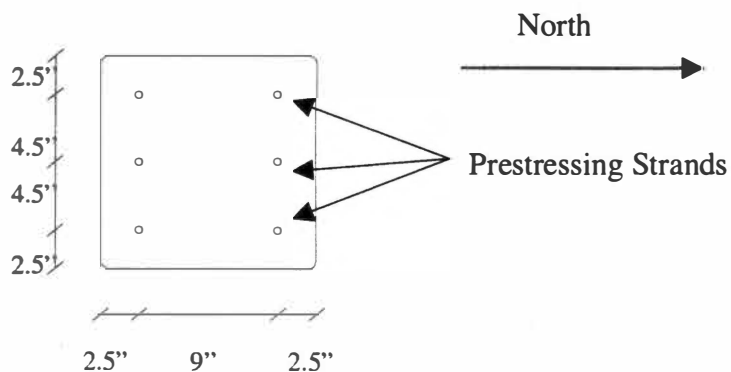


Figure 7: Pile Geometry

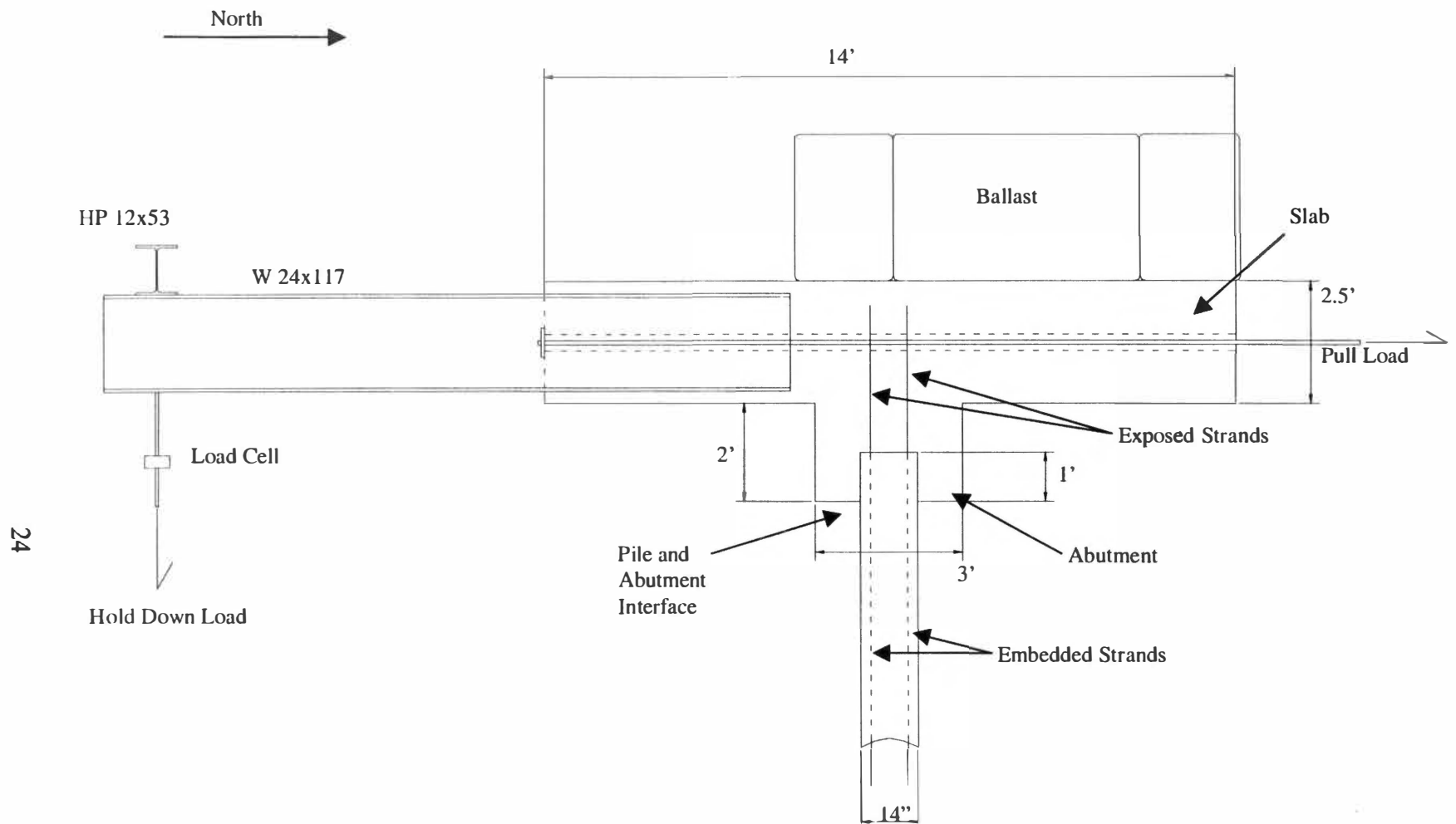


Figure 8: Test Assembly

The slab and abutment were cast monolithically using a single pour and shared reinforcement steel. The concrete used was TDOT Grade A, minimum 28 day strength 3000 psi. A hollow steel pipe running the 14 foot length and centered both vertically and horizontally was embedded in the slab. A 1 inch diameter Dywidag bar was passed through the tube and used to apply the horizontal load to the pile. Two W24x117 steel beams were secured to the east and west sides of the slab. They extended in the opposite direction of loading, either north or south. These two beams were spanned at their tips by a third HP12x53 beam. This beam supported a load cell connecting the beam to a concrete pad. The load cell measured the vertical, axial load used to counter rotation in the assembly during testing. The eccentricity in dead load created by the steel beams was offset by seven 4,500 pound concrete ballast blocks appropriately located on top of the slab. The entire assembly placed zero moment on Piles 2 and 3 until horizontal and vertical loads were applied during testing (3, pp. 7-19 and 4). The purpose of the beams, load cell, ballast, and slab was to simulate an actual bridge deck's stiffness (4). Once a test was begun, the moment at the pile and abutment interface was calculated based on the horizontal and vertical loads. The horizontal and vertical deflections were measured with Linear Variable Displacement Transformers (LVDT's). Two LVDT's were used to measure the horizontal displacement of the pile. One LVDT was placed immediately below the bottom of the abutment and referred to as the Pile Top LVDT. A second LVDT was placed 1 foot below the pile and abutment interface and referred to as the Ground LVDT. Unless otherwise stated, all deflections are based on the Ground LVDT. Concrete crushing below the pile and abutment interface rendered the Pile Top LVDT useless during some tests. The strain in the pile tension and compression faces between

the ground and the pile and abutment interface was measured with foil strain gages and DEMEC points. This geometry was oriented such that both piles were pulled north and south (3, pp. 7-19 and 4).

CHAPTER 5

TEST RESULTS

5.1 Introduction

The research on concrete piles involved the repeated testing of three piles and the tests to failure of two. All piles conformed to the geometry and material properties discussed in chapter 4 and illustrated in Figures 7 and 8 with two exceptions. Pile 1 used two different hold down assemblies. Piles 1 and 2 were constructed with 1 foot of embedded strand in the pile and 3 feet of exposed strand integrated in the abutment. Pile 3 was constructed with only 1 foot of embedded strand in the pile. Construction of the piles in two different manners was meant to simulate two design and construction techniques TDOT has used. Both types of design are in use in Tennessee. Because the single difference in these two techniques lies in the presence or absence of exposed strands, the piles provide an excellent opportunity to verify the theoretical model. The interaction diagrams for each pile configuration should accurately predict the test results. The maximum loads sustained by the pile before the pile begins to increase in its deflection without an associated load increase should lie on the interaction curve. All loads less than these maximum values should lie within the curve or within the failure envelope.

5.2 Interaction Diagram Calculation

The derivation of the interaction diagrams is provided in detail in chapter 3. In practice, the diagrams provided in this paper were created with spreadsheet software.

Each diagram is based on several individual points. The points correspond to the pile under pure compression, the pile when the compression zone extends the entire height of the pile, the pile when the compression zone extends to the extreme prestressing strand in tension, and the series of points calculated as the tensile strain in the prestressing strands increases from 0.001 to 0.015 in 0.0001 increments. Sample calculations are provided in Appendix 5 and the “pseudo-code” used in the spreadsheet is provided in Appendix 6.

5.3 Pile 1

Pile 1 was tested several times in both the north and south directions. Testing began on November 30, 2000 and continued through June 27, 2001. The pile was initially pulled north eleven times and then south eight times. The maximum deflection achieved by the pile was approximately 2 inches in a south pull on June 22, 2001. Prior to this test, the pile was tested to a maximum deflection of 1 inch. The maximum sustained pull load and moment at the pile and abutment interface of 68 kips and 128 kip-feet respectively were both reached during the test to 2 inches (Appendix 2). Two different hold down assemblies were used on Pile 1 over the course of testing. Each assembly provided a different degree of rotational stiffness (4). A final assembly was used for both Pile 2 and 3.

The interaction diagram for Pile 1 is based on the maximum developable strand stress (f_{ps}) and f_{se} at a particular location, the pile geometry, and material properties. Figure 9 consists of a series of curves relating embedded and exposed strand lengths

**Developed Strand Stress vs. Exposed Strand Length
0.5" Diameter, 7 Strand, Grade 270, Low Relaxation**

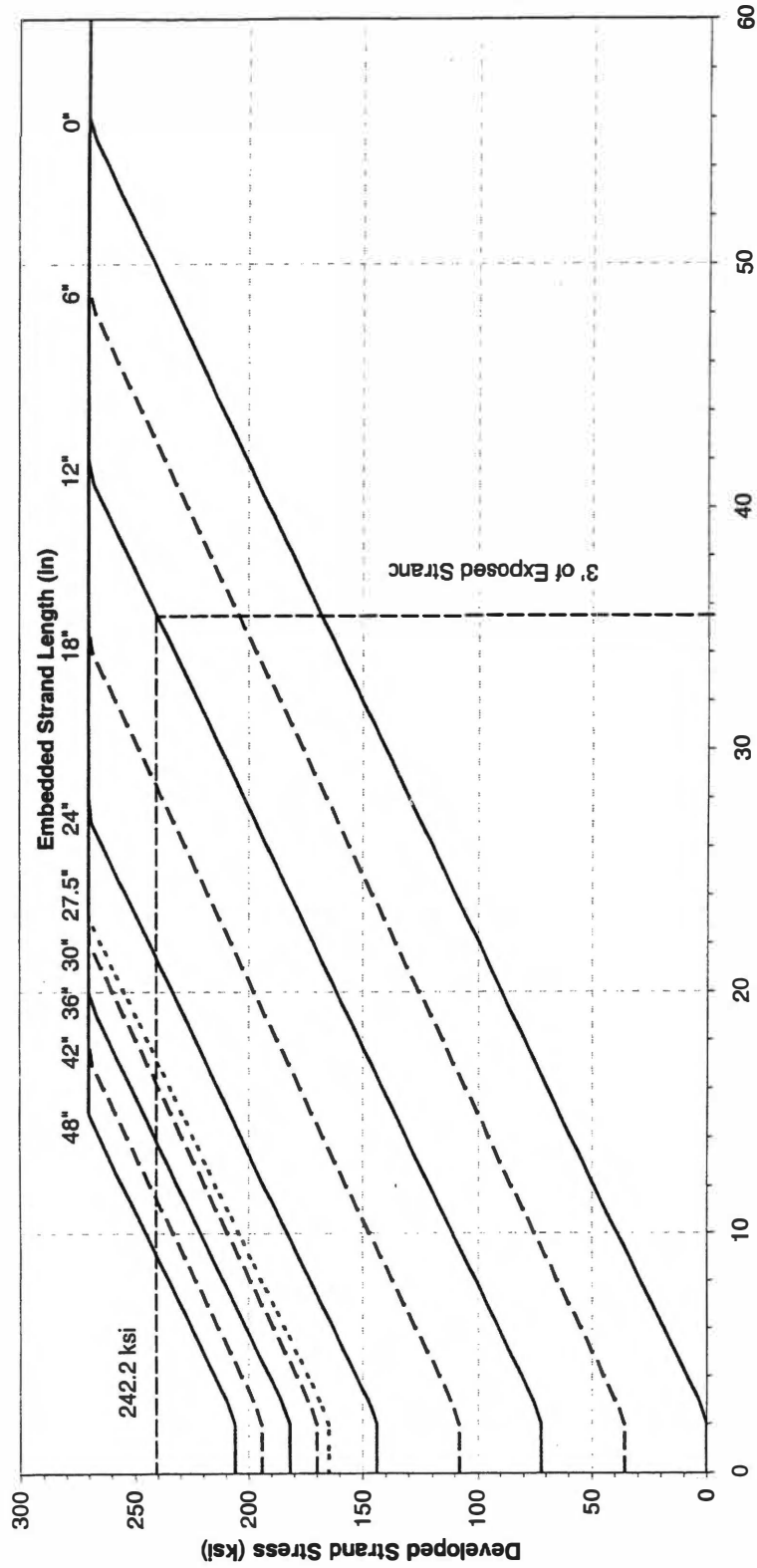


Figure 9: Strand developed stress chart

to the maximum developable stress in the strands. Based on these curves, 12 inches of embedded strand and 36 inches of exposed strand correspond to 240 ksi of developable strand stress and an f_{se} of 72 ksi. Figure 9 is specific to 0.5 inch diameter, grade 270, seven wire, low relaxation prestressing strand. Based on Figure 9, the strands in Pile 2 developed an f_{ps} of 270 ksi 4 inches below the pile and abutment interface and an f_{se} of 165 ksi 15.5 inches below the pile and abutment interface. Based on the maximum strand stress and f_{se} , pile geometry, and material properties, the interaction diagrams in Figure 10 were created. The three curves are based on a maximum strand stress of 240 ksi and an f_{se} of 72 ksi, a maximum strand stress of 270 ksi and an f_{se} of 96 ksi, and a fully developed strand with 270 ksi of stress and an f_{se} of 165 ksi. Therefore, one curve corresponds to the pile capacity at the pile and abutment interface, one curve corresponds to the pile 4+ inches below the pile and abutment interface, and one curve corresponds to pile 15.5+ inches below the pile and abutment interface.

The test results for Pile 1 should conform to the interaction curves in Figure 10. The pile did not fail during testing. A final 1 inch test was run after the 2 inch test without incident (Appendix 2). Therefore, all of the test results should lie within the failure envelope created by the 240 ksi interaction diagram. This is the case. All of the test results for Pile 1 do lie within the predicted failure envelope.

**Pile 1
Interaction Diagram**

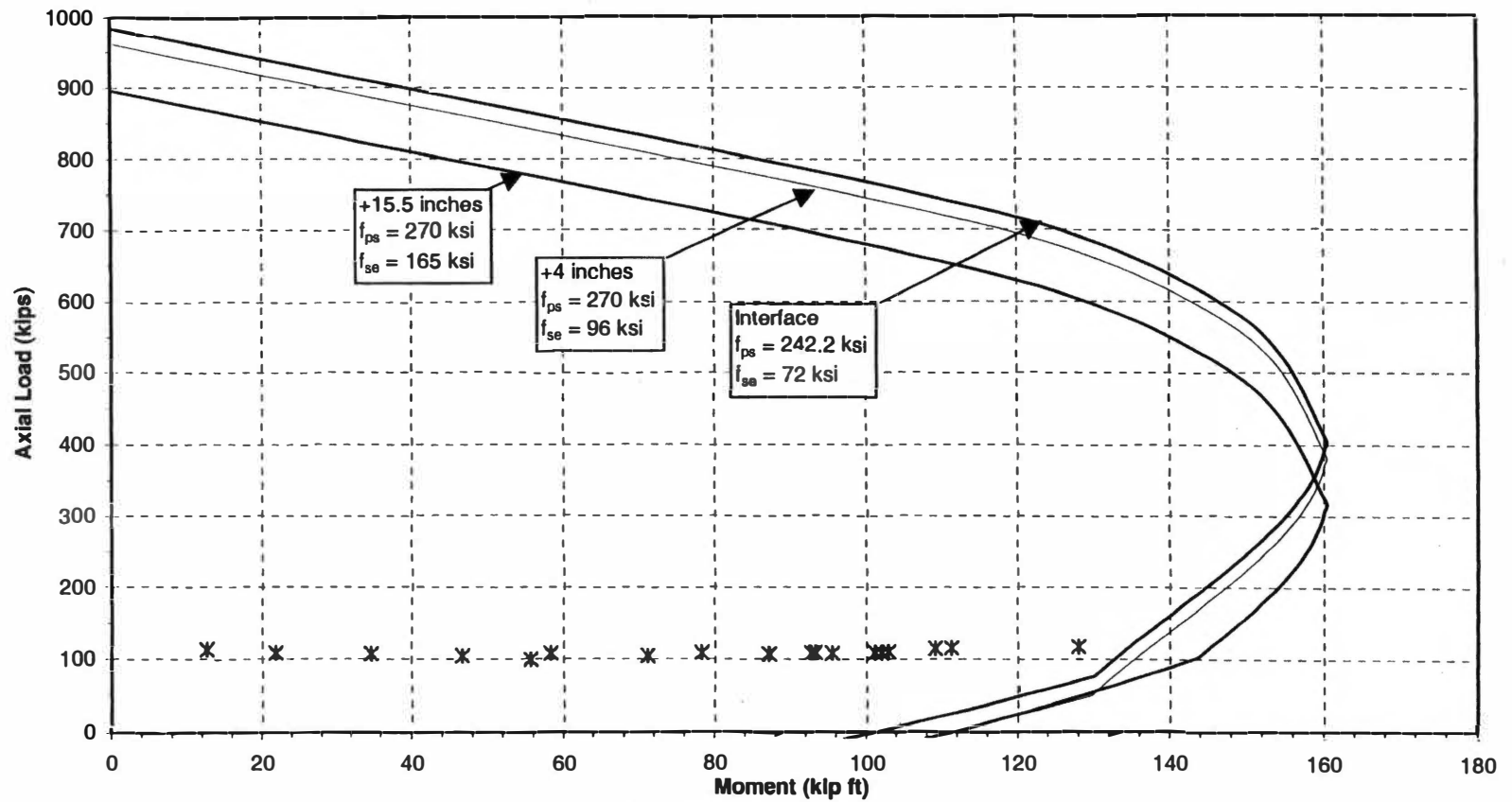


Figure 10: Pile 1 Interaction Diagrams

5.4 Pile 2

Pile 2 was tested several times before failure. Testing began on July 30, 2001 and continued through October 23, 2001. The pile was initially pulled south eight times and then north six times before the test to failure on October 23, 2001. Failure was defined as a complete structural failure of the pile or abutment. When tested to the south as well as the north, Pile 2 was pulled to 0.5 inches repeatedly before being pulled to 1 inch several times. After five 1 inch tests to the north, the pile was pulled to approximately 10 inches before failure. The pull load and moment at the pile and abutment interface necessary to deflect the pile increased each time the deflection increased (Appendix 2). The maximum pull load and moment sustained by the pile, 105 kips and 146 kip-feet respectively, were achieved during the test to failure.

The interaction diagram for Pile 2 is based on the maximum developable strand stress and f_{se} at a particular location, the pile geometry, and material properties. Figure 9 consists of a series of curves relating embedded and exposed strand lengths to the maximum developable stress in the strands. Based on these curves, 12 inches of embedded strand and 36 inches of exposed strand correspond to 240 ksi of developable strand stress and an f_{se} of 72 ksi. Figure 9 is specific to 0.5 inch diameter, grade 270, seven wire, low relaxation prestressing strand. Based on Figure 9, the strands in Pile 2 develop an f_{ps} of 270 ksi 4 inches below the pile and abutment interface and an f_{se} of 165 ksi 15 inches below the pile and abutment interface. Based on the maximum strand stress, f_{se} , pile geometry, and material properties, the interaction diagrams in Figure 11 were

**Pile 2
Interaction Diagram**

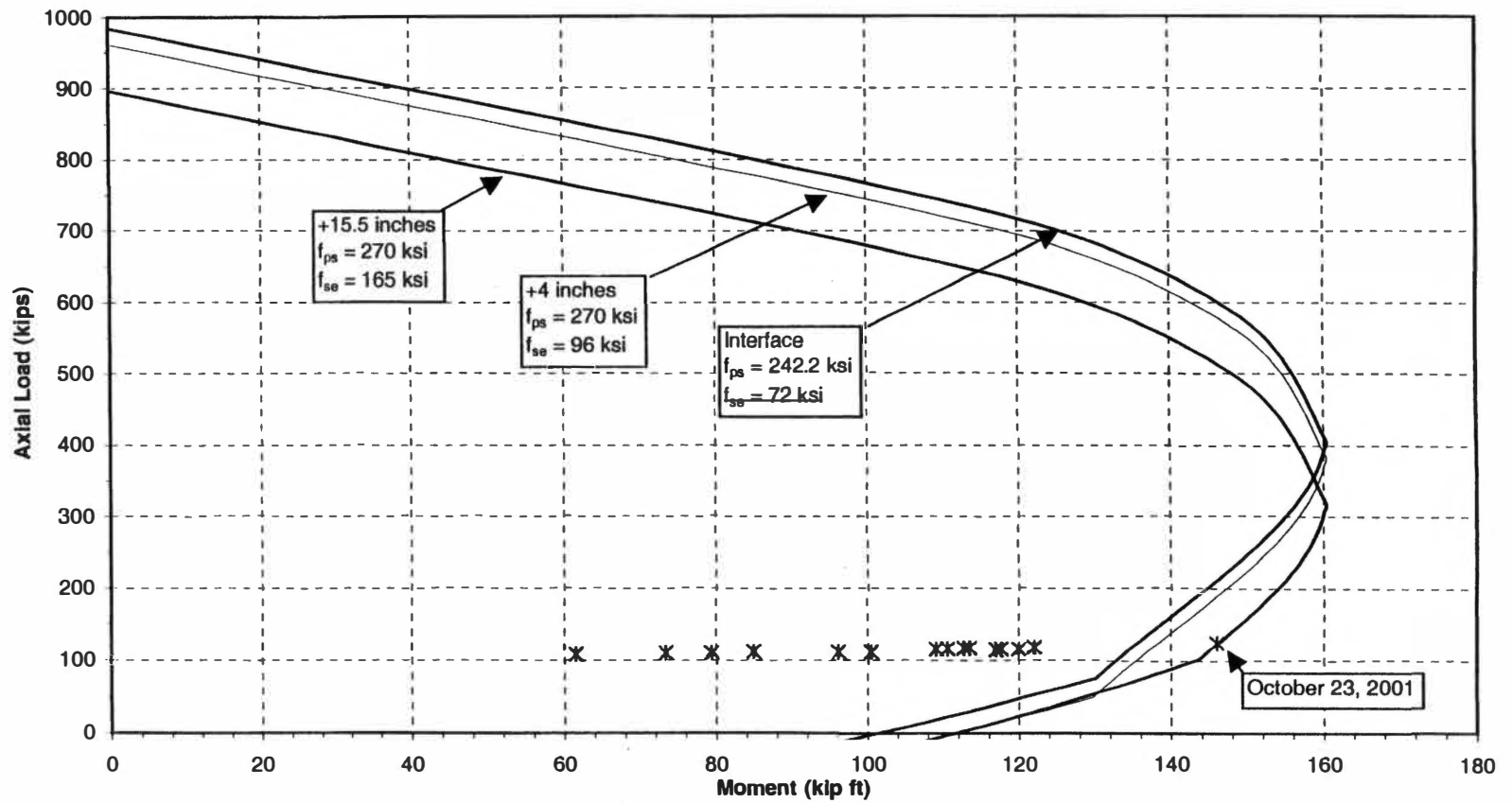


Figure 11: Pile 2 Interaction Diagrams

created. The three curves are based on a maximum strand stress of 240 ksi and an f_{se} of 72 ksi, a maximum strand stress of 270 ksi and an f_{se} of 96 ksi, and a fully developed strand with 270 ksi of stress and an f_{se} of 165 ksi. Therefore, one curve corresponds to the pile capacity at the pile and abutment interface, one curve corresponds to the pile 4+ inches below the pile and abutment interface, and one curve corresponds to pile 15+ inches below the pile and abutment interface.

