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Static Lateral Load Testing of Model Piles in Clay Soil Phase 1

Steven Douglas Dapp
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STATIC LATERAL LOAD TESTING OF MODEL PILES IN CLAY SOIL

PHASE I

by

Steven Douglas Dapp

A thesis submitted in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE

in

Civil and Environmental Engineering

Approved.

UTAH STATE UNIVERSITY
Logan, Utah
2000
ABSTRACT

Static Lateral Load Testing of Model Piles in Clay Soil
Phase I

by

Steven Douglas Dapp, Master of Science
Utah State University, 2000

Major Professor: Joseph Caliendo, Ph.D.
Department: Civil and Environmental Engineering

This research project was done on behalf of the Utah Department of Transportation (UDOT). Model piles were subjected to static lateral loading in homogeneous, undisturbed clay with a known undrained shear strength. The dimensions of length, diameter, height from soil to applied load, and a pile stiffness parameter as was determined by dimensional analysis to be consistent will common full-scale steel pipe piles commonly used by UDOT. Bending moment profiles of the model pile were obtained for Lateral loads using foil type strain gages. Pile head deflection and soil response (p-y curves) were determined from these measured pile moment profiles.

Model pile test results were compared to predictions made by the computer design packages Florida Pier (a 3-D, nonlinear, finite element analysis program written at the University of Florida) and COM624P.
ACKNOWLEDGMENTS

First and foremost, I would like to dedicate this work to the memory of my father, Harry Douglas Dapp (April 20, 1925 to July 28, 1998). He was the greatest teacher of life's most important lessons that a son could ever hope to have. I would also like my wife, Pei-Chia Dapp, to know just how important her love and support are to all that I endeavor.

I would like to thank all the teachers throughout my life who have patiently shaped my mind and affected me in such profound ways. In particular, I would like to thank Dr. Caliendo for being such a great friend, as well as instructor. I would also like to thank other outstanding mentors at Utah State University that I had the privilege of learning from: Dr. Watkins, Dr. Anderson, Dr. Sampaco, Dr. Haycock, Dr. Batty, and Dr. Folkman. These gentlemen are truly shining examples of the best that our engineering profession, and indeed humanity, has to offer.

Steven Douglas Dapp
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CHAPTER 1

INTRODUCTION

1.1 Background

Highway bridges are often supported on deep foundations (i.e., driven piles and/or drilled shafts). These foundations must be designed to resist both axial and lateral forces. The lateral forces often control the design of the foundation system. There are several sophisticated computer models available for predicting the response of piles and pile groups subjected to lateral loading. One such program being funded and endorsed by the Federal Highway Administration (FHWA) for state highway departments use is Florida Pier, a nonlinear, finite element analysis program written at the University of Florida. This is a result of increased concern over pile foundations being subjected to extreme lateral events such as earthquakes, ship impact, and scour. Florida Pier can model pile groups (including battered piles) and is capable of modeling a wide variety of loads and moments applied to the pile cap and/or superstructure.

It is essential that predicted computer results be validated by comparison with measured results. Because of the very high expense, limited full-scale lateral load test data is available, particularly for pile groups. Lateral response of pile groups remains a very high priority not only for the Utah Department of Transportation (UDOT), but also for the FHWA and the Transportation Research Board (TRB).
The state of Utah is experiencing significant growth and a corresponding increase in its infrastructure is needed, particularly along the Wasatch Front's I-15 corridor. When Florida Pier is validated for characteristic Utah soils, it can be used with confidence by the UDOT in the design of deep foundations.

1.2 Work Plan

The UDOT Technical Advisory committee and USU had discussed and decided that the best way for approaching the study was to divide it into three distinct phases. These phases follow in chronological order and are as follows:

1.2.1 Phase I (this study presented)

Develop and implement a model load test facility, and model pile. Perform lateral load tests on a single model pile, data reduction, and comparisons to Florida Pier and COM624P.

Phase I progress has been periodically documented during its development as listed chronologically to follow:

1) Northwest Geotechnical Workshop, FHWA / UDOT Big Fork Montana
   August 1994
2) U.S. / Taiwan Geotechnical Engineering Collaboration Workshop, Taiwan R.O.C., January 1995
3) Interim Report to UDOT, February 1995
4) Engineering Geology and Geotechnical Engineering Symposium, USU, March 1995
5) Load Test Demonstration for UDOT, September 1995
6) Presentation to TRB committee A2K03, Washington D.C., January 1995
7) Presentation to TRB committee A2K03, Washington D.C., January 1996
8) Presentation to AASHTO Bridge Meeting, Portland Oregon, 1996

1.2.2 Phase II

Develop equipment and testing procedures for model pile groups and cyclic loading.

1.2.3 Phase III

Verify model pile group response through full scale load testing of pile groups.