The test results for Pile 2 should conform to the interaction curves in Figure 11. Because the maximum sustained pull load and moment at the pile and abutment interface were achieved during the test to failure, this point should lie on the 240 ksi interaction curve. All loads and moments from previous tests should lie within the failure envelope. With the exception of the test to failure, this is the case. The test to failure achieved a pull load and moment that exceeded the predicted results. The maximum moment of 146 kip-feet with an axial load of 125 kips exceeds the predicted value of 136 kip-feet by 7%. This error may be due to incorrect concrete or steel strengths, increases in steel stress beyond the yield point, or inaccuracies in the geometric assumptions used in the model.

5.5 Pile 3

Pile 3 was tested several times before it failed. Testing began on December 11, 2001 and continued through April 11, 2002. The pile was initially pulled north six times and then south nine times before the two tests to failure on April 4 and 11, 2002. Failure was defined as either 4 inches of deflection or a 10% loss in pull load during testing. The pile was pulled to 4 inches during both tests to failure. During both tests to failure, the

pile reached a maximum load and continued to deflect without an associated increase in load or even with a decrease in pull load (Appendix 3). Unlike Pile 2, Pile 3 reached approximately 50 kips of pull load and 90 kip-feet of moment at the pile and abutment interface during almost every test through the first test to failure regardless of deflection. The exceptions were the three cyclic 1 inch tests on March 7, 2002 and the final test to failure (Table 2).

The interaction diagram for Pile 3 is based on the maximum developable strand stress and f_{se} at a particular location, the pile geometry, and material properties. According to the curves in Figure 9, 12 inches of embedded strand and no exposed strand corresponds to 72 ksi of developable strand stress. Based on the maximum strand stress the interaction diagrams in Figure 12 were created. The two curves are based on a maximum strand stress and f_{se} of 72 ksi and a fully developed strand capable of developing 270 ksi of stress and an f_{se} of 165 ksi. Based on Figure 9, the strands in Pile 2 are fully developed 68 inches below the pile and abutment interface. Therefore, one curve corresponds to the pile capacity at the pile and abutment interface, and one curve corresponds to the pile 68+ inches below the pile and abutment interface.

The test results for Pile 3 should conform to the interaction curves in Figure 12. Because each test with the exception of those on March 7, 2002 and April 11, 2002 reached the same maximum pull load and moment at the pile and abutment interface, these points should lie on the interaction curve. The tests on March 7, 2002 and April 11, 2002 do lie within the failure envelope (Figure 12). However, the remaining tests exceed

Table 2: Pile 3 maximum pull loads and moments

Pile 3			
	Date	Pull Load (kips)	Moment (kip-ft)
North	12/11/01	53.7	94.3
	12/18/01	49.1	90.3
	12/18/01	51.0	93.7
	12/19/01	58.8	95.0
	12/20/01	54.6	91.6
	12/21/01	54.6	96.9
South	2/26/02	50.0	89.7
	2/28/02	54.0	91.3
	3/1/02	51.0	86.7
	3/7/02	44.9	79.5
	3/7/02	43.9	78.4
	3/7/02	43.7	77.8
	3/8/02	61.7	92.0
	3/28/02	54.8	89.1
	3/29/02	51.9	86.5
	4/4/02	52.6	92.4
	4/11/02	49.9	82.7

Pile 3 Interaction Diagram

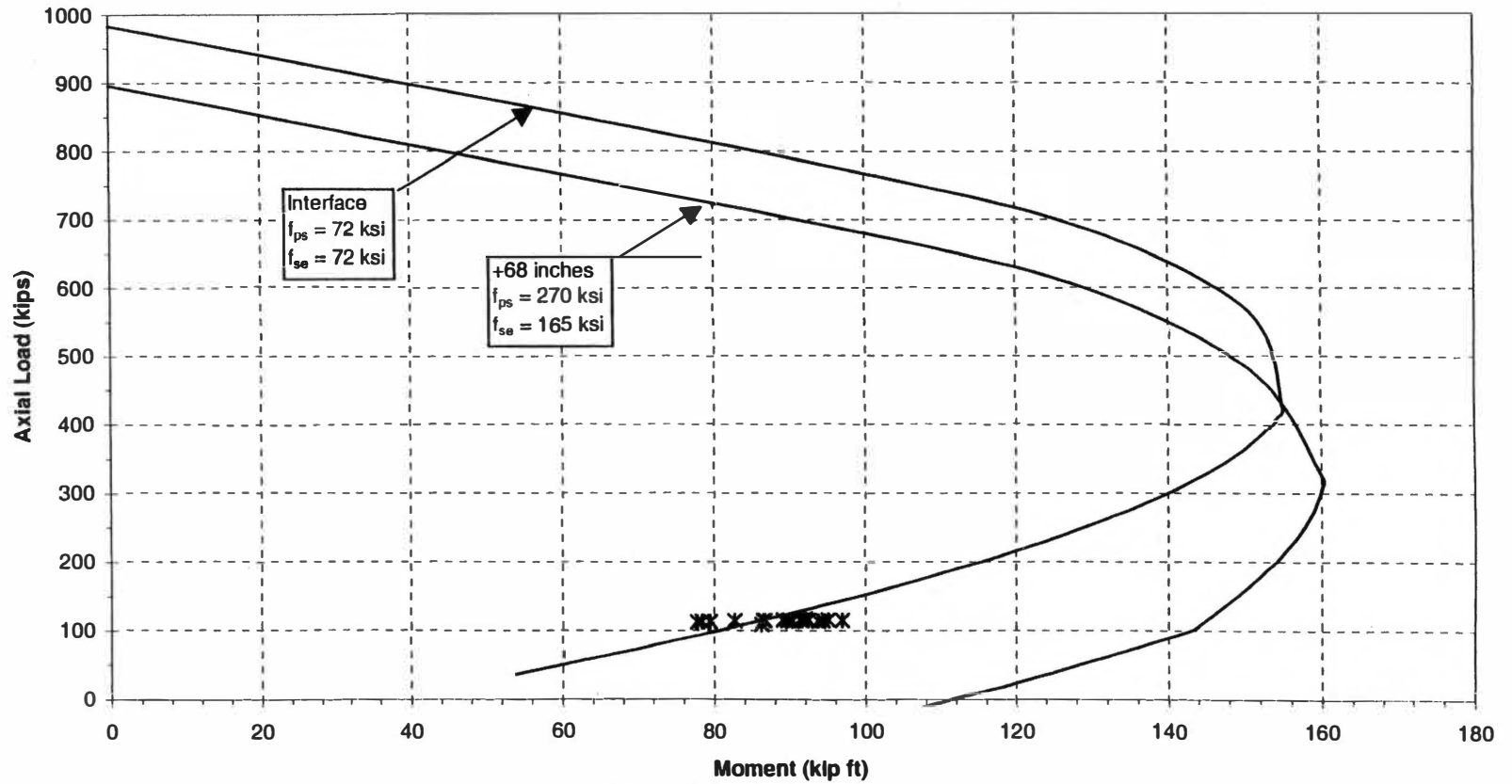


Figure 12: Pile 3 Interaction Diagrams

the predicted value by a range of values from 0.3% to 10.9% with an average of 5.6% (Table 3). The discrepancy between the maximum moments achieved during the two tests to failure suggests the strands began to slip during the test on April 4, 2002. A simultaneous slip of the strands could account for the decrease in moment capacity (9). Upon visual inspection, the pile did not appear to be significantly more damaged during the second test.

5.6 Model Sensitivity

The theoretical model concerning the maximum developable strand stress may be affected by several factors. Figure 9 is dependent on accurate measurements of the embedded and exposed strand lengths. Because the exposed strands develop stress according to a linear relationship, the developable stress in the exposed strands is changed by the same value per inch regardless of the strand length (Table 4). Changes in embedded strand length affect the developable stress by two separate values depending on the initial strand length and the value of the increase or decrease in length. Because the maximum embedded strand stress increases at two different rates depending on whether the strand length exceeds L_T or not, the rate of change as the strand length increases is also dependent on whether or not the strand length exceeds L_T or the change in length crosses the transition from L_T to L_F (Figure 2 and Table 4).

Table 3: Pile 3 error in moment predictions

Pile 3			
Axial Load (kips)	Actual Moment (kip-ft)	Predicted Moment (kip-ft)	Difference (%)
114.1	94.3	86.1	8.7%
113.0	90.3	85.7	5.1%
113.5	93.7	85.9	8.3%
115.1	95.0	86.5	8.9%
114.1	91.6	86.1	6.0%
114.4	96.9	86.3	10.9%
114.1	89.7	86.1	4.0%
115.0	91.3	86.5	5.3%
114.1	86.7	86.1	0.7%
112.4	79.5	85.5	-7.5%
112.2	78.4	85.4	-8.9%
112.1	77.8	85.4	-9.8%
116.6	92.0	87.1	5.3%
115.1	89.1	86.5	2.9%
114.3	86.5	86.2	0.3%
114.8	92.4	86.4	6.5%
113.7	82.7	86.0	-4.0%

Table 4: Theoretical model sensitivity

Strand Stress Model Sensitivity		
	Additional Strand Length	Additional Developed Stress
Embedded Length $< L_T$ $> L_T$	6"	35.8 ksi
	6"	12.0 ksi
Exposed Length	6"	31.8 ksi

The derivation of the interaction diagrams is also sensitive to changes in the material properties and strand stress. In particular, the model is sensitive to changes in f_{se} , f_{pu} , f'_c , and f_{ps} . Each change in the initial pile conditions affects the final interaction diagram in a different way. For instance, f_{se} is responsible for a compressive force in the pile. When the pile is subjected to a pure axial load, this compressive force is considered uniform. However, as a moment is applied to the pile, the effects of f_{se} are overcome by the effects of the moment induced couple. If f_{se} remains unchanged, but f_{ps} or f_{pu} is changed, the interaction diagram is affected when the moment in the pile is large enough to overcome f_{se} . Therefore, the point on the interaction diagram where changes in f_{se} stop having an effect is nearly the point where changes in f_{ps} or f_{pu} begin having an effect. Changes in the length of exposed and embedded strands and their ratio to one another can affect both f_{se} and f_{ps} . Changes in f'_c affect the entire interaction diagram as long as the axial load is compressive. Once the axial load transitions to tension, the effects of changing f'_c become less noticeable. The manner in which the pile's initial conditions are changed controls the effect on the final interaction diagram.

The effects of an increase or decrease in f_{se} are most noticeable as the pile approaches pure compression. The effects converge to zero as the pile approaches pure bending and tension loads (Figures 13 and 14). As f_{se} is increased, the ratio of axial load to moment is decreased, and as f_{se} is decreased, the ratio of axial load to moment is increased. The reason is that as f_{se} is increased or decreased, the initial compression in

Interaction Diagram Effects of f_{se} Increase

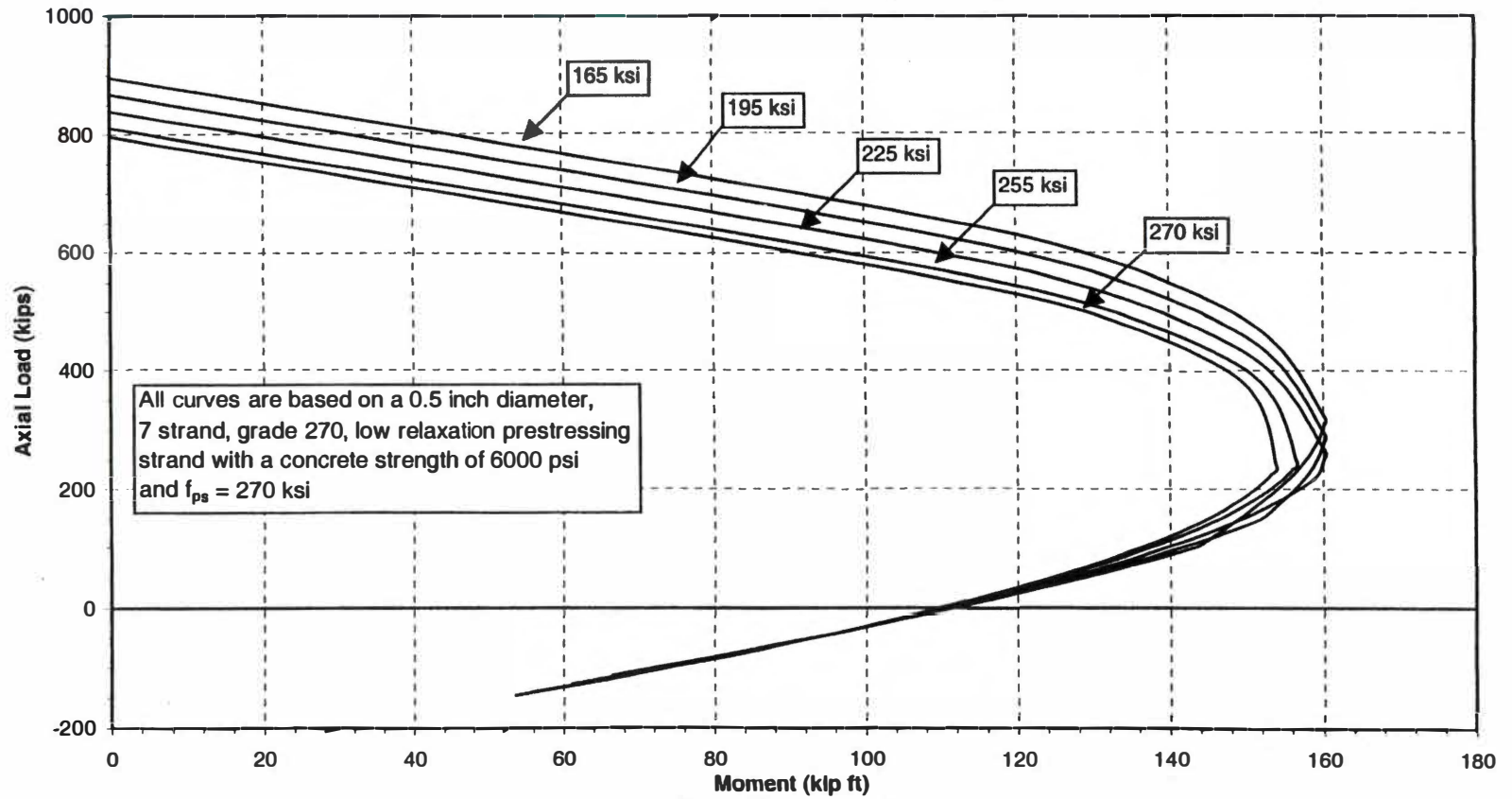


Figure 13: Increase in f_{se}

Interaction Diagram
Effects of f_{se} Decrease

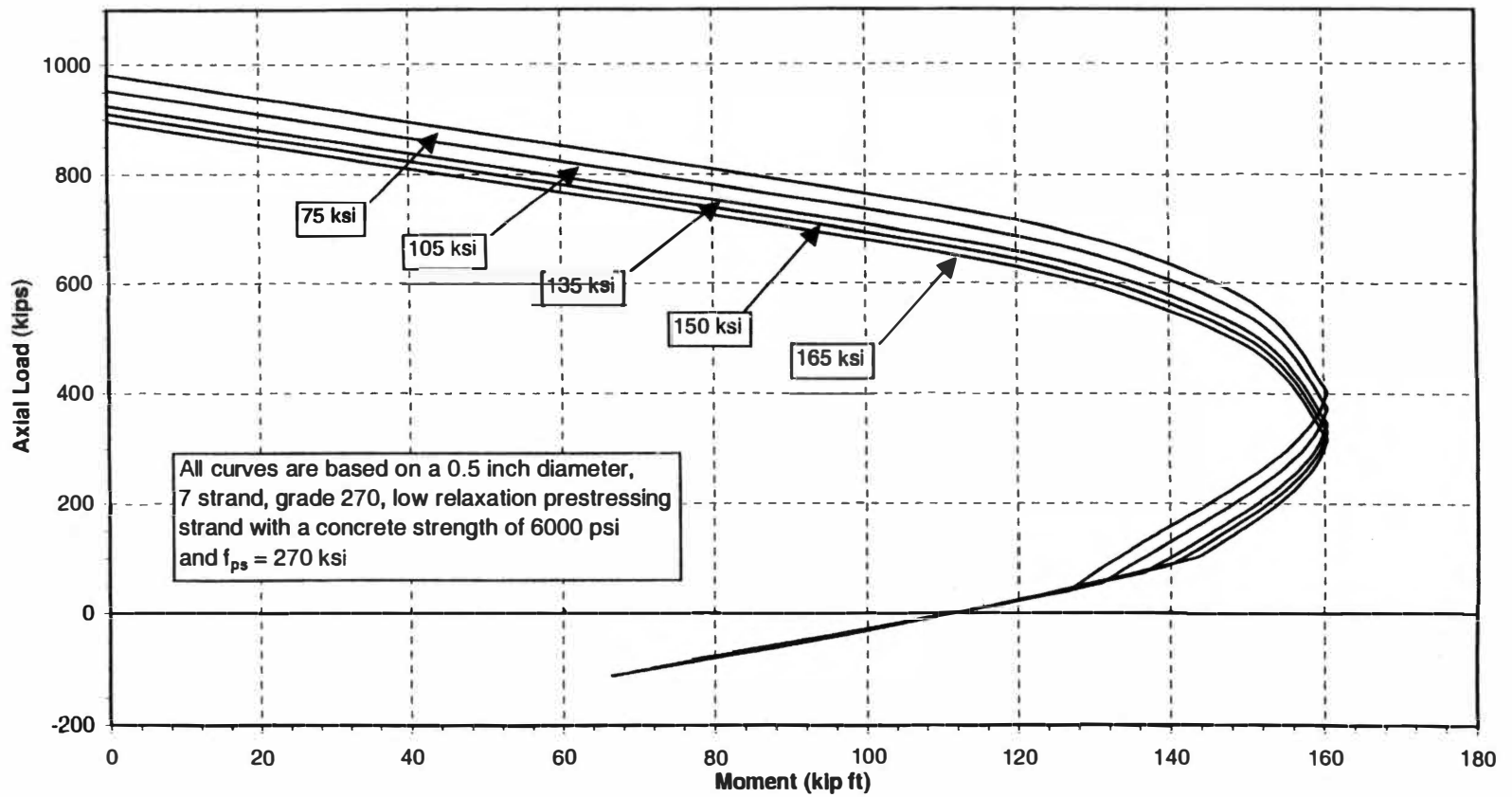


Figure 14: Decrease in f_{se}

the pile due to the prestress is increased or decreased. Therefore, either more or less of the pile's compressive capacity is used to support the prestressing strands. As the pile approaches pure bending, the effects of the moment outweigh the effects of the prestress in the strands.

When f_{se} remains constant but f_{pu} is either increased or decreased, the effect on the interaction diagram is similar. However, the effects are in the region of pure bending and tension loads. As f_{pu} is increased, the ratio of axial load to moment decreases, and as f_{pu} is decreased, the ratio of axial load to moment is increased (Figures 15 and 16). An increase in f_{pu} allows the prestressing strands to carry a greater force. This allows a larger moment to be sustained, but the axial capacity is reduced. For an example, refer to the sample calculations in Appendix 5. If f_{pu} is increased to 280 ksi, the axial capacity is reduced from 45.68 kips to 41.09 kips, and the moment is increased from 127.09 kip-feet to 128.8 kip-feet.