1.3 Objectives

The objectives of Phase I (this study) are:

1) Test facility: configuration and design, consisting of a consolidation/test cell, and lateral loading mechanism.
2) Consolidation of the test soil.
3) Model pile and data acquisition system: design, construction, calibration.
4) Load testing and comparisons: perform lateral load tests, data reduction, and comparisons to Florida Pier and COM624P.
LATERAL LOADS INDUCED ON PILES ARE MOST COMMONLY INDUCED FROM WAVES, WATER CURRENTS, SCOUR, TRACTION FORCES, SHIP IMPACTS, AND SEISMIC OCCURRENCES. ANALYSIS OF SINGLE PILE LATERAL LOADING IS COMPLICATED BY THE INTERACTION OF THE STRUCTURE AND SOIL. THREE COMMON APPROACHES HAVE TRADITIONALLY BEEN USED IN THE ANALYSIS OF THIS COUPLED SYSTEM:

1) The unit load transfer model, or Winkler approach, represents the soil reaction as discrete, uncoupled, nonlinear springs (Matlock and Ripperger, 1957; McClelland and Focht, 1958). This p-y subgrade reaction approach (soil load vs. pile deflection) is the most common approach as it is simple to use and easily modified for nonlinear soil response and cyclic loading.

2) Modeling the system as an ideal beam on an ideal elastic continuum (Poulos and Davis, 1980) is often employed. This model is most frequently used for problems involving small soil strains.

3) Modeling the soil using a finite element approach is becoming more frequently used in computer programs such as Florida Pier (Hoit et al., 1996).

Criteria for p-y curves have been developed for common categories of soil: sand (e.g., Reese, Cox, and Koop, 1974), soft clay (Matlock, 1970), stiff clay above water table (Reese and Welch, 1975), and stiff clay below the water table (Reese, Cox, and Koop,
Uncertainties for using these common p-y criteria for cohesive soils were analyzed statistically (Gazoiglu and O'Neill, 1984) and determined that the criterion with the greatest uncertainty is that for submerged, stiff clay. The principal source of degradation in soil response during cyclic loading was a permanent gap between the pile and soil caused by plastic yield of the soil, and intensified by hydraulic scour (Dunnavant and O'Neill, 1989).

The p-y criteria fore mentioned for common categories of soil were derived from a limited number of well instrumented field tests that reflect a unique set of conditions. The p-y curves have been shown not to be unique for a given soil. This nonuniqueness is due to the factors, other than the soil properties, including: influence of the structural stiffness of the pile (EI), the pile head condition (fixed vs. free), and a change in soil response immediately above or below (Norris, Ashour, and Pilling, 1995). Further, there exists large uncertainty in the soil properties themselves, as well as discrepancies among results from small-scale models and the field tests (Ruiz, 1986).

In situ soil response tests have been used to determine the p-y characteristics of a clay formation, these are the pressure meter test (LaVielle and Hughes, 1993), and dilatometer tests (Gabr, Lunne, and Powell, 1994). Laboratory test most suited to evaluating the soil response is undrained triaxial extension testing (Mayne, Kulhawy, and Trautmann, 1994). The undrained shear strength has most often been evaluated by vane shear tests or unconfined compression tests. However, many reports have emphasized the need to model the appropriate test mode and type for assessing the undrained shear strength value (e.g., Wroth, 1984; Ladd, 1991; Kulhawy, 1992).
2.2 Model Pile Studies

2.2.1 Mayne, Kulhawy, and Trautmann (1995) laboratory modeling of laterally loaded drilled shafts in clay

Three hundred and ninety-three full-scale load tests were compiled from a previous electric utility studies (Davidson et al., 1982; Kulhawy et al., 1983; Bragg and DiGioia, 1989) in order to evaluate typical depth-to-diameter ratios (D/B) for this study for. Of the 393 shafts surveyed, 18 had been dimensioned for unusual soil conditions or special foundation requirements, and were thus disregarded because they did not represent typical dimensions. The mean shaft diameter was 2.51 ft with a modal value of 2 ft, while the mean shaft length was 11.1 ft with a modal value of 10 ft. This yields a mean depth-to-diameter of 5 with a modal value of 4. Thus the focus of this study was established as short piles acting as rigid bodies.

The majority of full-scale load tests have been performed on large D/B ratios behaving as long flexible piles (e.g., Reese and Welch, 1975; Dunnivant and O'Neill 1989). Only a few have tested short shafts with a D/B < 12 (e.g., Ismael and Klym, 1978; Bhushan, Haley, and Fong 1979; Bierschwale, Coyle, and Bartoskewitz 1981; Lu, 1981; Davidson et al., 1982).

Forty-nine laboratory model-scale test programs are disclosed, 29 of which were in sand and discussed in a companion research (Agaiby, Kulhawy, and Trautmann 1992). Of the 20 test programs conducted in clay soil, the key test parameters are summarized to follow:
1) With only a few exceptions, the pile diameters have been limited to 1.0 inch, with the trend towards larger scaled model tests due to the difficulties existing in scaling the results to full size foundations (e.g., Turner and Kulhawy, 1994).

2) Prior research used piles composed of steel, aluminum, brass, acrylic resin, and nylon which were driven, jacked, or embedded in the soil. This study used concrete piles that were cast in place into the soil simulating the effects of concrete curing and soil/concrete interface roughness.

3) Most of the previous studies had used remolded, compacted, or packed clay deposits. Only two prior attempts were made to prepare consolidated specimens (Poulos, 1973; Ko, Atkinson, and Goble, 1984).

4) The undrained shear strength had mostly been evaluated by vane shear tests or unconfined compression tests. Many reports have emphasized the need to model the appropriate test mode and type for assessing the undrained shear strength value (e.g., Wroth, 1984; Ladd, 1991; Kulhawy, 1992). The test used in this study was a triaxial extension test.

5) Only two model testing programs previous to this study had investigated repeated lateral loading in clay (Matlock, 1970; Kishida, Suzuki, and Nakai, 1985).

Additionally, this study conducted static and cyclic load testing in anisotropically preconsolidated deposit of clay in a free-head mode. The models were representative of
rigid concrete drilled shafts. Twenty-eight cylindrical shafts were tested ranging in
diameters of 2.0, 3.5, and 6.9 inches with depth to diameter ratios ranging from 3 to 8.

Kaolinitic slurry was prestressed vertically using a pneumatically controlled rigid
piston with drainage layers at top and bottom of the rigid cell to ensure one-dimensional
consolidation. Prestress levels ranged from 0.18 to 0.60 tons/ft\(^2\). Vertical displacements
and pore water pressure were monitored to determine end of primary consolidation. They
were then allowed to rebound to atmospheric conditions to obtain the desired
overconsolidation profiles before load testing.

Twenty-six medium sized (24-inch diameter by 48-inch high) and one large sized
(54-inch diameter by 64-inch high) soil deposits were prepared to test the three sizes of
model piles. The medium sized deposit required a one month turnaround time, while the
large deposit required four months.

2.2.2 Meyerhof, Sastry, and Yalcin
(1988) lateral resistance and
deflection of flexible piles

Ultimate lateral resistance and ground line lateral deflections of freestanding
single model piles and small model pile groups were investigated. A wide range of piles
stiffnesses, embedment lengths, and depth/diameter ratios were investigated as suggested
by (Meyerhof and Yalcin, 1984). The concept of effective embedment depth is used to
correlate the behavior of flexible piles to the rigid model pile studies performed
previously (Meyerhof and Ranjun, 1972; Meyerhof, Mather, and Valsangkar, 1981;
Meyerhof, Yalcin, and Mather, 1983; Meyerhof and Yalcin, 1984; Meyerhof and Sastry,
An implicit condition for these relationships is that the pile is not undergoing any structural failure. Both sand and clay soils were studied; however, this review will focus on the results for clay soils.

The model piles in this investigation consisted of steel, timber (spruce), and nylon of 12.5 mm diameter, and various embedment lengths up to 610 mm to provide depth/diameter ratios (D/B) of 8, 15, 24, and 48. Small, freestanding 2 x 2 pile groups were also tested, with a three pile diameter center-to-center spacing of 38 mm. The piles were pushed fairly rapidly into the soil, then immediately loaded to failure at a horizontal displacement rate of 0.1 mm/minute; therefore, the effect of setup time was not studied. Piles were studied in both free head and fixed head conditions.

The clay used had a medium plasticity (liquid limit = 43, and plastic limit = 21), and an average water content of 30%. The clay was packed into the test boxes, and allowed to cure for one week prior to testing. Unconfined compression testing revealed slight anisotropy; undrained shear strengths were an average of 20 kPa in a vertical direction, and 24 kPa in a horizontal direction.

It was determined that the equation for ultimate lateral resistance previously developed for rigid piles in clay could be used for flexible piles with the use of an effective embedment depth in place of actual embedment depth as used previously. The resulting equations for ultimate lateral resistance are shown below.

Defining \( \frac{D_{ef}}{D} = \frac{Q_{uf}}{Q_{ur}} \leq 1 \), it follows that:
\[ Q_{uf} = 0.4cBD_{eu}K_{c} \]  
(Meyerhof and Yalcin, 1984)

where: \[ \frac{D_{eu}}{D} = 1.5K_{rc}^{0.12} \leq 1 \]  
(Meyerhof and Yalcin, 1984)

With substitution of \( Deu \), it follows that:

\[ Q_{uf} = 0.6cBDK_{c}K_{rc}^{0.12} \]

where: \( Q_{uf} \) = ultimate lateral resistance of a flexible pile

\( c \) = undrained shear strength of the soil

\( B \) = pile diameter

\( D \) = actual pile embedment depth

\( D_{eu} \) = effective embedment depth for ultimate lateral resistance

\( K_{c} \) = coefficient of net passive earth pressure on pile for zero soil adhesion (Meyerhof and Sastry, 1985)

\[ K_{rc} = \frac{E_{p}I_{p}}{E_{s}D^{4}} \]  
relative pile stiffness (Poulos and Davis, 1980)

where: \( E_{p} \) = modulus of elasticity of the pile

\( I_{p} \) = moment of inertia of the pile

\( E_{s} \) = average horizontal soil modulus
Similarly, expressions are obtained for pile head deflection and rotation of flexible piles can be obtained from the equations developed in previous studies for rigid piles with the use of the effective embedment depth.

Defining \( \frac{D_e}{D} = \frac{I_{yr}}{I_{yf}} = \sqrt{\frac{I_{or}}{I_{of}}} \leq 1 \), it follows that:

\[
Y_o = \frac{I_{yf}Q}{E_sD_eF_y} \quad \text{(Banerjee and Davis, 1978; Poulos and Davis, 1980)}
\]

\[
O_o = \frac{I_{or}Q}{E_sD_e^2F_o} \quad \text{(Banerjee and Davis, 1978; Poulos and Davis, 1980)}
\]

where:

\[
\frac{D_e}{D} = 2.1K_{rc}^{0.2} \leq 1
\]