The lengths of the embedded and exposed strands and their ratio to one another provide the ability to manipulate both f_{se} and f_{ps} (Figures 17 and 18). Only the length of the embedded strand can control f_{se} . However, f_{ps} is affected by the lengths of both the exposed and embedded strands. If the desire is to reduce f_{se} at the pile and abutment interface without reducing f_{ps} , the length of the embedded strand may be reduced and the length of the exposed strand increased. Similarly, if the desire is to decrease f_{ps} without reducing f_{se} , the embedded strand length must remain constant, but the

Interaction Diagram
Effects of an Increase in f_{ps}

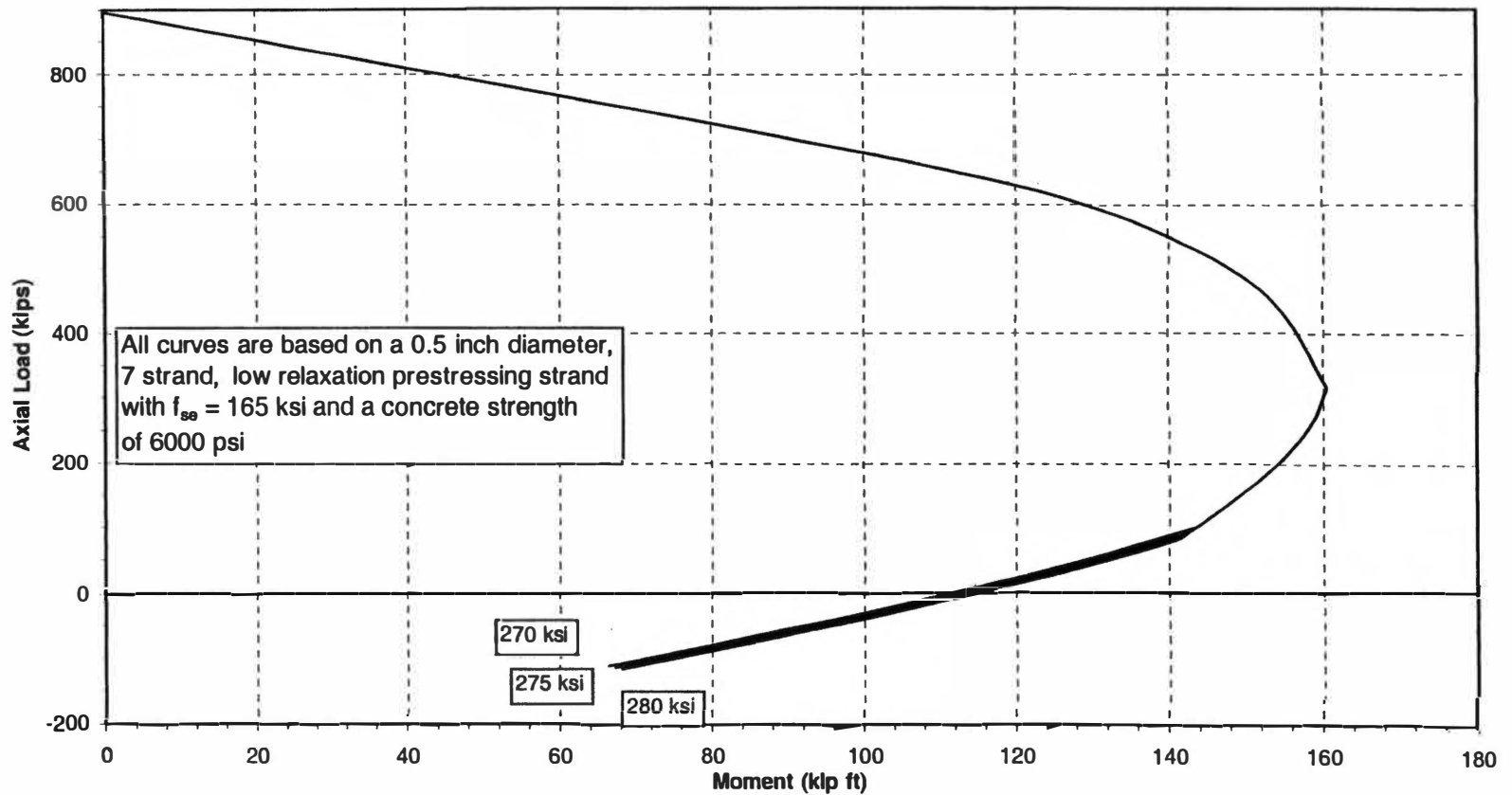


Figure 15: Increase in f_{ps}

Interaction Diagram
Effects of a Decrease in f_{ps}

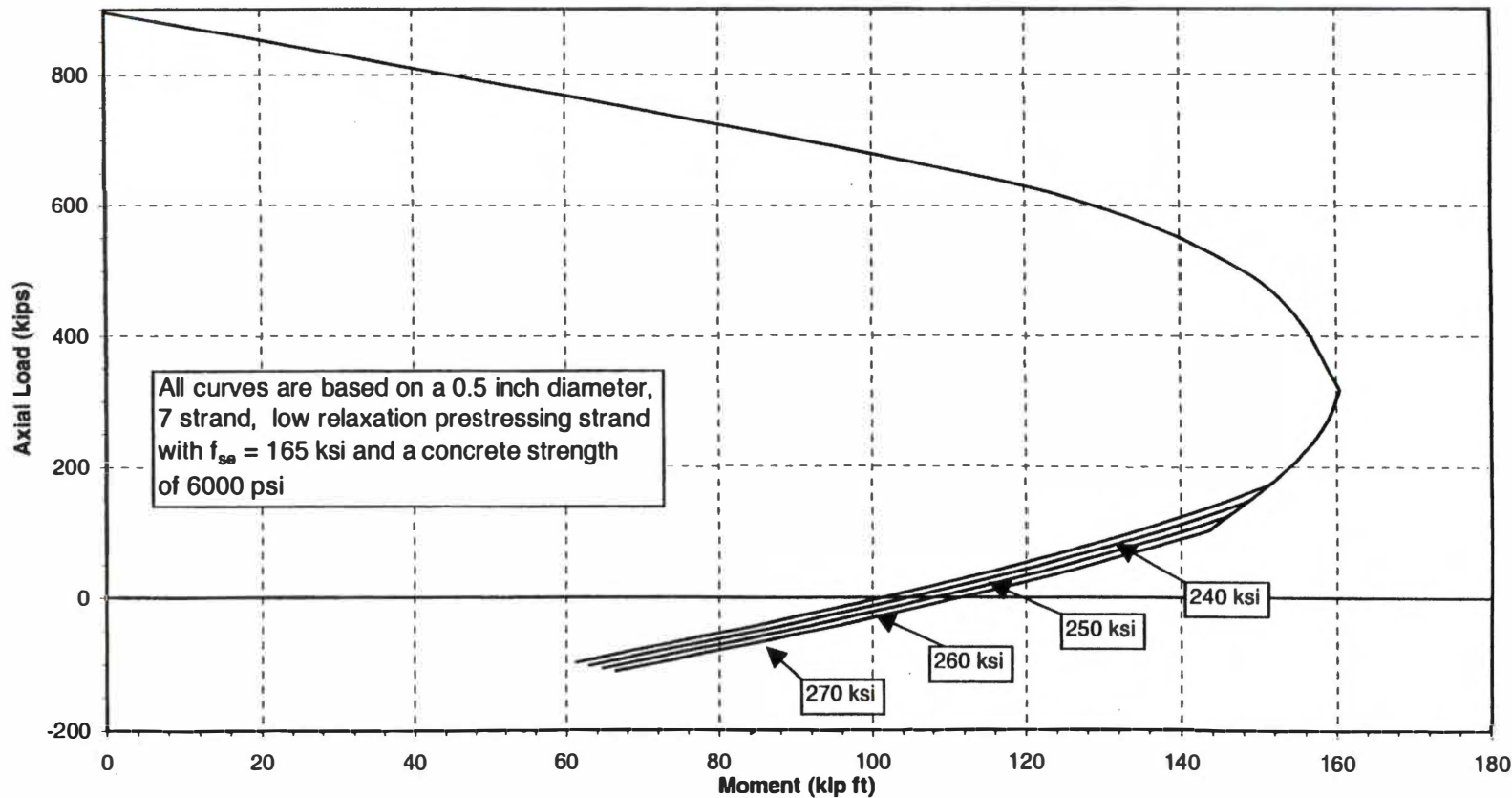


Figure 16: Decrease in f_{ps}

Interaction Diagram
Developable Strand Stress Reduced to f_{se}

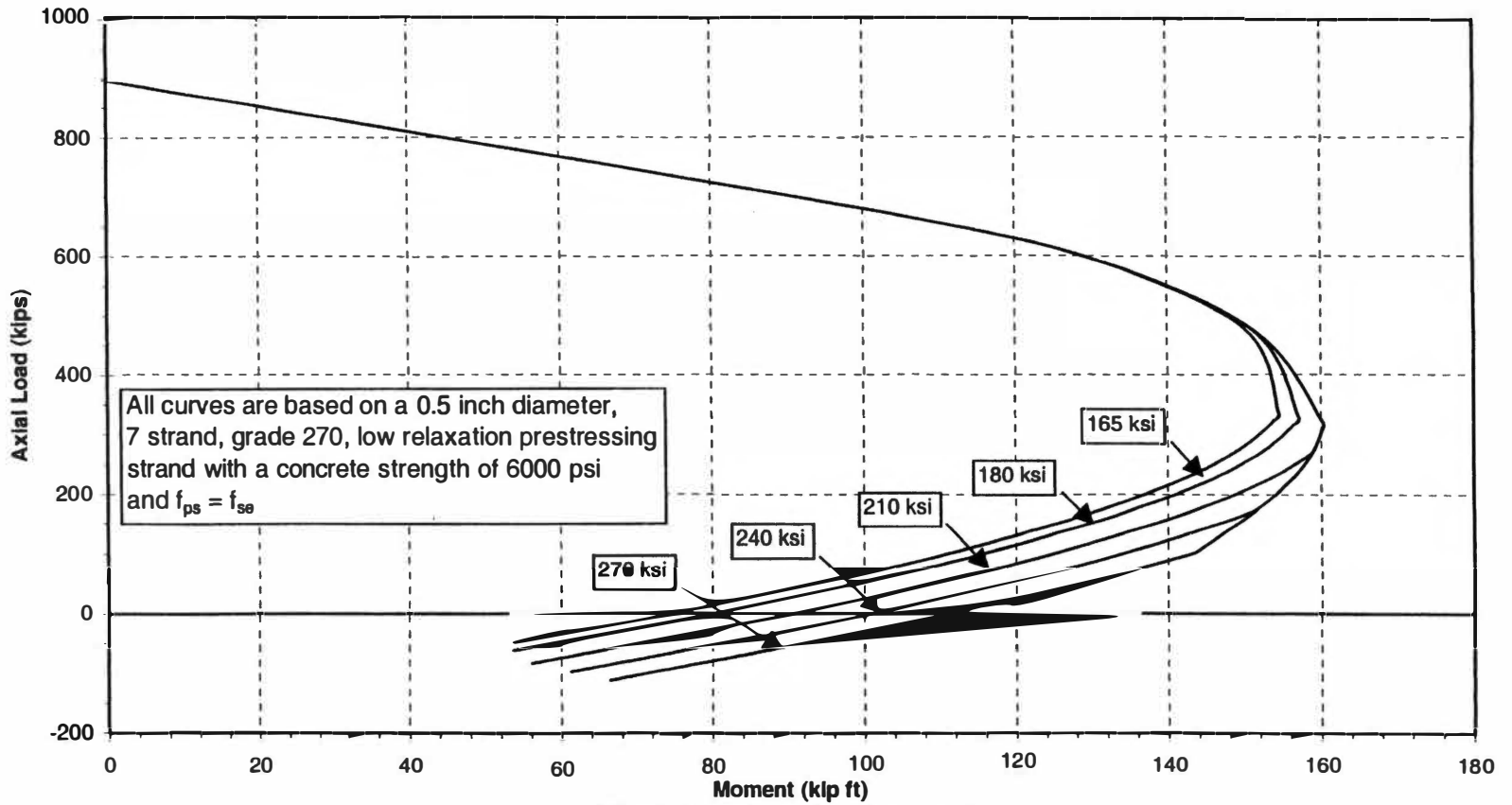


Figure 17: Decrease in Strand Stress to f_{se}

Interaction Diagram **Developable Strand Stress Reduced beyond f_{se}**

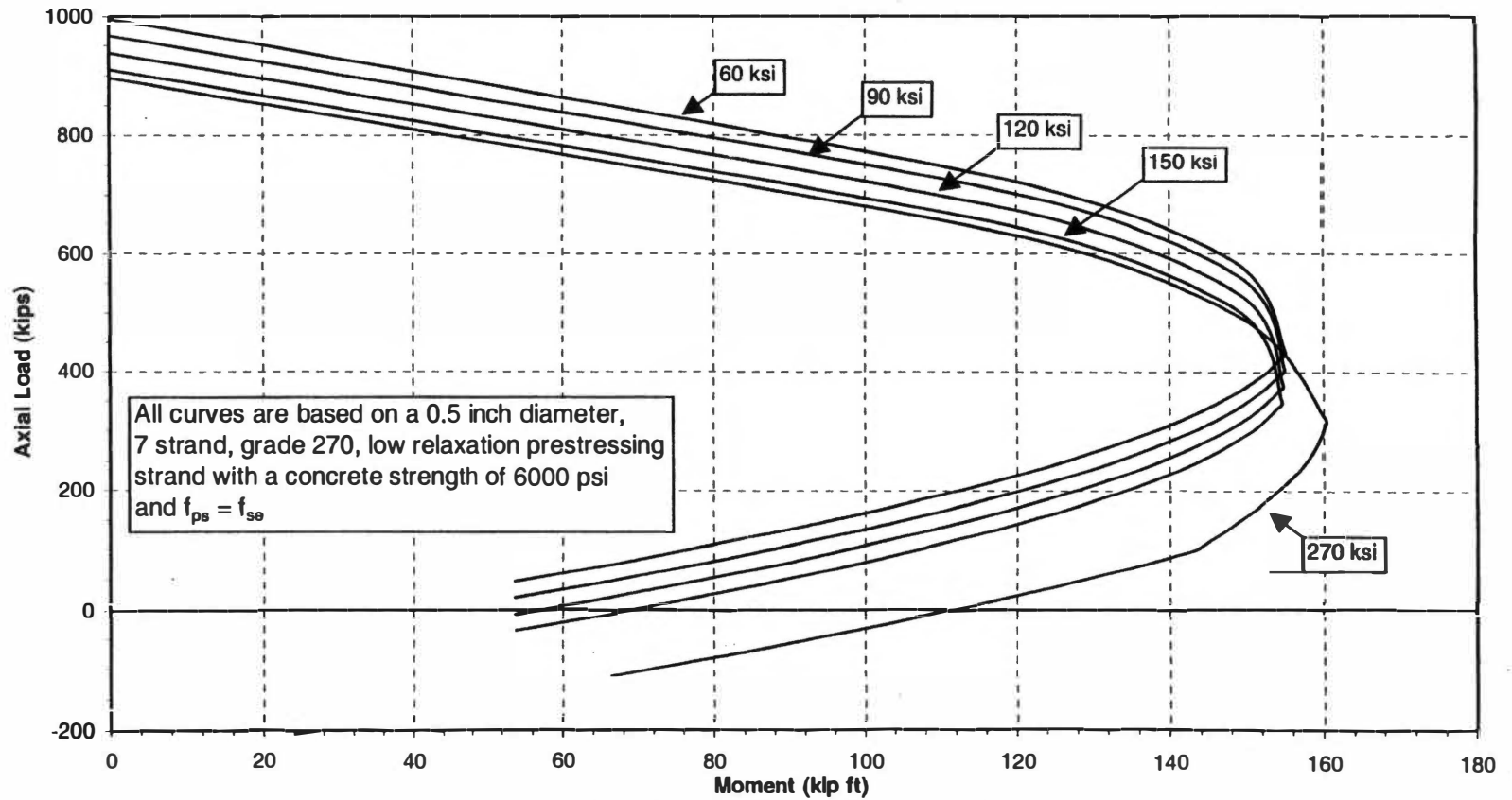


Figure 18: Decrease in Strand Stress beyond f_{se}

exposed length of strand may be reduced. The limit is that f_{pu} cannot be increased without changing the type of prestressing strand used.

An increase or decrease in the concrete strength also affects the interaction diagram. As the concrete strength is increased, the ratio of axial load to moment is increased. The opposite is also true. As the pile approaches pure bending, the effects of a change in the concrete strength approach zero, because in this region the tensile strength of the strands controls the pile's capacity (Figures 19 and 20).

Interaction Diagram Effects of Concrete Strength Increase

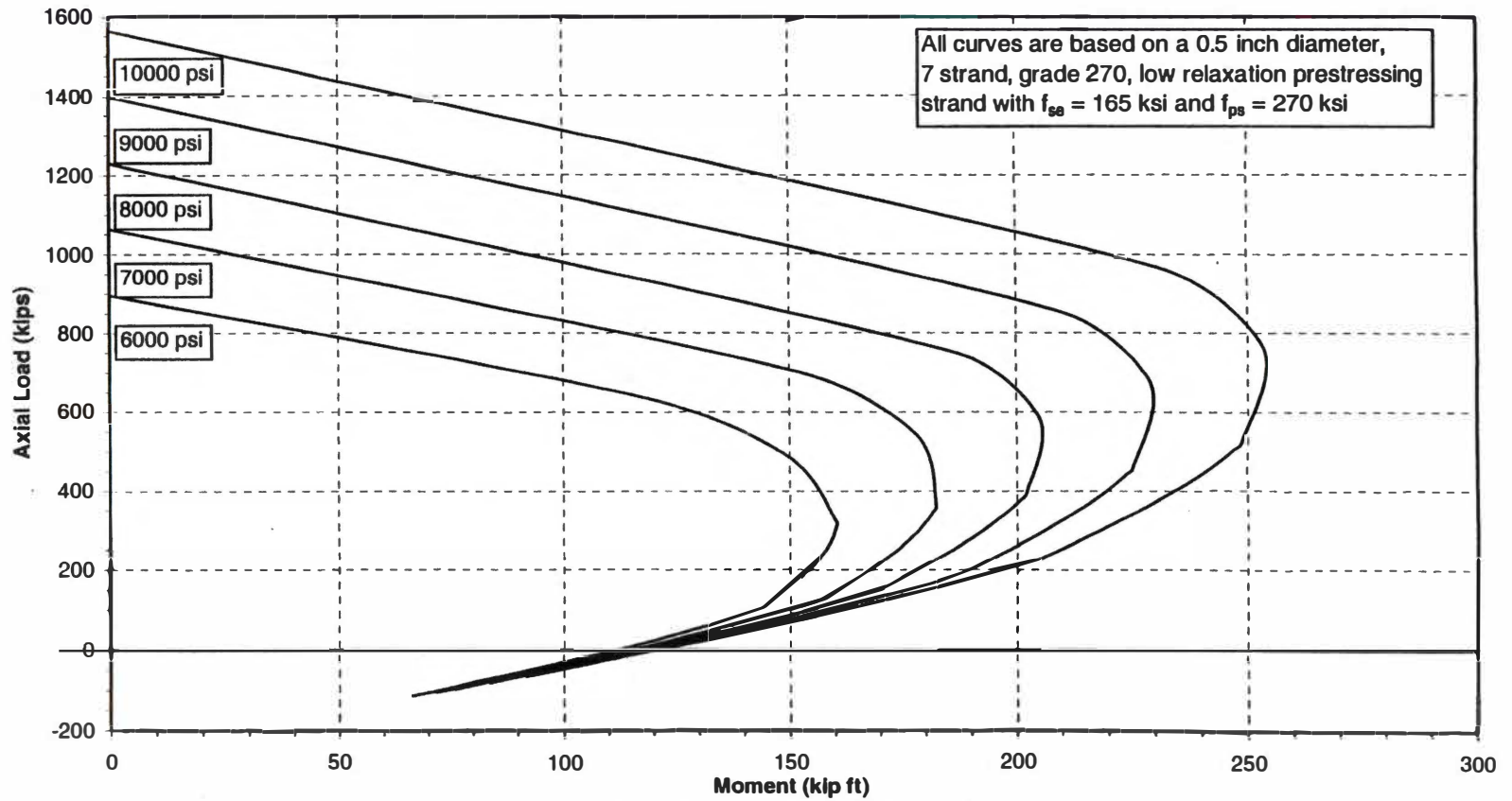


Figure 19: Increase in Concrete Strength

Interaction Diagram Effects of Concrete Strength Reduction

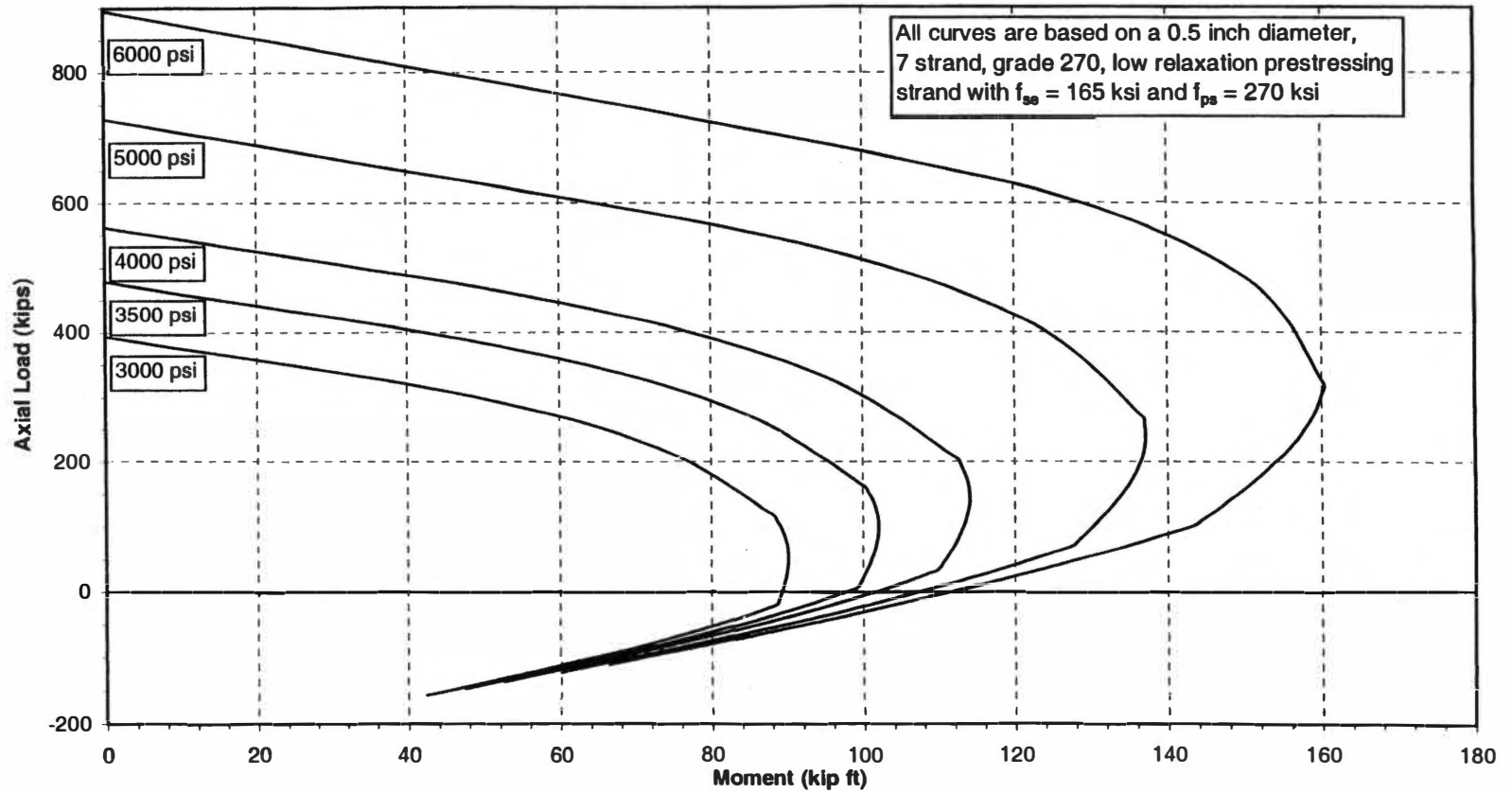


Figure 20: Decrease in Concrete Strength

CHAPTER 6

CONCLUSION

6.1 Introduction

Piles 1, 2, and 3 were tested with the same procedure. The only differences in the piles were the use of or absence of exposed strands and the use of three different hold down assemblies over the course of testing. Therefore, the tests for the piles provide an opportunity to verify the theoretical model, and the theoretical model provides an explanation of why each pile was able to sustain the loads it did.