\( D_e \) = effective embedment depth for deflection and rotation

\( I_{yr} \) = influence factor for deflection of rigid pile

\( I_{yf} \) = influence factor for deflection of flexible pile

\( I_{or} \) = influence factor for rotation of rigid pile

\( I_{of} \) = influence factor for rotation of flexible pile

\( F_y \) = yield displacement factor

\( F_o \) = yield rotation factor

For a free head pile condition use \( I_{yr} = 4 \), and for a fixed head pile use \( I_{yr} = 1.2 \)

(Banerjee and Davis, 1978; Poulos and Davis, 1980). These values are substituted, and
the above equations can be reduced as follows for the lateral displacement at ground line:

Free Head: \[ Y_o = \frac{2Q}{E_sD^F_jK_{rc}^{0.2}} \]

Fixed Head: \[ Y_o = \frac{1.2Q}{E_jD^F_jK_{rc}^{0.2}} \]

The maximum bending moment (\( M_u \)) in a fully embedded pile under any horizontal load \( Q \) at the surface of an elastic medium with constant soil modulus (Banerjee and Davis, 1978; Poulos and Davis, 1980) can be expressed by:

Free Head: \[ \frac{M_u}{QD} = 0.3K_{rc}^{0.2} \leq 0.15 \]

Fixed Head: \[ \frac{M_u}{QD} = 0.5K_{rc}^{0.2} \leq 0.55 \]

2.2.3 McManus and Kulhawy (1993)

preparation of large size laboratory deposits of cohesive soil

Large deposits of kaolin mixed with ground silica (plastic limit = 22, liquid limit = 11, specific gravity = 2.65, and water content = 33 %) were prepared in the laboratory for model lateral load testing. Soil deposits were prepared through reconsolidation of a slurry, the only process yielding the necessary control over stress history. Pore pressure transducers were embedded in the soil during placement, and a direct current displacement transducer (DCDT) was used to monitor settlement during consolidation. The coefficient of consolidation was determined to be \( 0.2 \times 10^{-6} \text{ m}^2/\text{s} \).
The soil deposits were of sufficient size to negate boundary effects and problems in model scale laws (e.g., Parkin, 1988; Scott, 1979). Two sizes used were 0.6 m diameter by 1.2 m deep with a turnaround time of 25 days, and 1.4 m diameter by 2.1 m deep with a turnaround time of 65 days.

The prominent reason for preparing laboratory soil samples is to remove any uncertainties including the magnitude of in-situ soil stresses, stress history, and inhomogeneity. The following list is recommended requirements for prepared laboratory deposits:

1) Uniformity - The soil must be uniform through entire deposit.
2) Repeatability - The soil must be repeatable from one deposit to another.
3) Stress History - The stress history must be known and controlled at all times.
4) Instrumentation - It must be possible to embed transducers during soil placement.
5) Dissection - It must be possible to examine soil failure and obtain undisturbed samples after testing is completed.
6) End Use - It must be possible to construct model foundations in, and perform in-situ soil tests using standard techniques.
7) Ease of Use - Must be relatively quick and easy, reasonable turnaround time.
8) Boundary Conditions - Boundary conditions must be known, and vessel size great enough to minimize boundary effects.
Rigid steel tanks, of circular cross section, were used that were split longitudinally and bolted together to allow easily removal and inspection of soil after testing was completed. The interior walls were coated with Teflon and silicone grease to minimize undesirable shear stresses along these boundaries.

The soil slurry was mixed in a drum using an impeller type mixer (300 rpm), then placed into the soil test vessel in three layers using a double diaphragm pump. The soil slurry placed would be consolidated before the next soil level was added. Pore pressure transducers were placed on top of soil layers one and two at the end of their respective consolidation process such that the next soil level added would then embed these transducers within the soil mass.

The surcharge stress of 110 kN/m² was applied to the soil by means of a rigid piston assembly. A regulated water pressure was applied to the top of this piston assembly to produce the desired surcharge stress. The soil vessel was drained at both the top and bottom to expedite the consolidation process. A miniature laboratory vane shear device was used to measure in situ undrained shear strength prior to placement of soil. The value of undrained shear strength ranged from 15.1 to 33.7 kN/m², with an average of 19.1 kN/m². These values were in good general agreement with those determined from triaxial compression testing of the soil performed after the completion of the model testing.
2.3 Full-Scale Studies

2.3.1 Dunnivant and O'Neill (1989)

experimental p-y model for submerged, stiff clay

Uncertainties for using common p-y criteria for cohesive soils were analyzed statistically (Gazoiglu and O'Neill, 1984), and it was determined that the criterion with the greatest uncertainty is that for submerged, stiff clay (Reese, Cox, and Koop, 1975). This is attributed to the expansive soil imbibing water during the course of testing, which severely degrades the soil response during cyclic loading. This paper describes a set of full-scale lateral load tests in a submerged stiff clay which is borderline type CL to CH clay.

Modifications to the p-y criteria for this soil are proposed, which investigated the effects of scale, relative pile-soil stiffness, and number of load cycles. Suggested is a slightly less stiff initial behavior, but a lower post-peak degradation of the soil response. Appreciable cyclic degradation did not begin until the pile head displacements had reached about 1% of the pile diameter, but once started did not appear to stabilize within 200 load cycles. The principal source of degradation was a permanent gap between the pile and soil caused by plastic yield of the soil, and intensified by hydraulic scour.

The borderline CL to CH clay deposits tested were moderately jointed with small discontinuous slickensides and some isolated sand seams and carbonate nodules. Shear strength profiles were developed from undrained triaxial compression tests, cone penetrometer soundings, and field vane shear tests. The shear strength generally
increased with depth, and ranged from 50 kPa to 200 kPa. The axial strain at 50% of the peak principal stress difference ($e_{50}$) needed for the $p$-$y$ models was determined with monotonic, undrained triaxial testing. The at-rest earth pressure coefficient ($K_0$) ranged from 3 at a depth of 0.6 m to 1 at a depth of 10.7 m, and a laboratory pinhole dispersion test indicated that the clay was nondispersive (Dunnavant, 1986).

Three piles were tested with varying diameter and flexural stiffnesses. Pile 1 was an open-ended, driven, steel pipe pile that plugged at a depth of 6.1 m during driving. It had a diameter of 0.273 m, a length of 11.8 m, a wall thickness of 9.27 mm, and an $E_I$ stiffness of 138 MN*m$^2$. Pile 2 was an open-ended, driven steel pipe pile that did not plug. It had a diameter of 1.22 m, a length of 11.4 m, a wall thickness of 15.9 mm, and an $E_I$ stiffness of 358,600 MN*m$^2$. Pile 3 was a bored, reinforced pile where the concrete was cast in place. It had a diameter of 1.83 m, a length of 11.4 m, and an $E_I$ stiffness of $1.98 \times 10^7$ MN*M$^2$.

The pile instrumentation consisted of strain gages in a full bridge configuration down the length of the piles, inclinometers, load cells, and linear voltage differential transducers (LVDT's). The soil instrumentation consisted of pore pressure cells, and seismic instrument casings in one case. Piles 1 and 2 were exercised ten times then calibrated prior to installation in a simple beam and cantilever beam loading configurations. Output was within 6% of expected with exception to locations that were in close proximity to the supports during calibration.

The piles were tested in a free head condition, with cycle periods ranging from 1 to 100 seconds. The first cycle of a test was considered to be the static response. Three
test series were performed on pile's 1 and 2: "primary" loading series, "healing" loading where there was a gap between the pile and soil present from the primary loading, and "sand" loading where this gap had been filled with a fine sand. Pile 3 was subjected to only primary loading with an irregular loading history over a period of more than one year.

Construction of the p-y curves was accomplished using local cubic polynomial fits of the moment data that fit 5 to 9 contiguous data points. The soil reaction per unit length of pile (p) was obtained through double differentiation, while the pile deflection (y) was obtained through double integration using the measured boundary conditions of pile head displacement and slope.

Results show that significant degradation in the soil response did not occur due to cyclic loading until the pile head deflection reached about 1% of the pile diameter. The rapid rate of degradation at larger displacements was associated with the formation of a permanent gap between the pile and soil due to plastic deformation of the soil and hydraulic scour during the cyclic testing. The initial slopes of the static and cyclic load-deflection curves were essentially identical, but the peak values were greatly reduced for cyclic loading. Significant dips in these cycling curves appeared at pile head deflections of 6 to 7 mm, believed to represent the onset of hydraulic scour. The introduction of sand into the gap did not produce significant strengthening to the soil response due to swelling of the soil during gap formation, and probable liquefaction of this sand during testing due to an upward water velocity in this gap zone.
2.3.2 Reese and Welch (1975)
lateral loading of deep foundations in stiff clay

Full-scale load tests were conducted to predict the short term to cyclic lateral load behavior of single piles in stiff clay. The authors report that previous full-scale studies have been performed on soft clays (Matlock, 1970) and sands (Reese, Cox, and Koop, 1974).

The piles were instrumented to obtain bending moment data vs. depth; and with the boundary conditions of pile head displacement and slope, the distributions of deflection, slope, shear, and soil reaction were all determined as a function of depth. The strain gages were mounted inside a 10.75-in. outside diameter pipe with a 0.25-in. wall thickness that extended the full 44-ft length of the 30-in. diameter drilled shaft. Of this total shaft length, 2-ft was above ground level while the point of load application was at ground level. An embedded Hi-pile was used to provide reaction for the hydraulic jacking force, and was located 25 ft away center to center. The pipe was first split to allow strain gage installation, then welded back together. The strain gages were wired in a four gage, full bending bridge arrangement, and were spaced at 15 inches the top two-thirds of the shaft and 30 inches the bottom third.

The interaction of the pile/soil system was then described in terms of p-y curves for the purpose of design recommendations in this soil type. The assumption is made that the soil reaction at a particular depth is independent of the soil strains above and below that level. While not strictly true, it is presented that experiments have shown this assumption reasonable for the relatively small pile deflections that occur in practice.
The site was near Houston, Texas, in near surface soil of the Pleistocene age known locally as Beaumont clay. The soil profile consisted of a stiff to very stiff red clay to a depth of 28 ft (8.5 m), then 2 ft (0.6 m) of interspersed silt and clay layers, and then a very stiff tan silty clay to a depth of 42 ft (13 m). The water table was at a depth of 18 ft (5.5 m) at the time of the test. Unconsolidated-undrained triaxial compression testing run at the in situ confining pressures were run on samples were run with the soil sample oriented vertical and horizontal, and was determined to be isotropic in strength characteristics. In the upper 20 ft (6.1 m) the average undrained shear strength was determined as 1.1 tsf (110 kN/m²), the secant modulus generally decreased with depth, and the average value of ε₅₀ was 0.005. There were significant variations in these properties due to a slickenside structure.