6.2 Theoretical Model

The assumptions and processes used to derive the interaction diagram and the prestressing strand stress are applicable to prestressed concrete piles. In particular, the process of independently determining the maximum prestressing strand stress for exposed and embedded strands and summing the values in order to determine the total maximum strand stress is valid. However, f_{se} is dependent solely on the embedded length.

In the case of Piles 1 and 2, the maximum strand stress at the pile and abutment interface was determined to be 240 ksi with an f_{se} of 72 ksi. All test data agreed with the predicted interaction diagrams with one exception. The final axial load and moment for Pile 2 exceeded the predicted values by 7%. In the case of Pile 3, the maximum strand stress at the pile and abutment interface was determined to be 72 ksi. Thirteen tests reached axial loads and moments that exceeded the predicted values by an average of

5.6%. Three cyclic tests were an average of 8.7% below the predicted value, and the final test to failure was 4% below the predicted value.

The model determining the maximum strand stress at a given location in the pile is subject to error based on inaccurate measurements of the embedded and exposed lengths. The interaction diagram is sensitive to changes in f_{se} , f_{pu} , f'_c , and f_{ps} .

6.3 Use of Exposed Strands

The use of exposed strands is a viable alternative to the use of larger sections of pile embedded in the abutment. The exposed strands may help develop f_{ps} but have no effect on f_{se} . The maximum developable stress in the exposed strands is based solely on their “pull out” strength.

In the case of Pile 3, the interaction diagram is based on the same f_{se} as Pile 1 and 2. However, the maximum developable stress in the strand at the pile and abutment interface is nearly a third of Piles 1 and 2 due to the lack of exposed strands.

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Appendix 1
Prestressed/Exposed Strand Length
vs.
Developed Stress

Developed Strand Stress vs. Exposed Strand Length 9/16" Diameter, 7 Strand, Grade 270, Low Relaxation

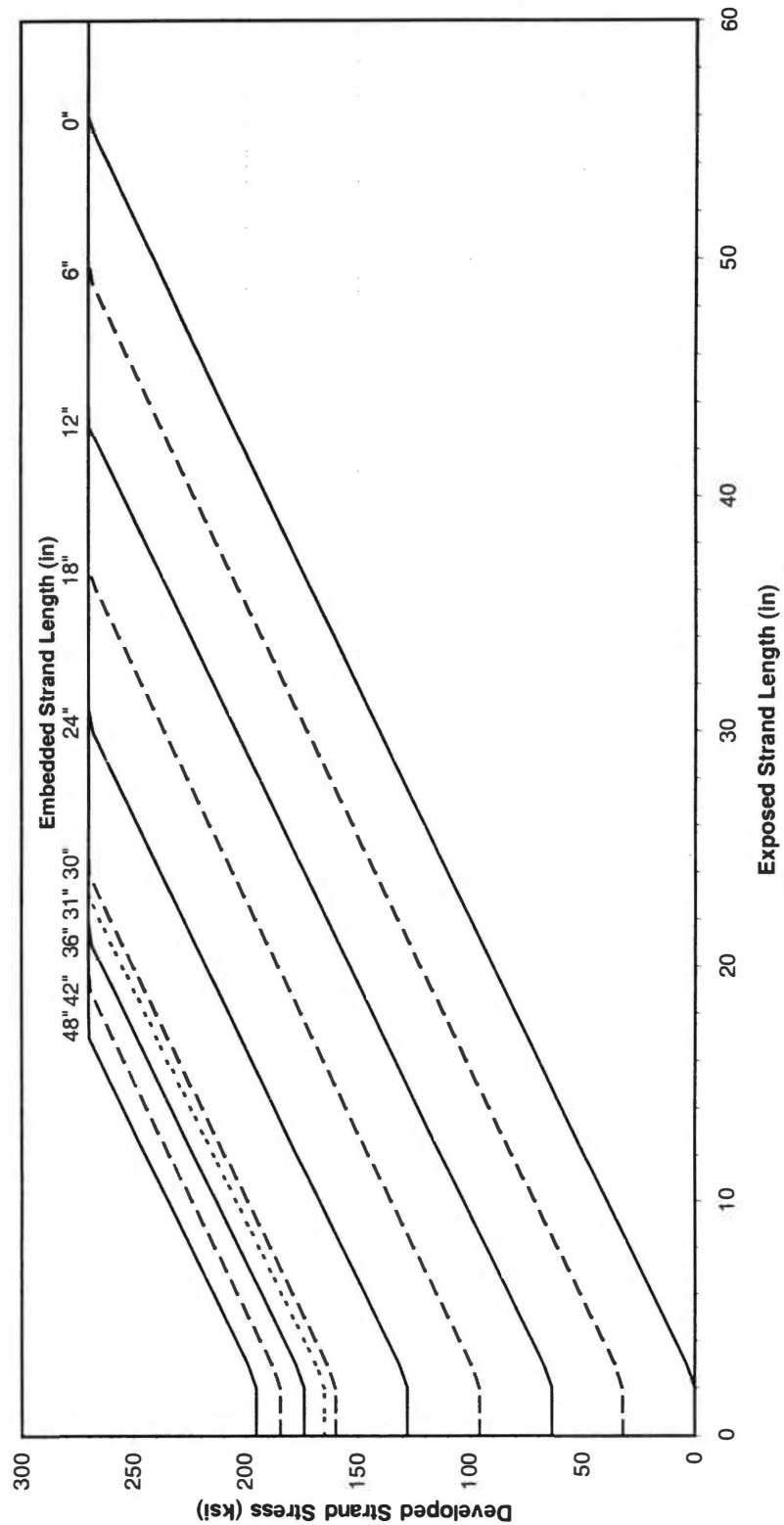


Figure A1.1: 9/16" Diameter Strand

Developed Strand Stress vs. Exposed Strand Length **0.6" Diameter, 7 Strand, Grade 270, Low Relaxation**

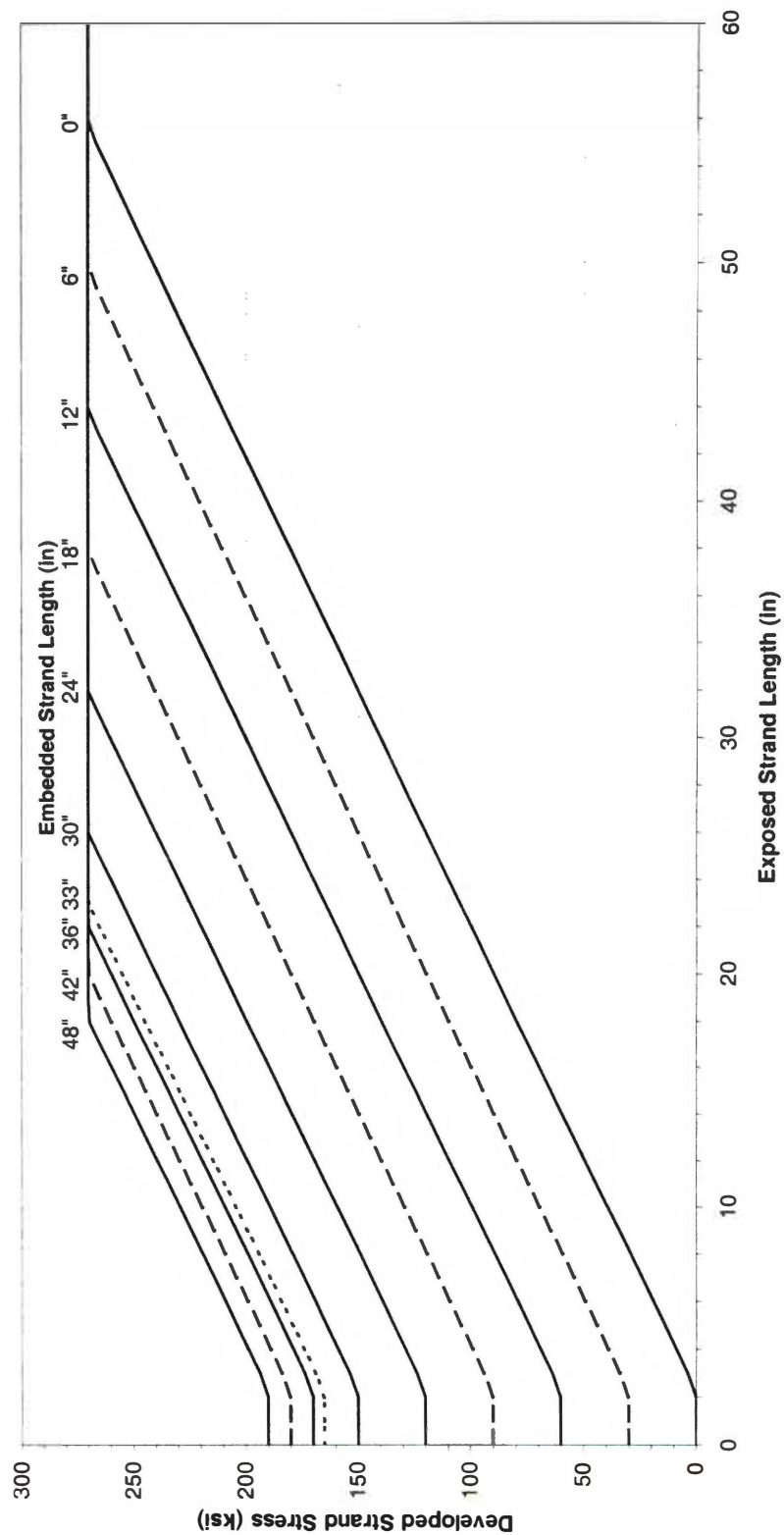


Figure A1.2: 0.6" Diameter Strand

Appendix 2

Test Schedule

Table A2.1: Pile 1 Pulling North

Pile 1 Pull North					
Date	Maximum				Notes
	Pile Ground Displacement (in)	Pull Load (kips)	Hold-Down Load (kips)	Moment (kip-ft)	
11/30/00	0.16	3.1	0.9	55.5	
12/1/00	0.24	31.8	6.0	71.1	
12/8/00	0.31	55.4	10.6	92.7	
1/23/01	0.54	40.0	5.7	46.6	
1/25/01	0.54	43.0	8.5	87.1	
2/8/01	0.56	47.9	9.6	95.5	
2/9/01	0.52	43.8	8.6	87.1	
3/1/01	0.81	55.3	11.0	101.1	
3/8/01	1.04	60.3	11.8	102.9	
3/16/01	1.02	57.7	11.0	93.2	
3/27/01	1.02	53.1	10.0	102.0	

Table A2.2: Pile 1 Pulling South

Pile 1 Pull South					
Date	Maximum				Notes
	Pile Ground Displacement (in)	Pull Load (kips)	Hold-Down Load (kips)	Moment (kip-ft)	
5/21/01	0.52	27.4	8.1	34.7	
5/23/01	0.58	29.7	10.5	21.8	
6/1/01	0.64	33.5	14.9	12.6	
6/13/01	0.39	33.8	9.3	58.4	
6/18/01	1.13	60.5	16.5	109.1	
6/20/01	1.03	60.9	16.8	111.3	
6/22/01	2.04	68.0	19.1	128.0	
6/27/01	1.02	35.5	10.6	78.2	

Table A2.3: Pile 2 Pulling South

Pile 2 Pull South					
Date	Maximum				Notes
	Pile Ground Displacement (in)	Pull Load (kips)	Hold-Down Load (kips)	Moment (kip-ft)	
7/30/01	0.54	44.2	11.09	73.3	
7/31/01	0.52	50.5	13.3	96.1	
8/3/01	0.51	50.1	13.5	100.4	
8/6/01	1.03	68.8	17.6	120.0	
8/12/01	1.02	63.6	16.5	116.9	
8/15/01	1.05	67.7	17.2	117.6	

Table A2.4: Pile 2 Pulling North

Pile 2 Pull North					
Date	Maximum				Notes
	Pile Ground Displacement (in)	Pull Load (kips)	Hold-Down Load (kips)	Moment (kip-ft)	
9/26/01	0.52	41.3	10.0	61.5	
9/28/01	0.55	52.4	12.8	79.4	
10/1/01	0.52	56.0	13.7	84.9	
10/3/01	1.03	77.9	18.7	112.8	
10/5/01	1.04	80.6	19.3	113.5	
10/8/01	1.01	72.9	17.8	110.4	
10/10/01	1.02	71.1	17.4	108.9	
10/18/01	1.33	86.6	20.7	122.0	load in 30 min hold-down failed
10/23/01	10.14	104.6	26.9	146.3	Test to Failure

Table A2.5: Pile 3 Pulling North

Pile 3 Pull North					
Date	Maximum				Notes
	Pile Ground Displacement (in)	Pull Load (kips)	Hold-Down Load (kips)	Moment (kip-ft)	
12/11/01	0.54	53.7	15.9	94.3	
12/18/01	0.54	49.1	14.8	90.3	electronic data lost
12/18/01	0.55	51.0	15.3	93.7	2 nd in a day
12/19/01	1.05	58.8	16.9	95.0	
12/20/01	1.03	54.6	15.9	91.6	
12/21/01	1.03	54.6	16.2	96.9	

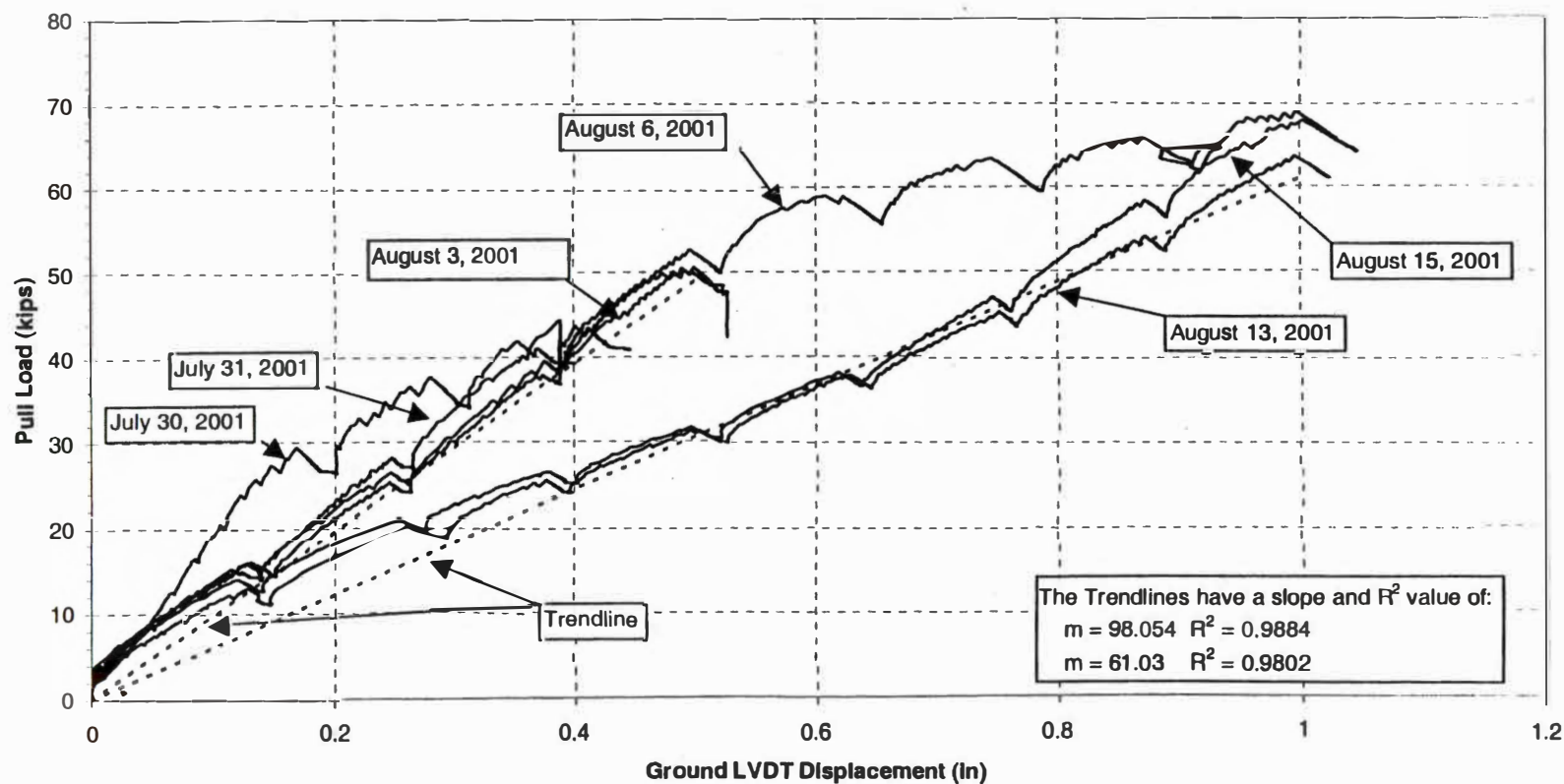
Table A2.6: Pile 3 Pulling South

Pile 3 Pull South					
Date	Maximum				Notes
	Pile Ground Displacement (in)	Pull Load (kips)	Hold-Down Load (kips)	Moment (kip-ft)	
2/26/02	1.16	50.0	15.9	89.7	
2/28/02	1.16	54.0	16.8	91.3	
3/1/02	1.13	51.0	15.9	86.7	
3/7/02	0.99	44.9	14.2	79.5	three cyclic 1" tests
3/7/02	1.00	43.9	14.0	78.4	
3/7/02	1.01	43.7	13.9	77.8	
3/8/02	2.05	61.7	18.4	92.0	
3/28/02	2.03	54.8	16.9	89.1	
3/29/02	2.02	51.9	16.1	86.5	
4/4/02	4.07	52.6	16.6	92.4	Test to Failure
4/11/02	4.31	49.9	15.5	82.7	Test to Failure

Appendix 3

P- Δ Curves

Pile 2
Pulling South
Pull Load versus Pile Ground LVDT Displacement



Ground LVDT Displacement (in)
 Figure A3.1: Pile 2 P-Δ Pulling South

Pile 2
Pulling North
Pull Load versus Pile Ground LVDT Displacement

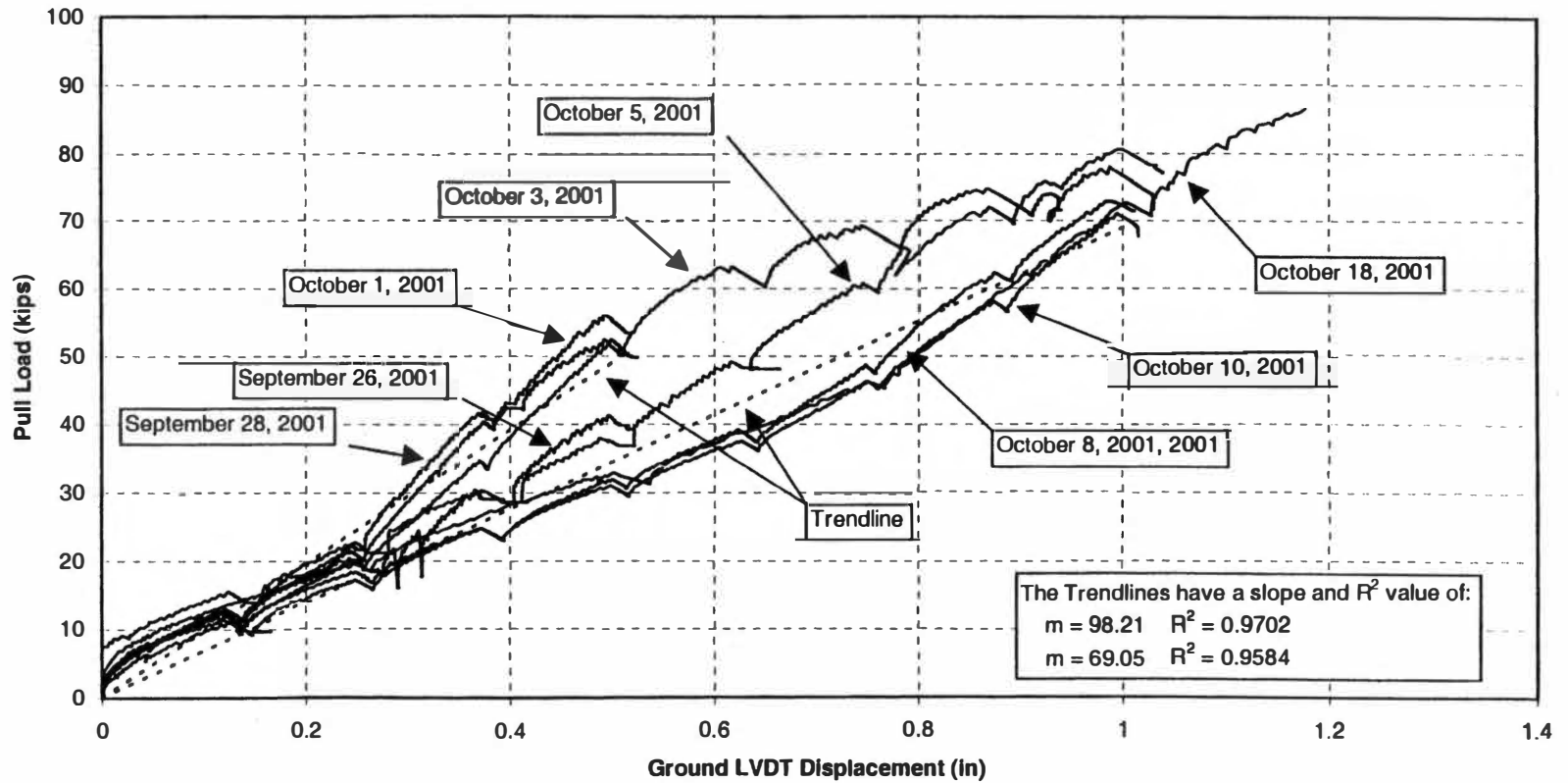


Figure A3.2: Pile 2 P-Δ Pulling North

Pile 2
Pulling North; Test to Failure
Pull Load versus Pile Ground LVDT Displacement