The shaft was subjected to repeated loadings of 10, 20, 30, 40, and 50 tons (89, 178, 267, 356, and 445 kN). Each 10-ton (89 kN) load increment was cycled a total of 20 cycles, or until a cycle produce no further head displacement, before proceeding to the next load increment. The shaft was excavated to a depth of 20 ft (6.1 m) and thoroughly cleaned after completing the load tests in order to accurately evaluate its geometry. The shaft was then laterally loaded again, without soil support the top 20 ft (6.1 m) in order to calibrate the structural stiffness.

Analysis of the bending moment showed that the depth to maximum moment and depth to counterflexure point increased with load, and the maximum moment was a non-linear function of the load. The maximum moment increased 10 to 20% over 20 load cycles, but the depth to the maximum moment increased very little. If this test had been
conducted in soil below the water table, caution would have been issued that much more severe degradation of soil response may be expected due to piping out of soil around the shaft.

The bending moment data was fit with a polynomial of degree 7 using a least squares curve fitting technique. Shaft deflection profiles were obtained by double integration of the moment profiles using the boundary conditions of the measured pile head displacement and an assumed zero displacement at the toe. Soil reaction was obtained through double differentiation of the moment profiles. The p-y curves were then generated at discrete depth, and presented as a function of depth for the purpose of the following design recommendation:

\[
\frac{p}{p_u} = 0.5 \left( \frac{y}{y_{50}} \right)^{\frac{1}{4}}
\]

where: 
- \( p \) = soil resistance
- \( p_u \) = ultimate soil resistance
- \( y \) = pile displacement
- \( y_{50} \) = pile displacement at e50

This differs from the cited recommendations for soft clay (Matlock, 1970) only by the exponent being 1/4 instead of 1/3 as for soft clay. Therefore, Matlock's equation format was adopted for the expression of the ultimate resistance of a stiff clay as the greater of the two to follow:
\[ p_u = \left( 3 + \frac{gx}{c} + 0.5 \frac{x}{b} \right) cb \]

or

\[ p_u = 9cb \]

where: \( x \) = depth

\( g \) = average effective unit weight to depth \( x \)

\( c \) = average effective unit weight to depth \( x \), (Matlock, 1970, treats this as effective unit weight at depth \( x \))

\( b \) = width of foundation
CHAPTER 3

TEST SOIL

3.1 Soil Containment and Consolidation Cell

The test cell consists of a ribbed steel tank 3 ft (0.91 m) by 10 ft (3.05 m) in plan, and 4 ft (1.22 m) deep with 10 hydraulic cylinders, on five yokes. Wood falsework was erected around the tank to aid in soil placement and overfilling, as well as freeboard to contain a static water head above the soil surface level. The inside of the cell had an impermeable liner, and a geocomposite drainage layer. Figure 1 is a diagram of the consolidation cell. The test facility is shown in Figure 2.

Figure 1. Consolidation/test cell.
3.1.1 Ribbed steel tank

The purpose of this tank is to confine the soil as consolidation, occurring under an imposed hydraulic reaction system, is taking place. The cell walls were required to be strong enough to resist the soil pressure and to provide a vertical reaction for the hydraulic cylinders to push against.

3.1.2 Wood falsework

Wood falsework, made with plywood with wood whalers, was erected around the outside of the ribbed steel tank. This wood falsework extended a minimum of 2 ft (0.61 m) above the top level of steel tank to allow for overfilling the cell with soil, while maintaining a static water head a few in. above the soil surface.
3.1.3 Impermeable liner

Two layers of polyethylene, 0.006 in. (0.15 mm) thick each, were used for the impermeable liner. The liner was placed inside the steel tank. The primary purpose of the impermeable liner was to retain the water, thus keeping the soil completely saturated at all times. If the water level had been allowed to drop below the soil level, then the test soil would become stratified in its strength parameters, and thus would not be a homogeneous sample that was hoped for and was obtained.

A secondary effect of the two layers of liner was to help reduce the amount of shear stress between the side of the cell and the soil. As the soil was undergoing consolidation, differential movement was experienced between the rigid cell and the soil. This differential movement causes shear stress. Any shear stress on the soil would contribute to the effective stress, thus producing a slight increase in effective consolidation stress near the boundaries of the cell.

3.1.4 Geocomposite drainage layer

A geocomposite drainage layer was placed between the impermeable layer and the soil to facilitate the consolidation process. Consolidation occurred rapidly because the longest drainage path that existed in the soil was 1.5 ft (0.46 m).

The drainage layer consisted of a geocomposite in contact with all six sides of the soil (includes top and bottom surfaces) with total and continuous coverage. The geocomposite had a layer of nonwoven, needle-punched geosynthetic on each of the surfaces, with a stiff plastic drainage flow net sandwiched in the middle.
The geocomposite had to be cut in order to contour to the edges of the cell. Strips of nonwoven, needle-punched geosynthetic were laid along the edges of the cell, both inside and outside of the geocomposite to provide a continuous flow conduit. The cut edges of the geocomposite had sharp edges, and the drainage net was exposed. These strips that were placed both inside and out of the geocomposite layer had the additional benefits of protecting the impermeable layers from being punctured by the geocomposite, and to keep soil from infiltrating into the geocomposite drainage net.