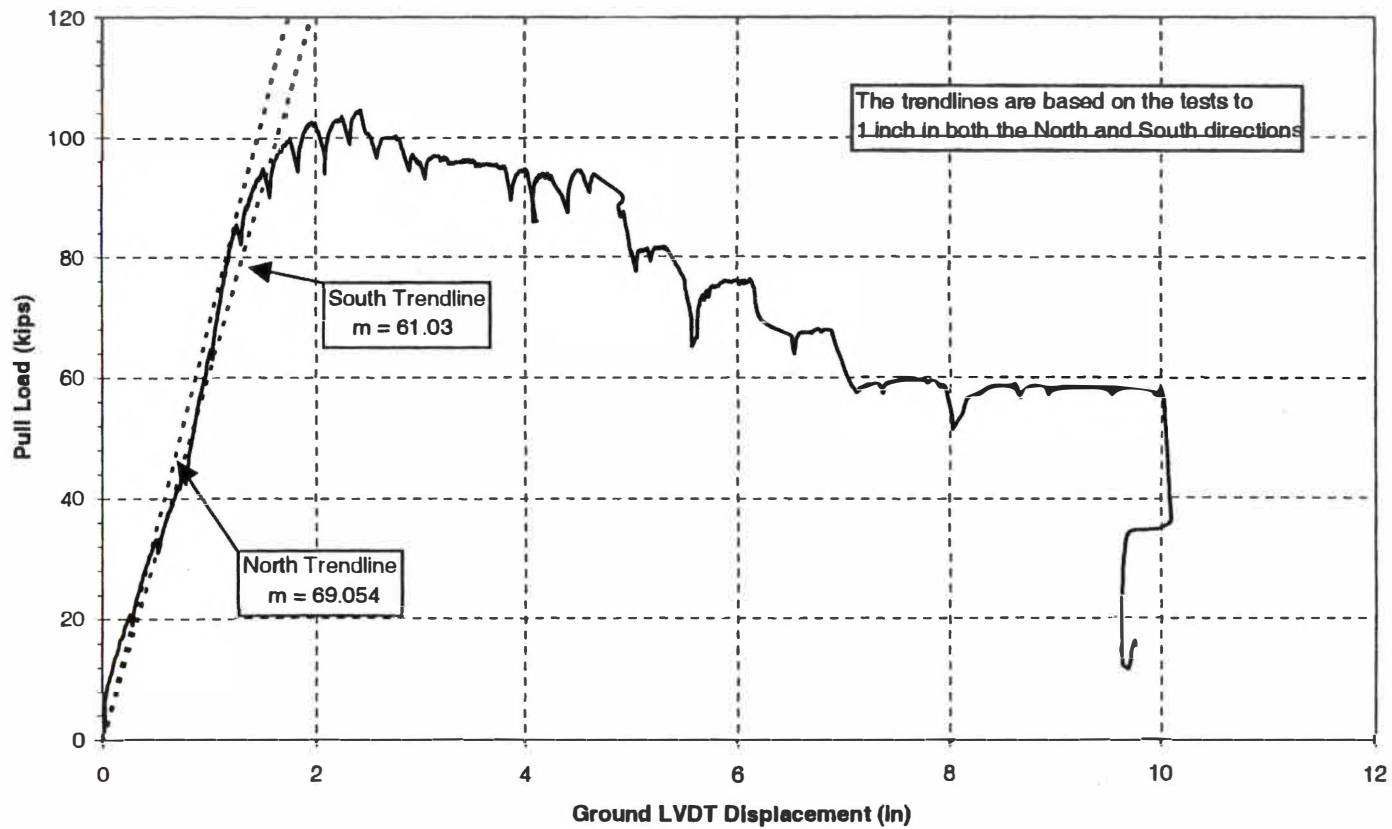


Figure A3.3: Pile 2 P- Δ Test to Failure

Pile 3
Pulling North
Pull Load versus Ground LVDT Displacement

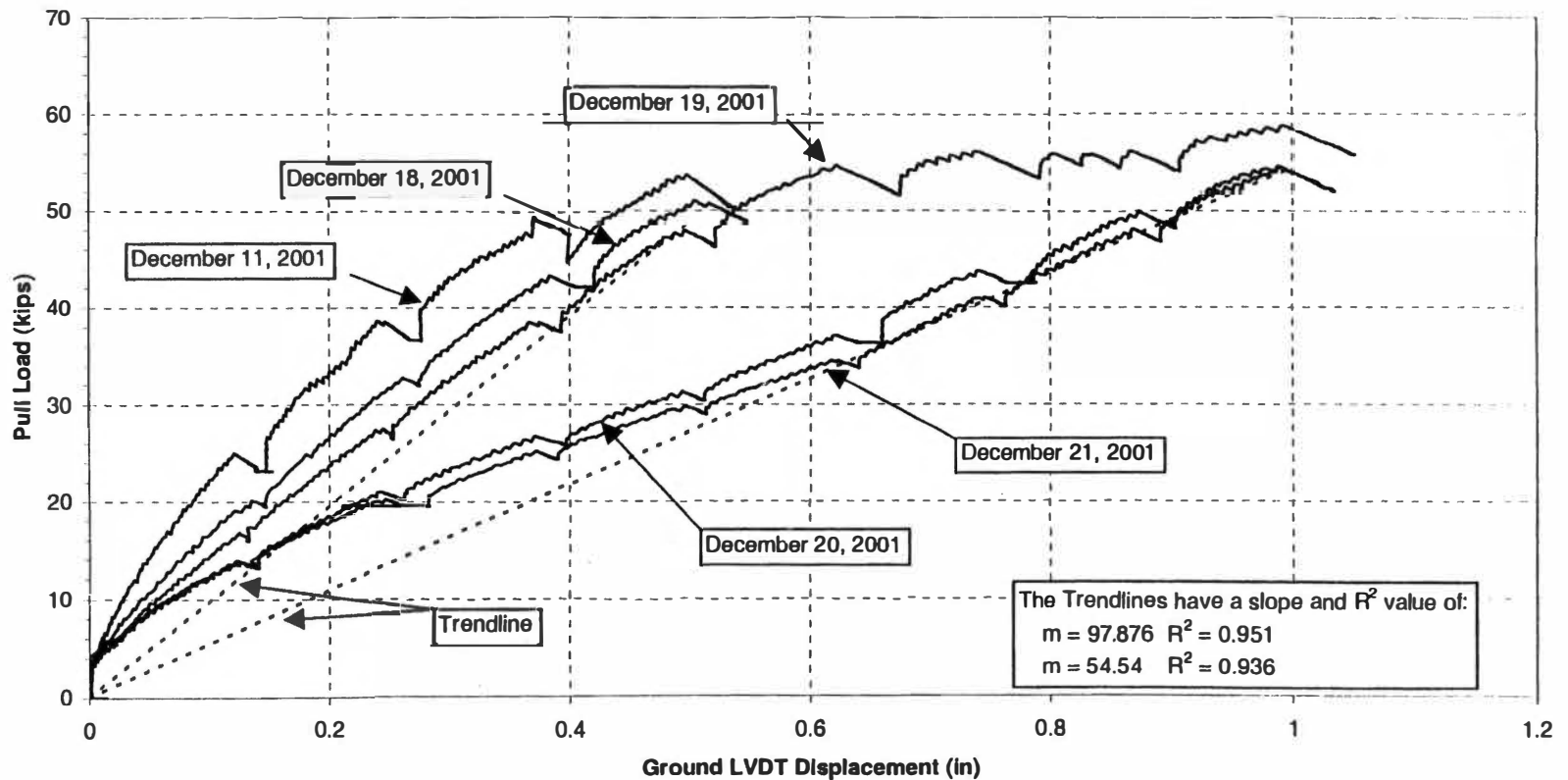


Figure A3.4: Pile 3 P-Δ Pulling North

**Pile 3
Pulling South
Pull Load versus Ground LVDT Displacement**

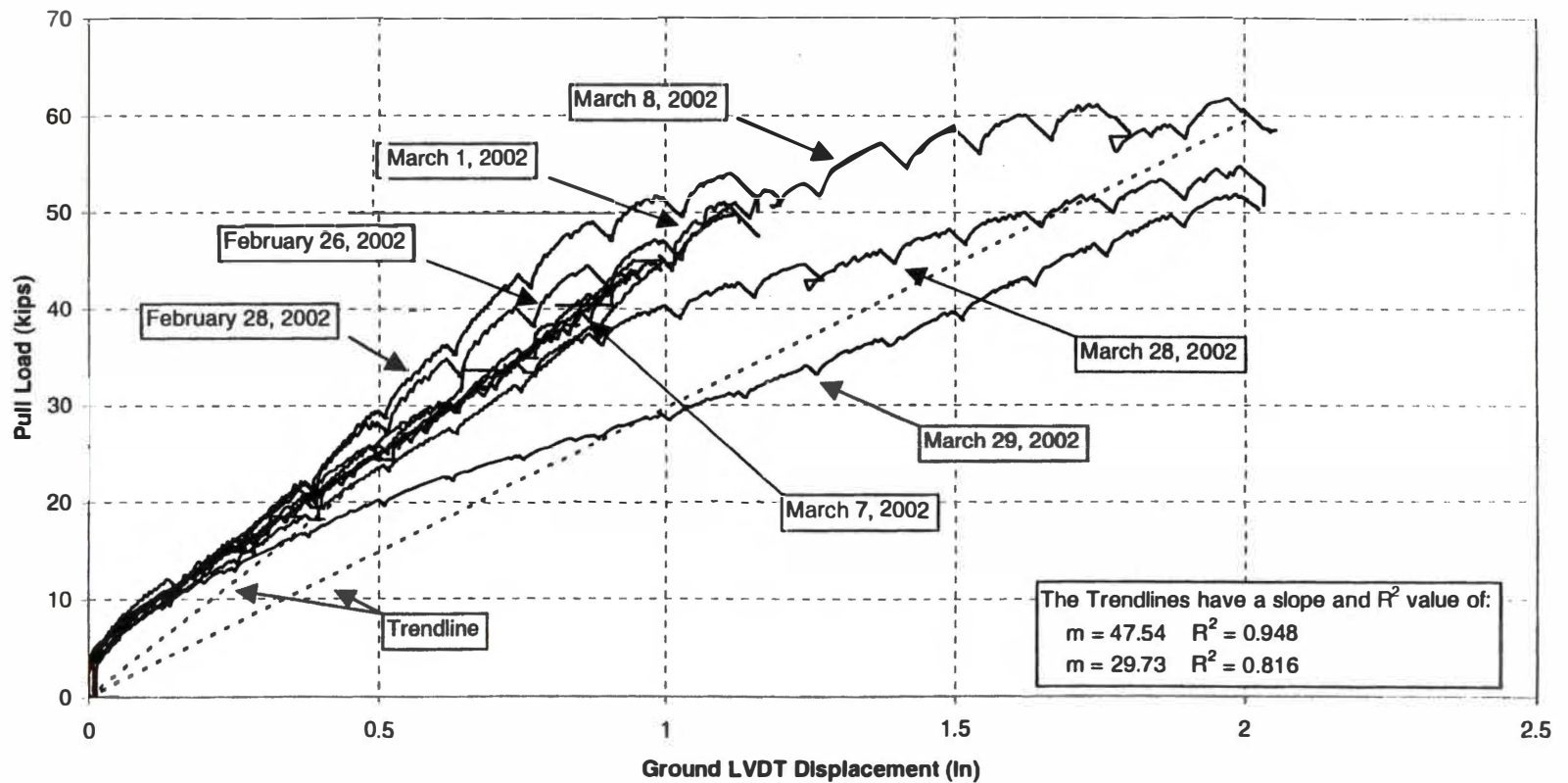


Figure A3.5: Pile 3 P-Δ Pulling South

Pile 3
Pulling South; Tests to Failure
Pull Load versus Ground LVDT Displacement

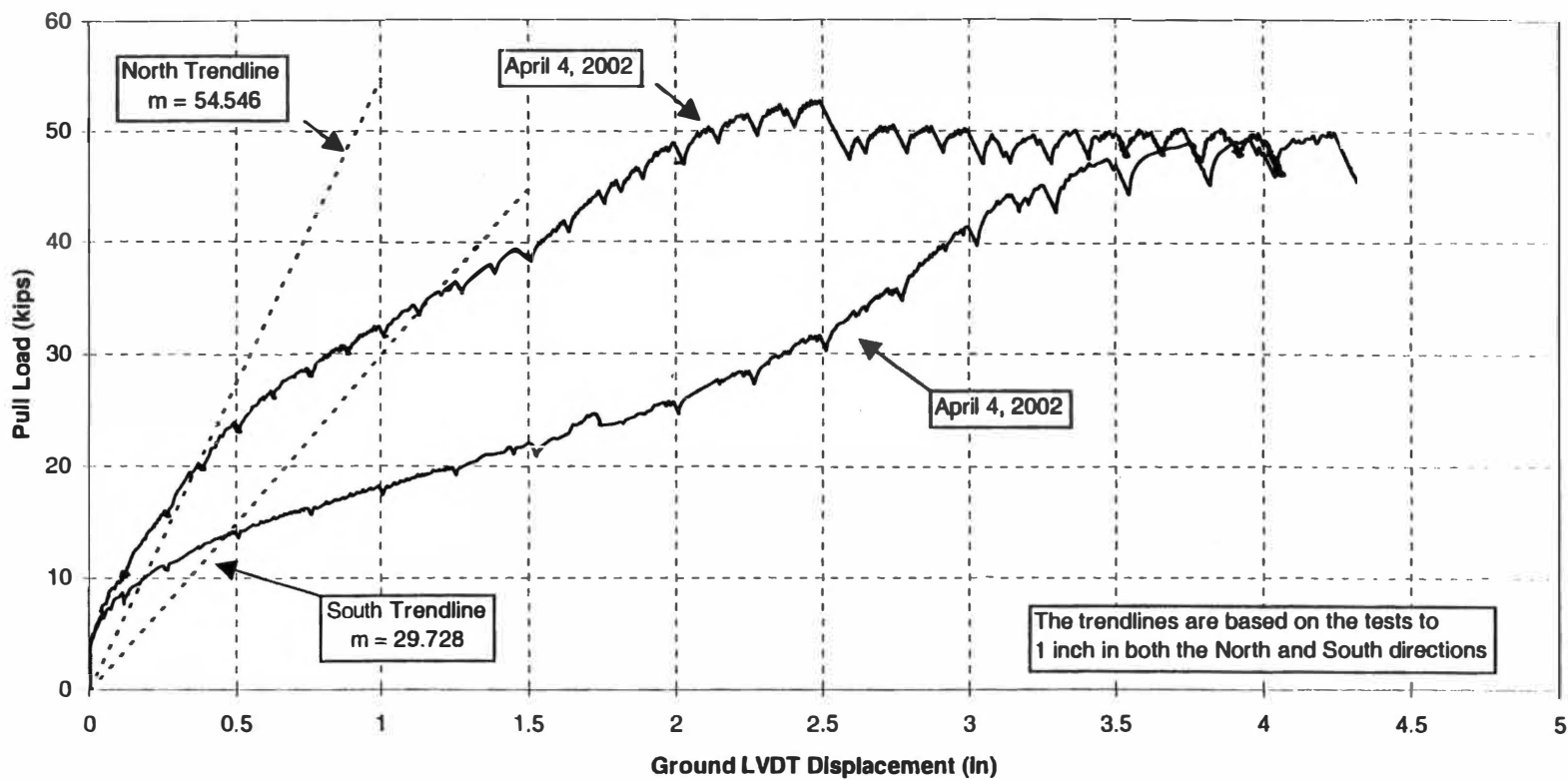


Figure A3.6: Pile 3 P-Δ Tests to Failure

Appendix 4

M- Δ Curves

Pile 2
Pulling South
Moment versus Pile Ground LVDT Displacement

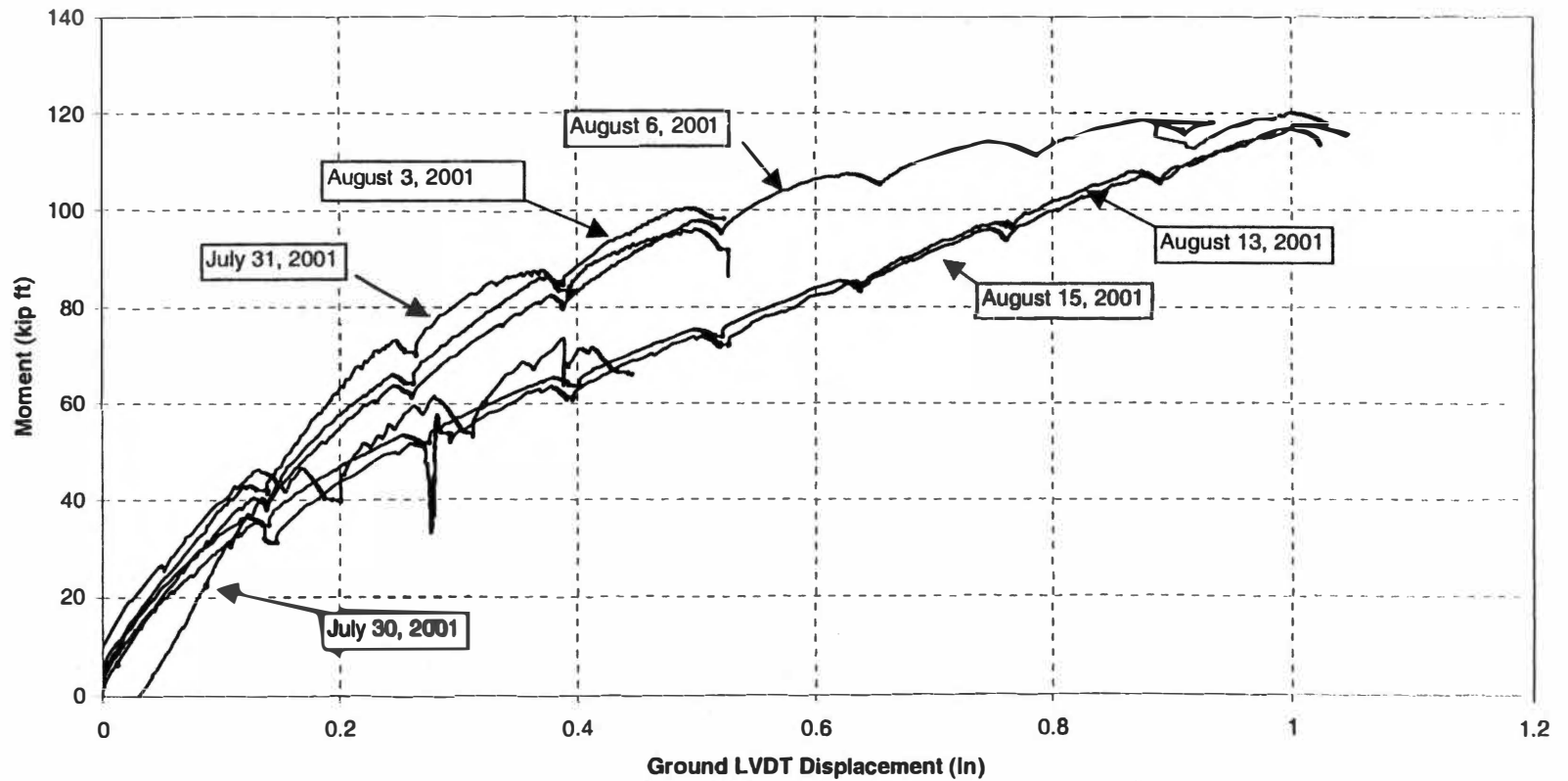


Figure A4.1: Pile 2 M-Δ Pulling South

Pile 2
Pulling North
Moment versus Pile Ground LVDT Displacement

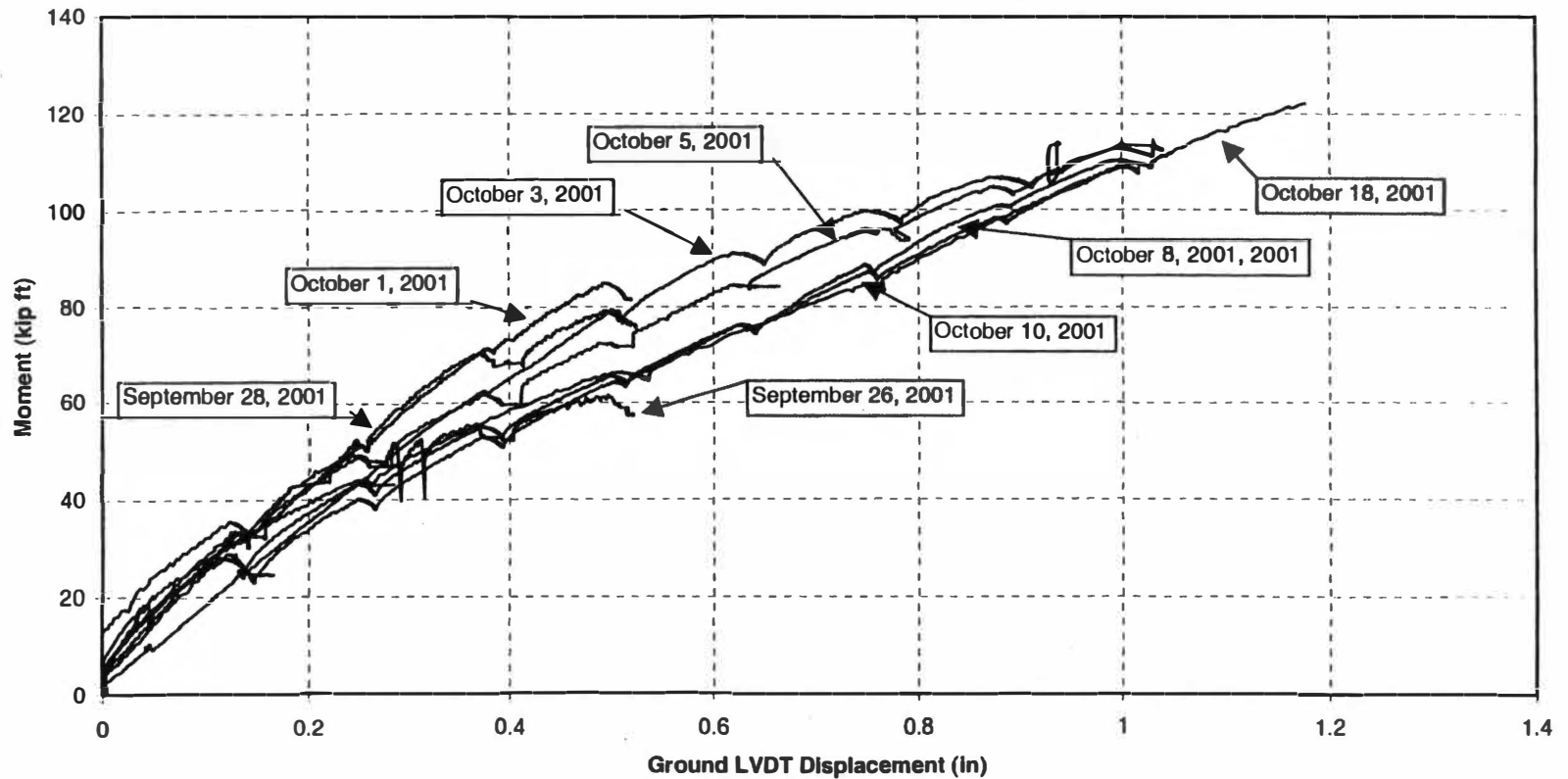


Figure A4.2: Pile 2 M-Δ Pulling North

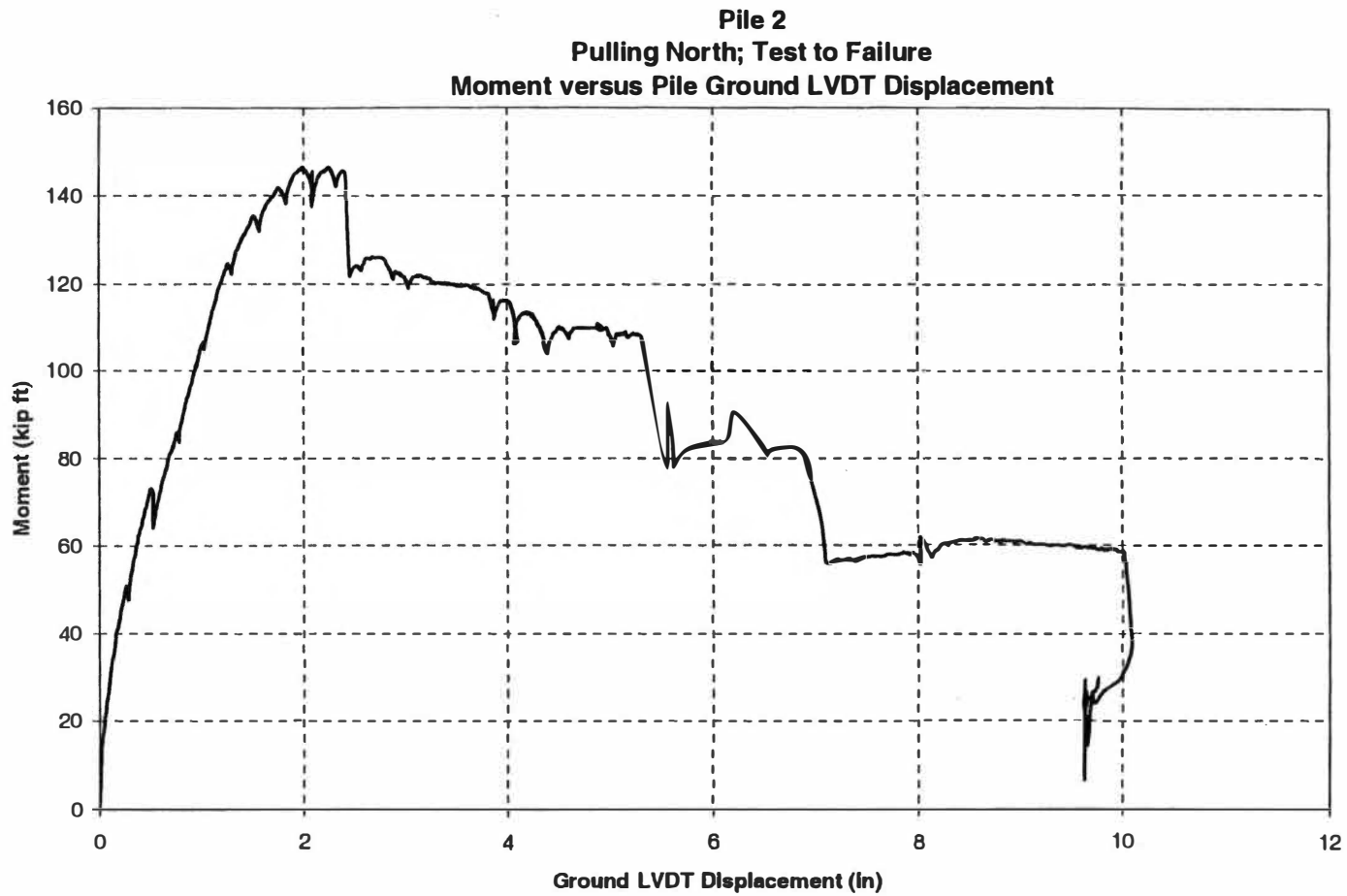


Figure A4.3: Pile 2 M- Δ Test to Failure

Pile 3
Pulling North
Moment versus Ground LVDT Displacement

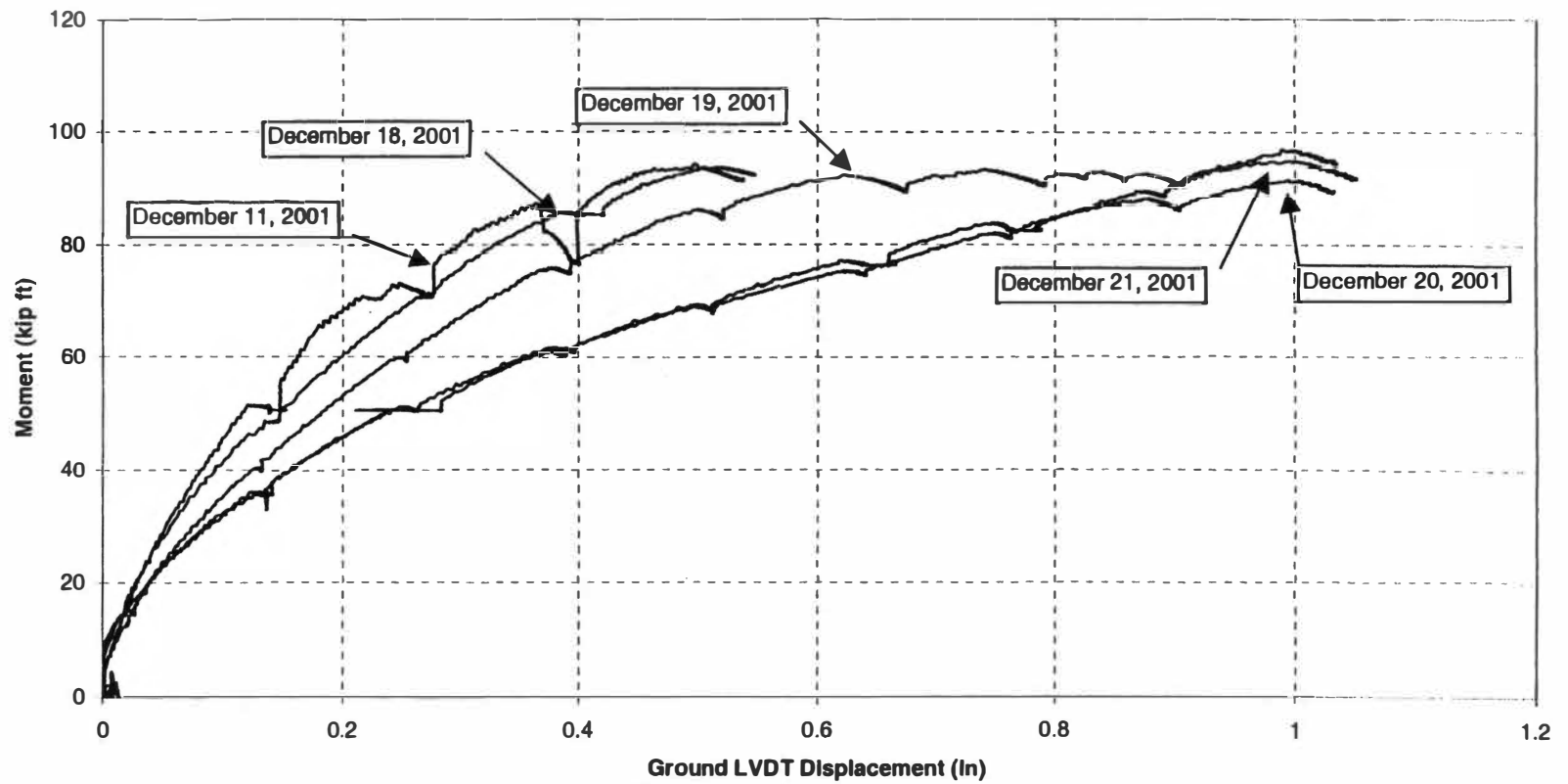


Figure A4.4: Pile 3 M-Δ Pulling North

**Pile 3
Pulling South
Moment versus Ground LVDT Displacement**

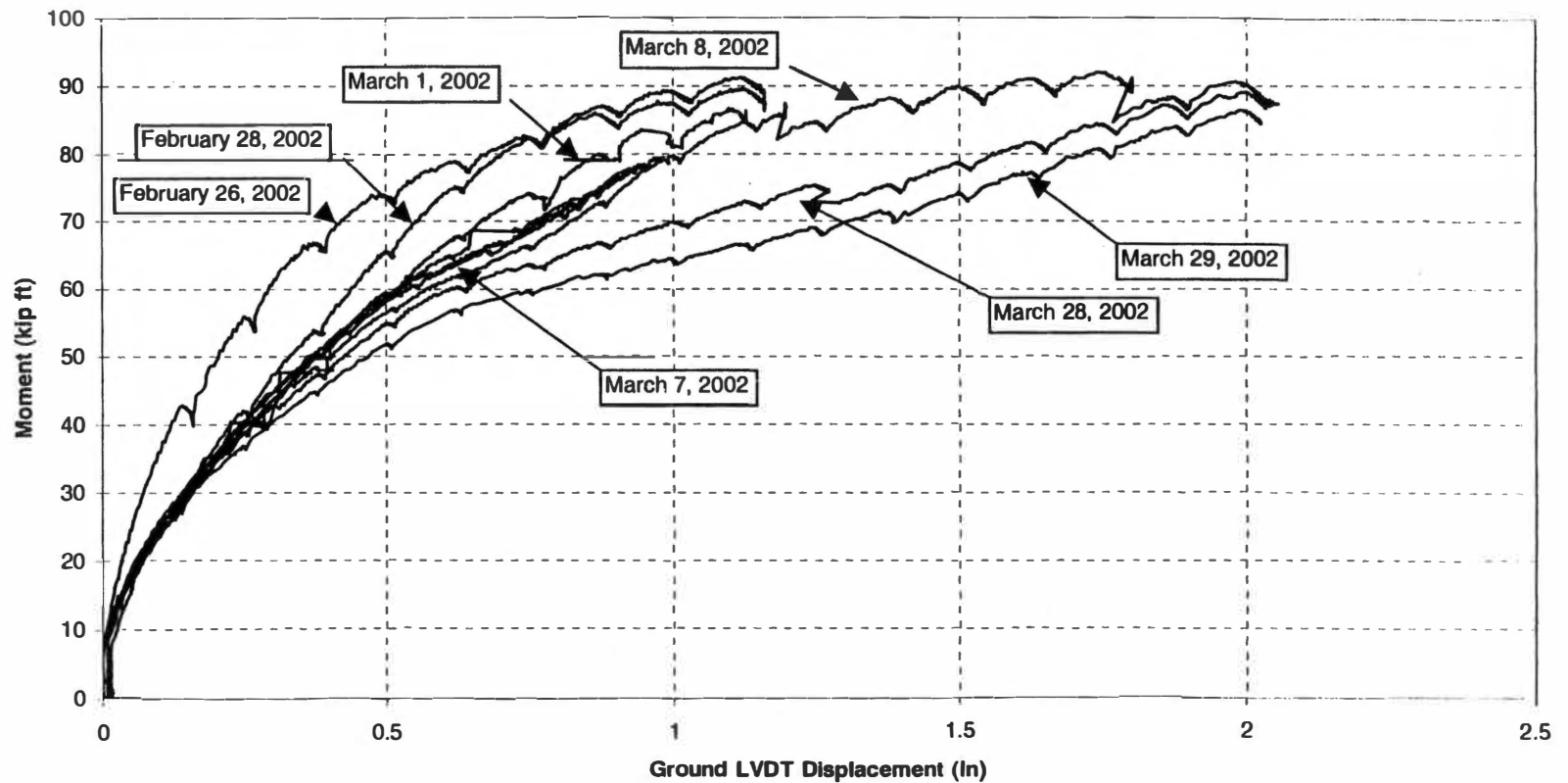


Figure A4.5: Pile 3 M- Δ Pulling South

Pile 3
Pulling South; Tests to Failure
Moment versus Ground LVDT Displacement

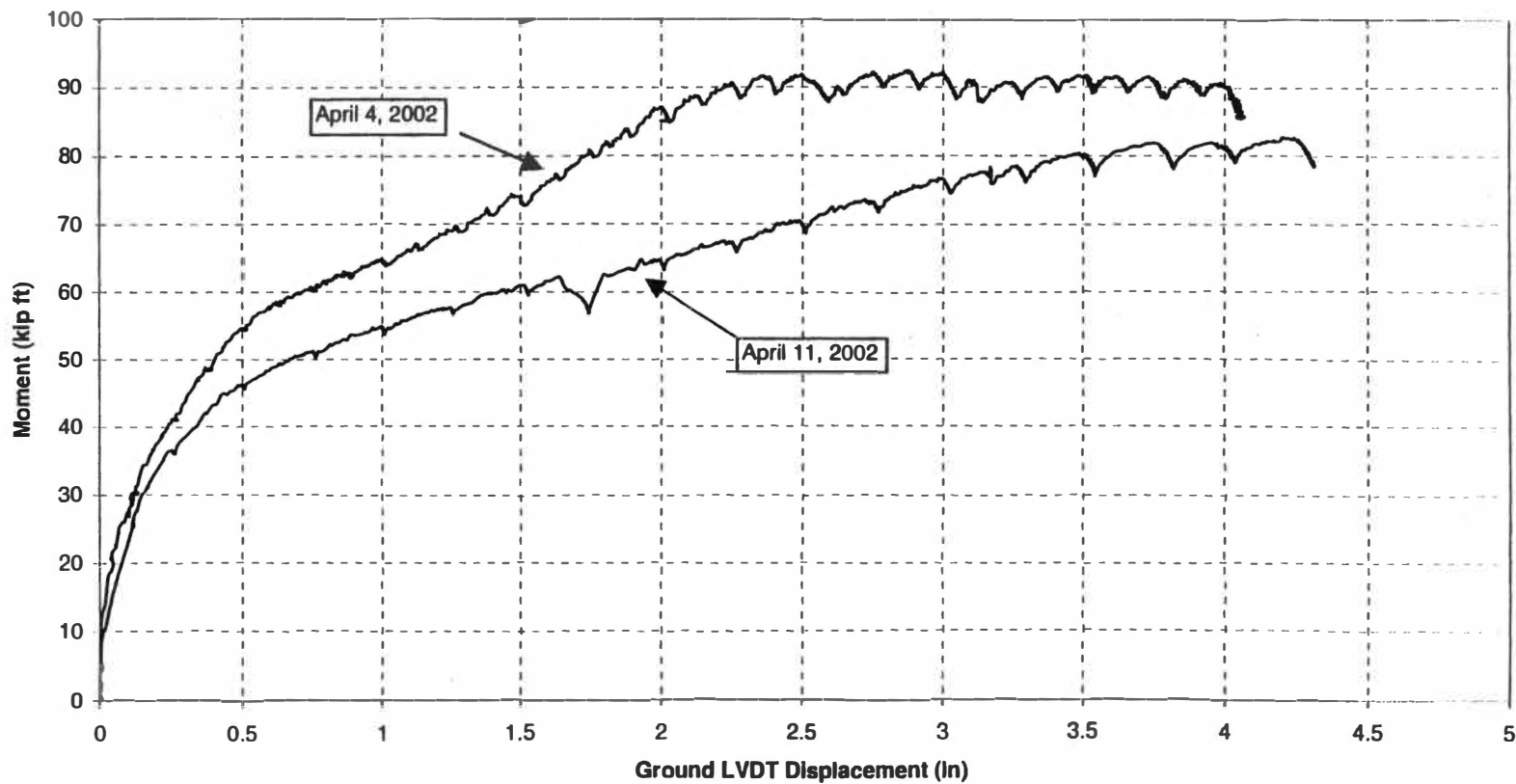


Figure A4.6: Pile 3 M-Δ Tests to Failure

Appendix 5

Sample Calculations

A5.1 Given Values (14)

0.5 inch diameter, seven wire, Grade 270, low relaxation prestressing strand

L_{total}	48	inches
$L_{embedded}$	12	inches
$L_{exposed}$	36	inches

f_{se}	165	ksi
f_{ps}	270	ksi
E_s	28500	ksi

d_b	0.5	inches
A_{ps}	0.153	sq. in.

f'_c	6000	psi
β_1	0.75	
β_2	0.85	

b	14	inches
h	14	inches
d	11.5	inches
d'	2.5	inches

Note: Please refer to Figure A5.1 or Nomenclature for term definitions

A5.2 Maximum Developable Strand Stress (14)

$f_{exposed}$

$$\begin{aligned} f_{exposed} &= \frac{L - 2.26}{0.198} \\ &= \frac{36 \text{ inches} - 2.26}{0.198} \\ &= 170.4 \text{ ksi} \end{aligned}$$

L_T

$$\begin{aligned} L_T &= \left(\frac{1}{3} \right) * f_{se} * D_{ps} \\ &= \left(\frac{1}{3} \right) * 165 \text{ ksi} * 0.5 \text{ inches} \\ &= 27.5 \text{ inches} \end{aligned}$$

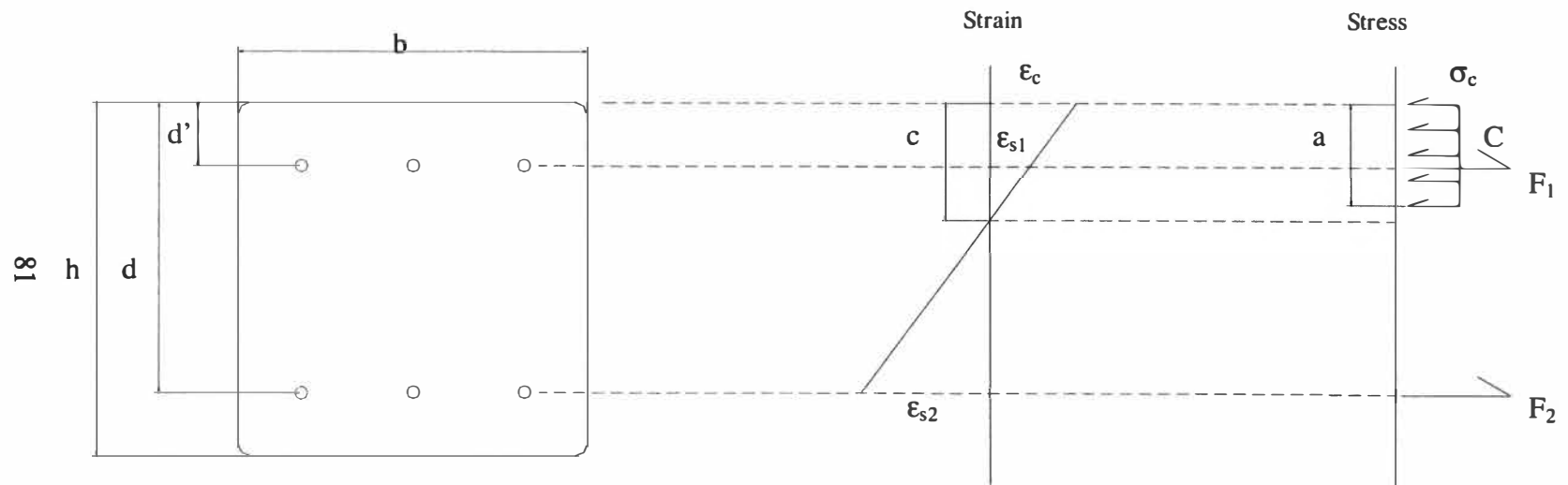


Figure A5.1: Sample calculation diagram

f_{embedded}

$$L \leq L_T$$

$$\begin{aligned} f_{\text{embedded}} &= L * \frac{f_{\text{se}}}{\frac{f_{\text{se}}}{3} * d_b} \\ &= 12 \text{ inches} * \frac{165 \text{ ksi}}{\frac{165 \text{ ksi}}{3} * 0.5 \text{ inches}} \\ &= 72 \text{ ksi} \end{aligned}$$

f_{total}

$$\begin{aligned} f_{\text{total}} &= f_{\text{exposed}} + f_{\text{embedded}} \\ &= 170.4 \text{ ksi} + 72 \text{ ksi} \\ &= 242.4 \text{ ksi} \end{aligned}$$

A5.3 Pile Zero Time Conditions (14)

ϵ_{se}

$$\begin{aligned} \epsilon_{\text{se}} &= \frac{f_{\text{se}}}{E_s} \\ &= \frac{165 \text{ ksi}}{28500 \text{ ksi}} \\ &= 0.005789 \end{aligned}$$

A_{ps}

$$\begin{aligned} A_{\text{ps}} &= 6 * 0.153 \text{ sq. in.} \\ &= 0.918 \text{ sq. in.} \end{aligned}$$

F_{ps}

$$\begin{aligned} F_{\text{ps}} &= A_{\text{ps}} * f_{\text{se}} \\ &= 0.918 \text{ sq. in.} * 165 \text{ ksi} \\ &= 151.47 \text{ kips} \end{aligned}$$

f_c

$$\begin{aligned} f_c &= \frac{F}{A_{pile}} \\ &= \frac{151.47 \text{ kips}}{(14 \text{ inches} * 14 \text{ inches})} \\ &= 0.7728 \text{ ksi} \end{aligned}$$

ϵ_{ce}

$$\begin{aligned} \epsilon_{ce} &= \frac{f_c}{E_c} \\ &= \frac{f_c}{57 * \sqrt{f_c}} \\ &= \frac{0.7728 \text{ ksi}}{57 * \sqrt{6000 \text{ psi}}} \\ &= 0.000175 \end{aligned}$$

A5.4 Pile Interaction Diagram Calculation (14)

ϵ_c 0.003

ϵ_{s2} 0.005

ϵ_{s1} and c Solved for using similar triangles

$$\begin{aligned} c &= \epsilon_c * \frac{d}{\epsilon_c + \epsilon_{s2}} \\ &= 0.003 * \frac{11.5 \text{ inches}}{0.003 + 0.005} \\ &= 4.31 \text{ inches} \end{aligned}$$

$$\begin{aligned} \epsilon_{s1} &= (c - d') * \frac{\epsilon_c}{c} \\ &= (4.31 \text{ inches} - 2.5 \text{ inches}) * \frac{0.003}{4.31 \text{ inches}} \\ &= 0.0013 \end{aligned}$$

a

$$\begin{aligned} a &= \beta_1 * c \\ &= 0.75 * 4.31 \text{ inches} \\ &= 3.23 \text{ inches} \end{aligned}$$

ϵ'_{s1}

$$\begin{aligned} \epsilon'_{s1} &= \epsilon_{se} - (\epsilon_{s1} - \epsilon_{ce}) \\ &= 0.005789 - (0.0013 - 0.000175) \\ &= 0.004664 \end{aligned}$$

ϵ'_{s2}

$$\begin{aligned} \epsilon'_{s2} &= \epsilon_{se} + \epsilon_{ce} + \epsilon_s \\ &= 0.005789 + 0.000175 + 0.005 \\ &= 0.01096 \end{aligned}$$

f_{s1}

$$\begin{aligned} f_{s1} &= \epsilon'_{s1} * E_s \\ &= 0.004664 * 28500 \text{ ksi} \\ &= 132.92 \text{ ksi} \end{aligned}$$

f_{s2}

$$\begin{aligned} f_{s2} &= \epsilon'_{s2} * E_s \\ &= 0.01096 * 28500 \text{ ksi} \\ &= 312.474 \text{ ksi} \geq f_{ps} \\ &= 270 \text{ ksi} \end{aligned}$$

F_1

$$\begin{aligned} F_1 &= f_{s1} * A_{ps1} \\ &= f_{s1} * (3 * A_{ps}) \\ &= 132.92 \text{ ksi} * (3 \text{ strands} * 0.153 \text{ sq. in.}) \\ &= 61.01 \text{ kips} \end{aligned}$$

F_2

$$\begin{aligned}
F_2 &= f_{s2} * A_{ps2} \\
&= f_{s2} * (3 * A_{ps}) \\
&= 270 \text{ ksi} * (3 \text{ strands} * 0.153 \text{ sq. in.}) \\
&= 123.93 \text{ kips}
\end{aligned}$$

C

$$\begin{aligned}
C &= (\beta_2 * f'_c) * a * b \\
&= (0.85 * 6 \text{ ksi}) * 3.23 \text{ inches} * 14 \text{ inches} \\
&= 230.62 \text{ kips}
\end{aligned}$$

P

$$\begin{aligned}
P &= C - F_1 - F_2 \\
&= 230.62 \text{ kips} - 61.01 \text{ kips} - 123.93 \text{ kips} \\
&= 45.68 \text{ kips}
\end{aligned}$$

M

$$\begin{aligned}
M &= \left[(F_2 - F_1) * \left(\frac{d - d'}{2} \right) + C * \left(\frac{h}{2} - \frac{a}{2} \right) \right] \left(\frac{1 \text{ foot}}{12 \text{ inches}} \right) \\
&= \left[(123.93 - 61.01) \text{ kips} * \left(\frac{11.5 - 2.5}{2} \right) \text{ inches} \right. \\
&\quad \left. + 230.62 \text{ kips} * \left(\frac{14}{2} - \frac{3.23}{2} \right) \text{ inches} \right] * \left(\frac{1 \text{ foot}}{12 \text{ inches}} \right) \\
&= (283.14 \text{ kip} - \text{in.} + 1241.889 \text{ kip} - \text{in.}) * \left(\frac{1 \text{ foot}}{12 \text{ inches}} \right) \\
&= 127.09 \text{ kip} - \text{ft.}
\end{aligned}$$

Appendix 6

Pseudo-Code

Initial Parameters

Prestress Tendon Inputs:	"Type of Strand" (text) "Grade" (ksi) "Diameter" (in) Cross Sectional "Area" (sq in) " f_{se} " (ksi) " E_s " (ksi) Area of Prestressing tendon in group 1 " $(A_{ps})_1$ " (sq in) Area of Prestressing tendon in group 2 " $(A_{ps})_2$ " (sq in) f_{ps} "limit" (ksi)
Pile Inputs:	Pile width "b" (in) Pile height "h" (in) Distance from pile top to first group of tendons "d'" (in) Distance from pile top to second group of tendons "d" (in) " f'_c " (psi) " β_1 "

$$E_c \text{ (ksi)} = (57000 * (\text{square root of } f'_c)) / 1000$$

Pure Compression

Input: Compressive strain " ϵ_c "

$$(A_{ps})_T \text{ (sq in)} = (A_{ps})_1 + (A_{ps})_2$$

$$F \text{ (kips)} = (A_{ps})_T * f_{se}$$

$$\epsilon_{se} = f_{se} / E_s$$

$$f_c \text{ (ksi)} = F / (b * h)$$

$$\epsilon_{ce} = f_c / E_c$$

$$\Delta\epsilon_c = \epsilon_c - \epsilon_{ce}$$

$$\text{IF } (\epsilon_{se} - \Delta\epsilon_c > 0)$$

$$\epsilon_s = \epsilon_{se} - \Delta\epsilon_c$$

$$\text{IF } (\epsilon_{se} - \Delta\epsilon_c \leq 0)$$

$$\epsilon_s = 0$$

$$f_s \text{ (ksi)} = \epsilon_s * E_s$$

$$T \text{ (kips)} = f_s * (A_{ps})_T$$

$$f_c \text{ (ksi)} = T / (b * h)$$

$$P \text{ (kips)} = (b * h) * ((0.85 * (f'_c / 1000)) - f_c)$$

$$M \text{ (kip-ft)} = 0$$

$$\text{RECORD } (P, M)$$

$$c = h$$

$$c \text{ (in)} = h$$

$$\text{IF } (\epsilon_{se} - (((0.003 / h) * (h - d')) - \epsilon_{ce}) > 0$$

$$\epsilon_{s2} = (\epsilon_{se} - (((0.003 / h) * (h - d')) - \epsilon_{ce}))$$

$$\text{IF } (\epsilon_{se} - (((0.003 / h) * (h - d')) - \epsilon_{ce}) \leq 0$$

$$\epsilon_{s2} = 0$$

$$\text{IF } (\epsilon_{se} - (((0.003 / h) * (h - d)) - \epsilon_{ce}) > 0$$

$$\epsilon_{s1} = (\epsilon_{se} - (((0.003 / h) * (h - d)) - \epsilon_{ce}))$$

$$\text{IF } (\epsilon_{se} - (((0.003 / h) * (h - d)) - \epsilon_{ce}) \leq 0$$

$$\epsilon_{s1} = 0$$

$$\text{IF } (\epsilon_{s2} * E_s) < \text{limit}$$

$$f_{s2} \text{ (ksi)} = (\epsilon_{s2} * E_s)$$

$$\text{IF } (\epsilon_{s2} * E_s) \geq \text{limit}$$

$$f_{s2} \text{ (ksi)} = \text{limit}$$

$$\text{IF } (\epsilon_{s1} * E_s) < \text{limit}$$

$$f_{s1} \text{ (ksi)} = (\epsilon_{s1} * E_s)$$

$$\text{IF } (\epsilon_{s1} * E_s) \geq \text{limit}$$

$$f_{s1} \text{ (ksi)} = \text{limit}$$

$$T_2 \text{ (kips)} = f_{s2} * (A_{ps})_2$$

$$T_1 \text{ (kips)} = f_{s1} * (A_{ps})_1$$

$$a \text{ (in)} = c * \beta_1$$

$$C \text{ (kips)} = ((0.85 * f'_c) / 1000) * a * b$$

$$P \text{ (kips)} = C - T_2 - T_1$$

$$M \text{ (kip-ft)} = (1/12) * (((T_1 - T_2) * ((d - d') / 2)) + (C * ((h / 2) - (a / 2))))$$

RECORD (P,M)

$$c = d$$

$$c \text{ (in)} = d$$

$$\text{IF } (\epsilon_{se} - (((0.003 / d) * (d - d')) - \epsilon_{ce}) > 0$$

$$\epsilon_{s2} = (\epsilon_{se} - (((0.003 / d) * (d - d')) - \epsilon_{ce})$$

$$\text{IF } (\epsilon_{se} - (((0.003 / d) * (d - d')) - \epsilon_{ce}) \leq 0$$

$$\epsilon_{s2} = 0$$

$$\epsilon_{s1} = \epsilon_{se} - (0 - \epsilon_{ce})$$

$$f_{s2} \text{ (ksi)} = (\epsilon_{s2} * E_s)$$

$$\text{IF } (\epsilon_{s1} * E_s) < \text{limit}$$

$$f_{s1} \text{ (ksi)} = (\epsilon_{s1} * E_s)$$

$$\text{IF } (\epsilon_{s1} * E_s) \geq \text{limit}$$

$$f_{s1} \text{ (ksi)} = \text{limit}$$

$$T_2 \text{ (kips)} = f_{s2} * (A_{ps})_2$$

$$T_1 \text{ (kips)} = f_{s1} * (A_{ps})_1$$

$$a \text{ (in)} = c * \beta_1$$

$$C \text{ (kips)} = ((0.85 * f'_c) / 1000) * a * b$$

$$P \text{ (kips)} = C - T_2 - T_1$$

$$M \text{ (kip-ft)} = (1/12) * (((T_1 - T_2) * ((d - d') / 2)) + (C * ((h / 2) - (a / 2))))$$

RECORD (P,M)

$$\epsilon_{cs1} = \text{---}$$

Input:

Starting strain “ ϵ_{cs1} ”

Final Strain “ ϵ_{cs2} ”

Strain Increase interval “ $\Delta\epsilon$ ”

WHEN ($\epsilon_{cs2} > \epsilon_{cs1}$)

$$c \text{ (in)} = (d / (0.003 + \epsilon_{cs1})) * 0.003$$

$$\text{IF } (\epsilon_{se} - (((0.003 / c) * (c - d')) - \epsilon_{ce}) > 0$$

$$\epsilon_{s2} = (\epsilon_{se} - (((0.003 / c) * (c - d')) - \epsilon_{ce})$$

$$\text{IF } (\epsilon_{se} - (((0.003 / c) * (c - d')) - \epsilon_{ce}) \leq 0$$

$$\epsilon_{s2} = 0$$

$$\epsilon_{s1} = \epsilon_{se} + \epsilon_{ce} + \epsilon_{cs1}$$

$$\text{IF } (\epsilon_{s2} * E_s) < \text{limit}$$

$$f_{s2} \text{ (ksi)} = (\epsilon_{s2} * E_s)$$

$$\text{IF } (\epsilon_{s2} * E_s) \geq \text{limit}$$

$$f_{s2} \text{ (ksi)} = \text{limit}$$

$$\text{IF } (\epsilon_{s1} * E_s) < \text{limit}$$

$$f_{s1} \text{ (ksi)} = (\epsilon_{s1} * E_s)$$

$$\text{IF } (\epsilon_{s1} * E_s) \geq \text{limit}$$

$$f_{s1} \text{ (ksi)} = \text{limit}$$

$$T_2 \text{ (kips)} = f_{s2} * (A_{ps})_2$$

$$T_1 \text{ (kips)} = f_{s1} * (A_{ps})_1$$

$$a \text{ (in)} = c * \beta_1$$

$$C \text{ (kips)} = ((0.85 * f'_c) / 1000) * a * b$$

$$P \text{ (kips)} = C - T_2 - T_1$$

$$M \text{ (kip-ft)} = (1/12) * (((T_1 - T_2) * ((d - d') / 2)) + (C * ((h / 2) - (a / 2))))$$

$$\text{RECORD (P,M)}$$

$$\epsilon_{cs1} = \epsilon_{cs1} + \Delta\epsilon$$

$$\text{GRAPH (P,M)}$$

END

Appendix 7

Pile Removal Report

A7.1 Introduction

All testing on Pile 1 was completed by June 27, 2001. At the completion of testing, cracks were clearly visible in the pile below the abutment interface. The abutment/slab assembly was still fully supported by the pile with no noticeable displacement in the assembly other than vertical settlement that occurred over the course of testing. Testing on Pile 2 was completed by October 23, 2001 with a test to ultimate failure (Figure A7.1). The pile showed extensive damage. The abutment/slab assembly was displaced 10 inches to the north and rested on the ground with no support from the pile. A complete concrete compressive failure was visible at the abutment interface. Large pieces of the southern side of the pile also broke off in the failure (Figures A7.2 and A7.3).

A7.2 Slab and Abutment Removal

In preparation for the removal of the concrete slabs and abutments, wood falsework was constructed to support the slabs and abutments as they were cut free of the pile. In the case of Pile 1, the abutment was also shored for cutting. Tru-Line bored eight 4 inch diameter holes completely through each wing of the slabs to aid in their future removal before beginning to cut. In order to remove the slabs, three cuts were made in each assembly (Figure A7.4). One continuous cut was made through the entire depth of the slab running in a north to south direction. Two cuts were then made through the slab depth running in the east to west direction 1 foot 6 inches to the north and south of the pile center. This offset corresponded to the edge of the abutment. The four blocks



Figure A7.1: Pile 1 Interface

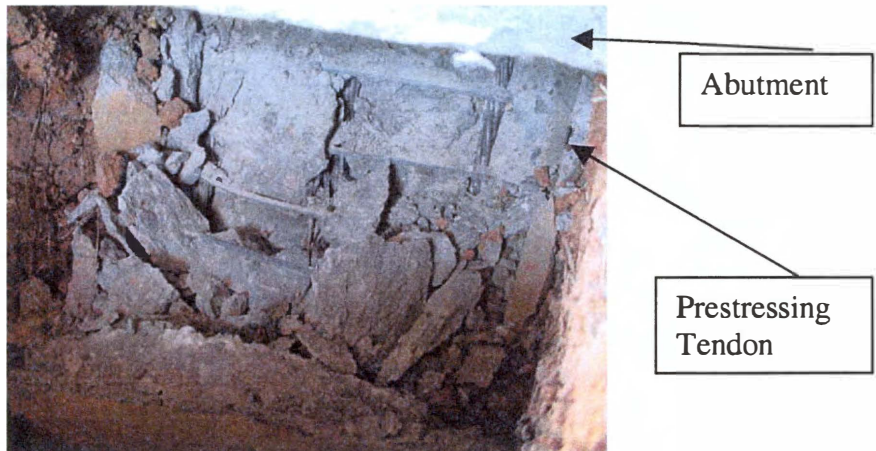


Figure A7.2: Pile 2 Interface, South Side

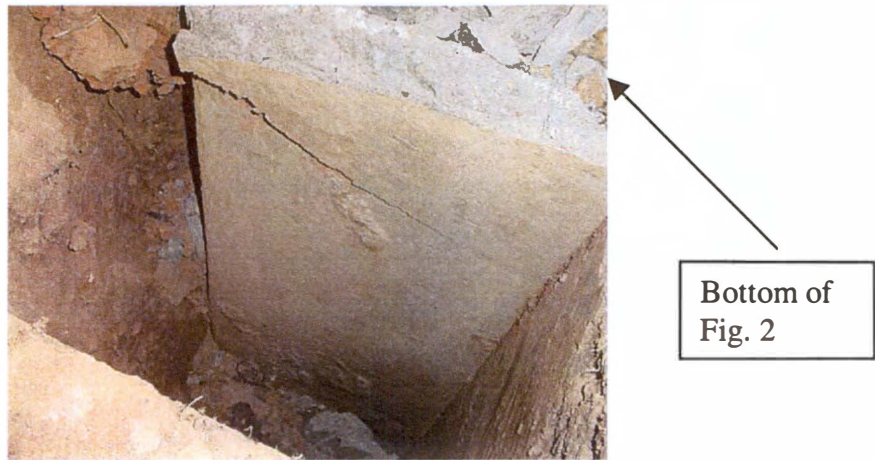


Figure A7.3: Pile 2 Failure, South Side

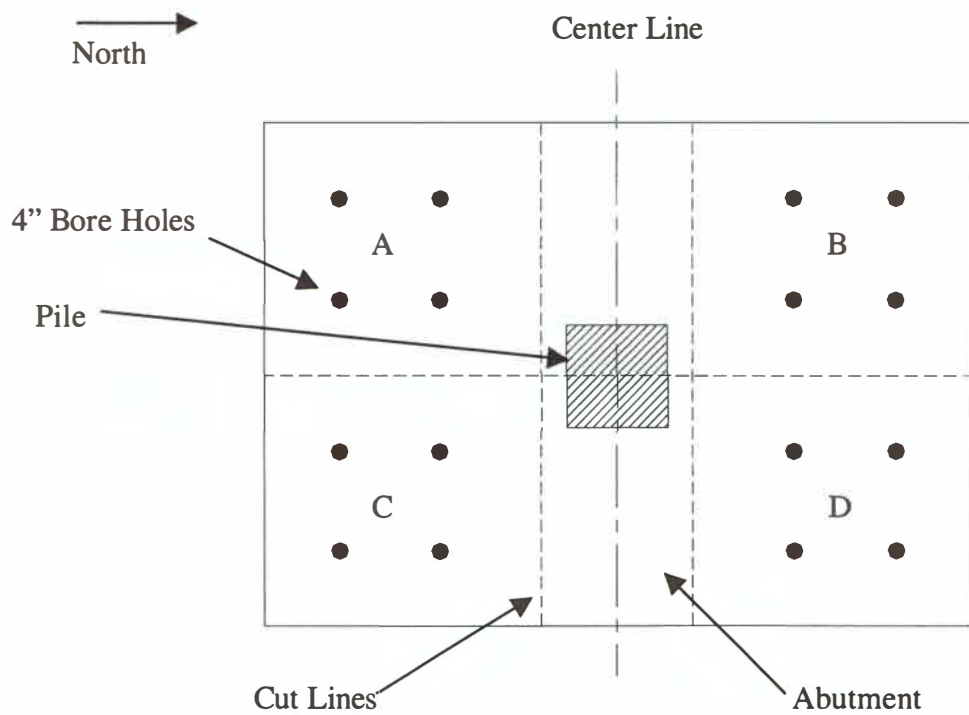


Figure A7.4: Plan View of Slab Cut Pattern

from each pile, A, B, C, and D, were then removed by a crane and stacked on site for future disposal.