3.1.5 Consolidation pressure

The clay soil was consolidated inside the cell by an overburden stress provided by 10 hydraulically actuated cylinders, each with a ram diameter of 4 in. (101.6 mm) reacting against five yokes connected to the ribbed steel tank. These yokes, containing two hydraulic rams each, are loosely pinned to the steel tank. The yokes thus self-align themselves normal to load plate bearing on the top surface of drainage layer and soil. The load plate was two pieces of 3 ft (0.91 m) x 5 ft (1.52 m) x 0.75 in. (19.1 mm) thick steel plates, with two 6 x 6 in. (152.4 x 152.4 mm) wood beams lying along the length to help ensure a constant load distribution to the load plate.

With this load plate arrangement, a constant strain was maintained during the consolidation process. It could be argued that a constant stress condition may be more ideal for obtaining a homogeneous test soil with respect to the soil's strength properties. A constant stress condition could be provided by a flexible membrane sandwiched between the soil surface and a reaction plate and then pressurized either pneumatically or
hydraulically. However, this constant stress technique could present its own unique difficulties such as bursting pressure of the membrane used, and difficulties in maintaining overburden pressure. The technique used for loading the soil into the cell (discussed in Section 3.2), and the design of the cell ensured that a homogeneous sample was placed into the cell. Thus, the constant strain imposed from the rigid load plates produced a near constant stress within the soil sample.

The effective consolidation stress (an equivalent height of saturated overburden) was imparted to the soil by ten hydraulic cylinders. The relationship of the effective consolidation stress to the cylinders hydraulic pressure is as follows:

\[
\sigma_y = H(y_{at} - y_{water}) = (10 \text{cylinders}) \frac{A_{cylinder}}{A_{plan}} \times P
\]

where: 
- \( \sigma_y \) = effective consolidating stress (lb/ft\(^2\))
- \( H \) = height of simulated, saturated overburden (ft)
- \( y \) = unit weight (lb/ft\(^3\))
- \( P \) = hydraulic pressure inside cylinders (lb/in.\(^2\))
- \((10 \text{ cylinders})\) = number of hydraulic cylinders
- \( A_{cylinder} = \pi \times (2 \text{ in})^2 = \text{area of hydraulic cylinder rams (in.}^2\text{)}\)
- \( A_{soil} = (10 \text{ ft}) \times (3 \text{ ft}) = \text{soil area in plan view (ft}^2\text{)}\)

This simple relationship assumes a uniform stress distribution in plan view under the rigid load plates. It further assumes negligible side friction between the soil and sides of the cell such that the soil stress remains constant in the vertical direction with no load shedding to side friction.
The hydraulic system consisted of a pump supplying pressure to the cylinders through a four-way slide valve with the return (drainage) being dumped to the sump. The valve could be opened, and the cylinders pressurized to a desired level providing a load to the soil. A hydraulic accumulator was placed into the system in order to maintain a constant pressure to the clay soil. The accumulator allowed the load plate to "follow" the soil downward as the soil consolidated. Any desired constant hydraulic consolidation pressure may be maintained within about $\pm 3$ lb/in.$^2$ (20.7 kPa), which corresponds to a deviation in simulated overburden pressure of $\pm 12.57$ lb/ft$^2$ (0.60 kPa). Figure 3 shows the simplified hydraulic schematic.

![Figure 3. Hydraulic schematic of consolidation/test cell.](image-url)
3.2 Procurement and Placement of Soil

The clay soil was taken from a settling pond at a gravel washing operation in Hyrum, Utah. The clay soil was dredged out, in a near slurry state, from the end of a secondary settling pond with the use of an extendable backhoe, as pictured in Figure 4. Taking the clay soil from the same area of a settling pond was done to help ensure a homogeneous, uniform clay sample. Further, the clay procured in this manner was underconsolidated, with no stress history.

Figure 4. Soil dredging from settlement pond.
Approximately 9 cubic yards, or 243 ft\(^3\) (6.88 m\(^3\)), of the clay was transported to the Utah Water Research Laboratory (UWRL). The bed of the dump truck was lined with an impermeable membrane, 0.006 in. (0.15 mm) thick polyethylene, in order to retain water and keep the soil in a saturated state. The lined dump truck contained the soil overnight, until it could be unloaded and placed the next day. The liner inside the dump truck was slit open and the soil was then unloaded onto the concrete floor. The soil was then loaded into the consolidation tank in a near slurry state, with the use of a bobcat loader. See Figure 5 for a picture of this.

Figure 5. Soil loading into the consolidation/test cell.
The consolidation tank was first filled with about 2 ft (0.61 m) of water before any soil was loaded into it. A water level was maintained above the soil fill level at all times throughout the course of this project. This ensures that there will be no air entrained in the soil, and there will be no macro-structure imperfections (desiccation cracking, etc.).

### 3.3 Settlement and Shear Strength

The soil consolidation proceeded in three steps: dead load, a constant overburden pressure of 1250 lb/ft² (59.85 kPa), and finally a constant overburden pressure of 1675 lb/ft² (80.20 kPa). Settlement was measured throughout the consolidation process and will be presented in the sections to immediately follow. Measurements of settlement during dead load consolidation were made with the use of a survey level. During the consolidation process involving the use of the hydraulically actuated system, settlement measurements were made from the load plate to the reaction yoke at each of the ten ram locations. The settlement curves, presented in the section to follow, are thus an average of readings at these ten locations.