With blocks A, B, C, and D removed, the false work was disassembled and removed. Tru-Line made three more cuts in order to remove the remaining abutment/slab assembly, blocks E, F, G, H, I, and J, from the pile tops. The first cut was a horizontal cut 12 inches above the bottom of the abutment. Two vertical cuts were then made 18 inches to the east and west of the pile center (Figure A7.5). Two 12 inch long, 1 inch diameter anchor bolts were then driven into each of the 5 blocks on each pile in order to be used as lift points. However, when the blocks were removed, it was discovered that several of the Tru-Line cuts were incomplete. In the case of Pile 1, blocks E, F, and G were lifted as a single block (Figure A7.6). Blocks H and J were removed without incident or apparent damage to the pile abutment interface. Blocks E, F, and G on Pile 2 were removed without incident. The vertical cuts below the horizontal cut were incomplete, and a reinforcing bar running east to west was left uncut as well as a portion of the concrete (Figure A7.7). This situation was discovered when the western block, J was removed. The southern anchor bolt sheared off before the reinforcing bar failed during the lift. A chain was used as an cinch around the block and pulled taut until the bar failed along the plane of the cut. When the eastern block, H, was removed, the anchor bolts did not fail, but the reinforcing bar was pulled out of the abutment pile interface. Extensive damage was done to the concrete (Figures A7.8 and A7.9).

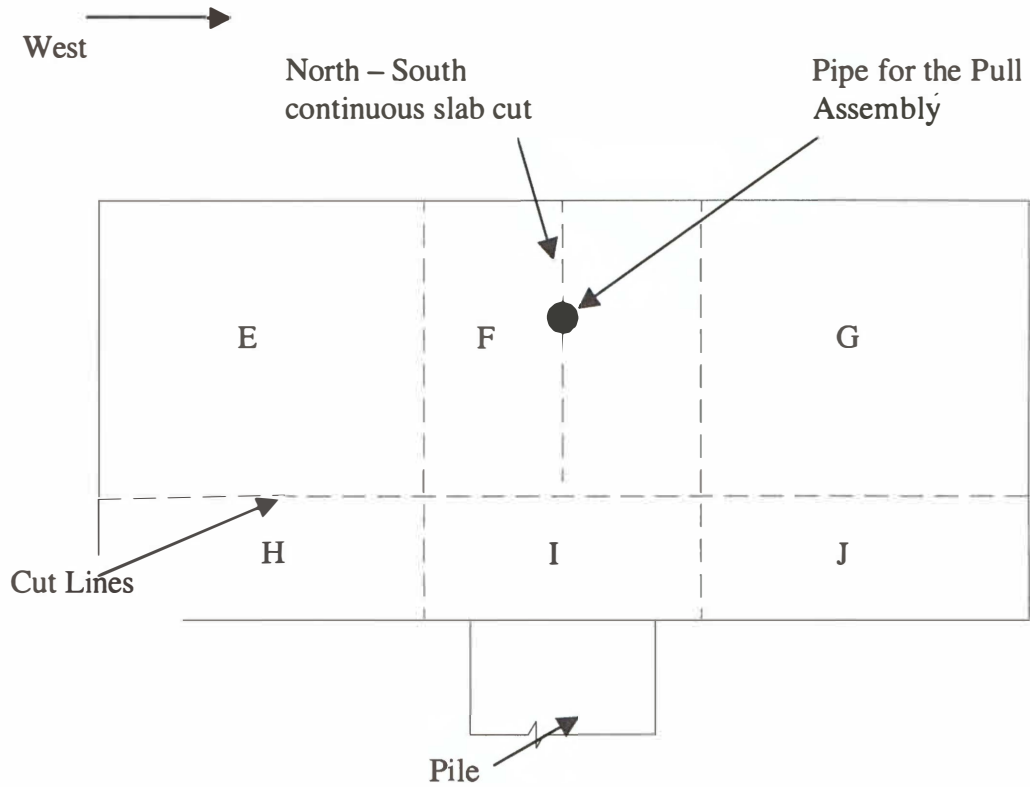


Figure A7.5: Side View of Cut Pattern

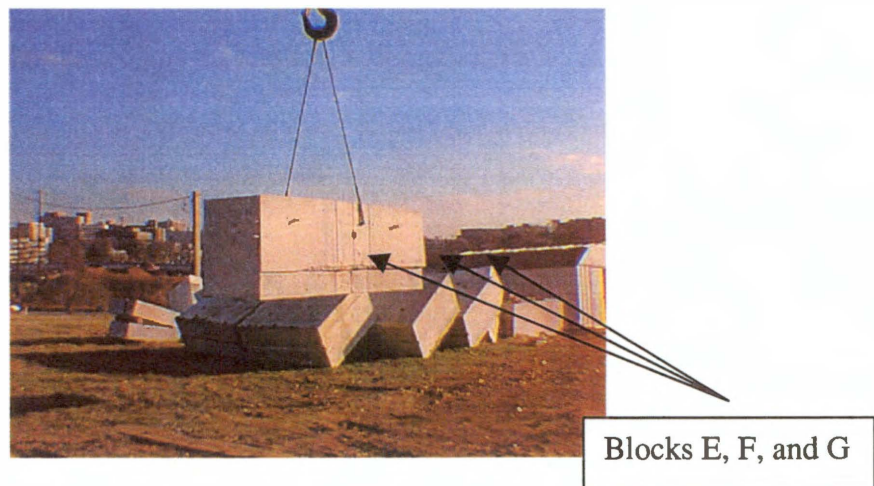


Figure A7.6: Pile 1 Slab/Abutment Assembly during Removal

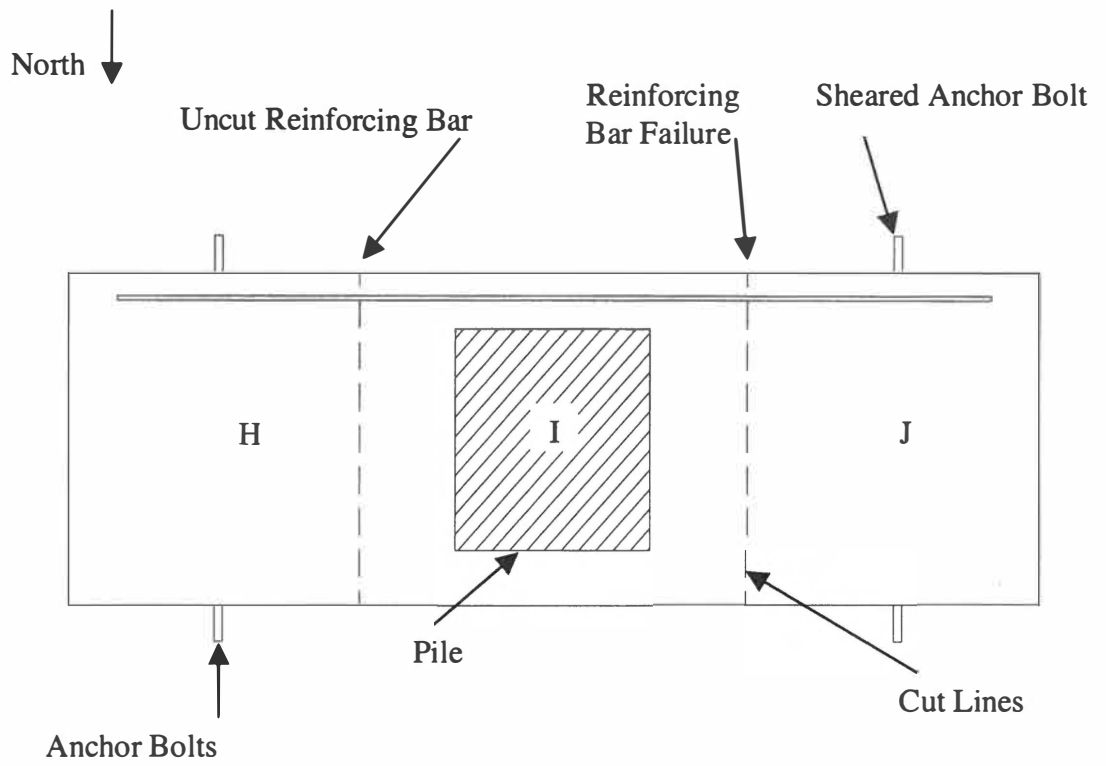


Figure A7.7: Plan View Pile 2 Incomplete Cuts



Figure A7.8: Pile 2 Interface 1



Figure A7.9: Pile 2 Interface 2

A7.3 Pile Excavation

Both Piles 1 and 2 were excavated with the use of a track hoe operated by Britton Bridge. In the case of Pile 1, the pile was excavated on all four sides to a depth of 14 feet to 15 feet (Figure A7.10). The pile was scraped during excavation, but care was taken to prevent any structural damage. Once the pile was excavated, a chain was secured to the pile and held taut by the crane. The pile was cut 12 feet 3 inches below the bottom of the abutment beginning on the south side and working around the pile west, north, and east. The south and north cuts were 2 ½ inches to 2 ¾ inches deep in order to cut all 6 prestressing tendons. The east and west cuts were made to unspecified depths in order to attempt to reduce moment capacity. Once the cuts were complete, the base of the pile was knocked free using the track hoe, while the top was still held in place by the crane. The entire assembly was then placed on blocks for future transportation.

Pile 2 was heavily damaged during the test to failure, and the pile abutment interface was further damaged during the removal of blocks H and J (Figure A7.7). The pile was completely crushed just below the abutment. For safety reasons, the tendons holding block I were cut and the piece removed (Figures A7.11 and A7.12). A second failure in the pile was visible after testing that varied in depth from 1 foot 11 inches below the abutment on the southern side to 2 feet 2 inches on the northern side. The region above this failure was heavily damaged.

During excavation a third failure was discovered in the pile at 5 feet 4 inches below the abutment. This region was previously hidden from sight, but concrete paste

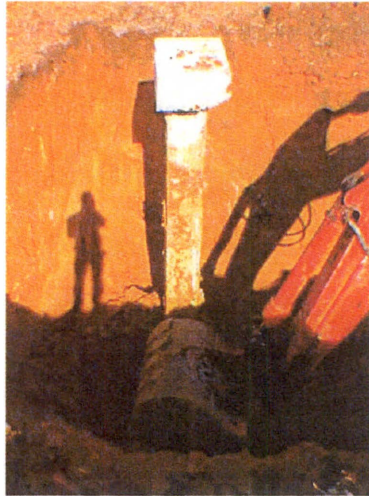


Figure A7.10: Pile 1 Excavation

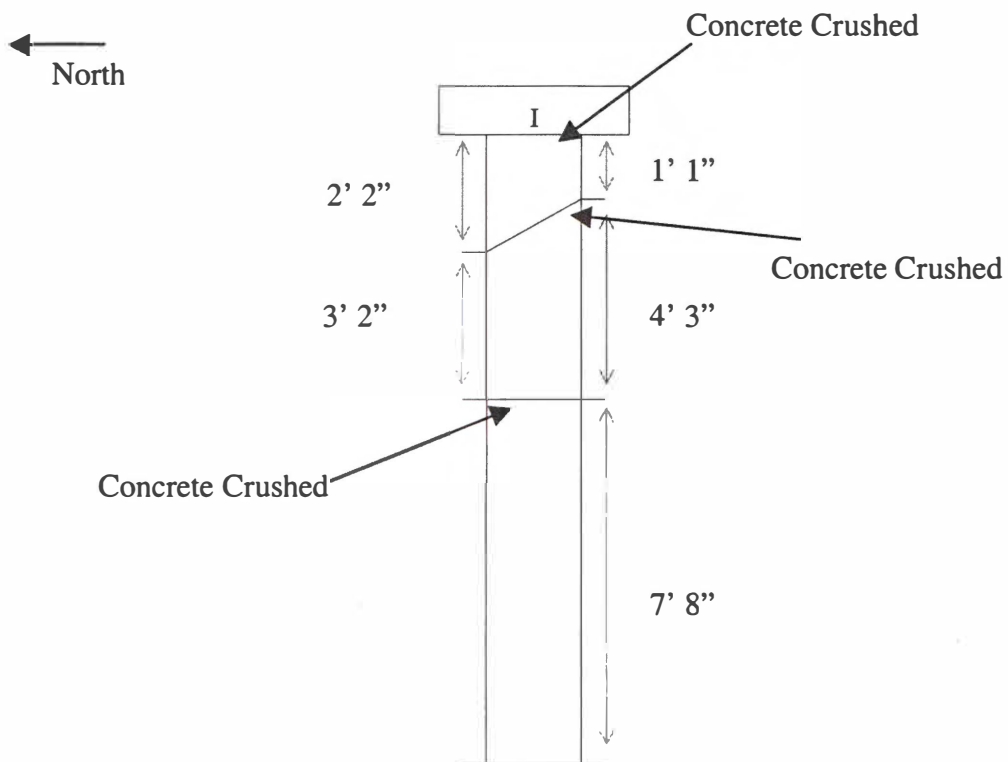


Figure A7.11: Side View Pile 2

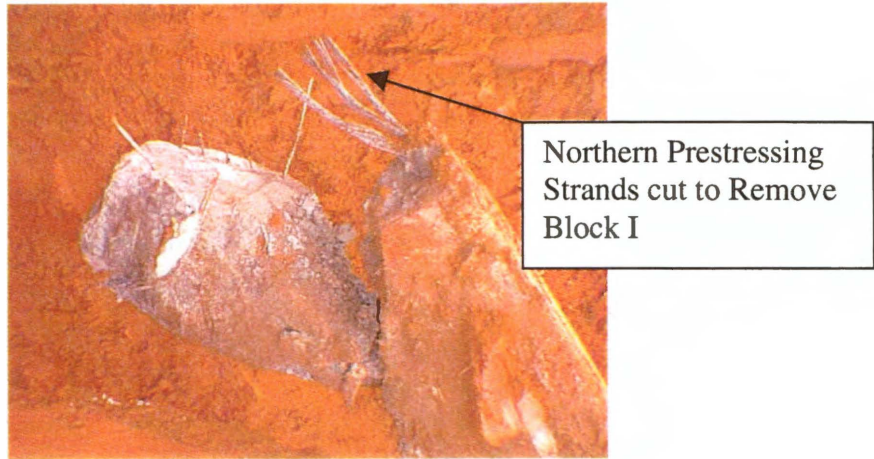


Figure A7.12: Pile 2 Failure 1

from the Tru-Line cutting was visible in the crack (Figure A7.13). It was also apparent that the 3 southern tendons had failed in tension during the test to failure. The section above the third failure was unable to support its own weight once the soil around it was removed (Figure A7.14). Consequently, the 3 northern tendons were cut using a hand-held electric grinder, and the pile was removed. The remaining pile was then excavated to the maximum depth allowed by the track hoe and cut 7 feet 8 inches below the third failure (Figure A7.15). The section of remaining pile was held taut by the crane, and the southern, western, and northern sides were cut in that order. The southern and northern sides were cut 2 ½ inches to 2 ¾ inches deep and the western side to an unspecified depth. When the track hoe was used to knock the base free, it was discovered that the southwest and south central tendons were uncut. In order to cut the tendons, the pile was laid down to the south and the tendons exposed with a sledgehammer. The tendons were then cut and the pile removed.

Both excavations were filled immediately upon removal of the piles. The piles were moved to the Structural Laboratory in Perkins Hall on The University of Tennessee Campus on January 26 and 29, 2002, with the help of a Britton Bridge truck.

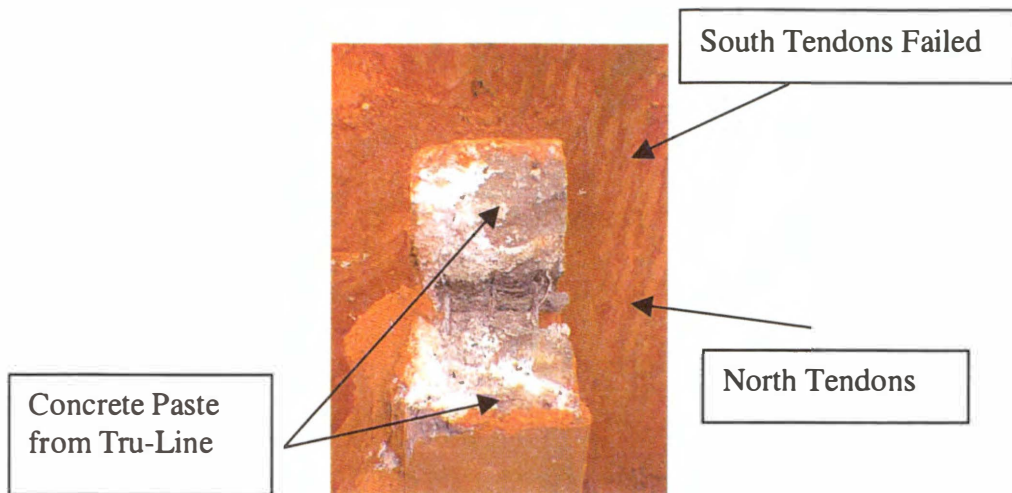


Figure A7.13: Pile 2 Failure 2

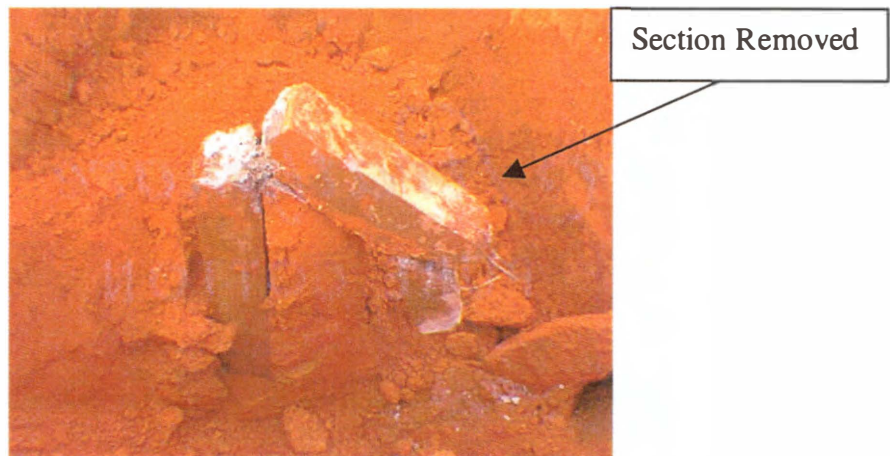
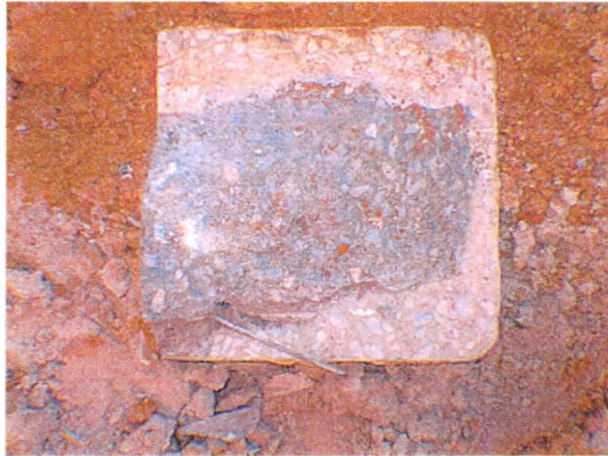


Figure A7.14: Pile 2 Failure 3



North
↓

Figure A7.15: Pile 2 Final Cut

VITA

Alan William Eichelman was born at The University of Tennessee, Knoxville hospital on July 8, 1978. He was raised in Morristown, TN. He graduated from Morristown Hamblen High School West in 1996. He began his undergraduate studies at Cornell University before transferring to The University of Tennessee in the spring of 1998 and receiving his Bachelor of Science in Civil Engineering with a minor in General Business in May of 2001. He began his graduate studies in the summer of 2001 and received his Masters of Science in Civil Engineering in the summer of 2002.

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