Mini vane shear tests were made throughout the consolidation process, and immediately after lateral load testing was performed. This method of obtaining undrained shear strength is the most appropriate for lateral load testing and input for analysis for clays in free-standing water in this strength range according to Matlock (1970).
3.3.1 Initial dead load consolidation

The soil slurry was first subjected to consolidation under a dead load. In this manner, some strength was attained by the soil before it was subjected to the higher hydraulically actuated consolidation loads. The geosynthetic drainage layer was placed over the top surface of the soil with the geosynthetic drainage layers from the sides of the bin coming up and overlapping the top. The two steel load plates were placed. Dead weight in the form of concrete blocks were systematically added on top of the load plate over a 15-day period.

Each time the soil settlement stabilized, another block was added. This was accomplished from 8 Sep. 94 to 23 Sep. 94, and stabilized at a final overburden pressure of 180 lb/ft² (8.62 kPa). There was a total average settlement of about 0.23 in. (5.8 mm) during this dead load consolidation. The original soil fill level was approximately 1.5 ft (0.46 m) above the top of the ribbed steel tank, and the overfilled soil and water was retained by the wood falsework/liner system.

3.3.2 Overburden consolidation pressure of 1,250 lb/ft²

A hydraulic pressure of 300 lb/in² (2,068 kPa) providing an effective overburden consolidation pressure of 1,250 lb/ft² (59.85 kPa) was continuously applied to the test soil from 14 November 94 to 2 January 1995. The settlement curve is shown in Figure 6.

The settlement curve slowed considerably, but never really flattened out as would be expected when nearing the end of primary consolidation. The soil at this time was about 18 in. (457.2 mm) above the top edge of the metal test bin with no concern or
Settlement: 14 Nov 94 to 2 Jan 95
Under 1,250 psf Consolidation Pressure

Figure 6. Settlement curve of soil under 1,250 lb/ft² consolidation pressure.

danger of desiccation as the soil and water was still being adequately contained by the wood falsework/liner system.

The 18 in. (457.2 mm) of soil above the ribbed metal test bin extended about 2 to 3 in. horizontally past the upper, inside edge (shoulder) of this metal test bin where it was then retained by the wood falsework. At this time it was suspected that this small change in cross-sectional area with depth may have been causing localized stress conditions near the top of the soil profile. Specifically, as the stress paths flowed downward past the near discontinuous cross-sectional area change there would be a localized stress increase just above the upper edge of the metal bin at the edge of the bin, and below the upper edge of the metal bin at the center of the bin.
This was verified on 23 Dec 94 when vane shear strength profiles were taken at both the edge and center locations in the test bed. Figure 7 shows these shear strength profiles. There are substantial deviations in the shear strength due to the stress concentrations discussed above.

3.3.3 Overburden consolidation

pressure of 1,675 lb/ft²

The soil on the "shoulders," as discussed above, was removed before further consolidation was initiated. This soil was removed while maintaining a small water head on the soil, and in a manner to least disturb the remaining soil sample.

Undrained Shear Strength: Ave.=452 psf

Mini-Vane Shear, Tested: 23 Dec 94

Figure 7. Mini-vane shear strength profile after consolidation under 1,250 lb/ft².
First the load plates and the top drainage layer were removed, exposing the top surface of the soil. The soil on the shoulders was then meticulously sliced away from the bulk of the soil with a small piece of sheet metal. This soil was then removed piece by piece with a small hand spade. The space left empty above the shoulders was filled with bricks between the dry side of the test bin liner and the wood falsework. The top drainage layer and load plates were then replaced.

The hydraulic system was then pressurized to 400 lb/in.$^2$ (2,758 kPa), providing an effective overburden consolidation pressure of 1,675 lb/ft$^2$ (80.20 kPa). This pressure was continuously applied to the test soil from 6 Jan 95 to 16 Feb 95. This settlement curve is shown in Figure 8. The settlement curve flattened out quickly, with 2.1 in. (53.3 mm) of total settlement in 41 days.

On 26 Jan 95 soil strength profiles were once again taken at the edge and center of the test bed, and are shown in Figure 9. The soil strength profile was reasonably consistent at an average undrained shear strength of 657 lb/ft$^2$, or 4.56 lb/in.$^2$ (31.45 kPa). Two exceptions, near the top of the front edge profile, were significantly higher. A hole first had to be drilled through geosynthetic drainage layers. When these two readings were taken, the mini-vane could be felt catching up on geosynthetic fibers partially pushed down with it, and thus these two readings may be high.

A cone penetrometer test was made on 23 Feb 95, with the assistance of Dr. Kyle Rollins from BYU. Figure 10 shows the shear strength profile obtained, with an average undrained shear strength of 595 lb/ft$^2$, or 4.13 lb/in.$^2$ (28.49 kPa). This is in good general agreement with the mini-vane shear tests shown previously in Figure 9.
Settlement: 6 Jan 95 to 16 Feb 95
Under 1,675 psf Consolidation Pressure

Figure 8. Settlement curve of soil under 1,675 lb/ft² consolidation pressure.

Undrained Shear Strength: Ave. = 657 psf
Mini-Vane Shear, Tested: 26 Jan 95

Figure 9. Mini-vane shear strength profile after consolidation under 1,675 lb/ft².
3.3.4 **Shear strength profile immediately proceeding the pile tests**

The undrained shear strength of the soil in the test cell was measured immediately after the load tests were completed. The mini-vane shear apparatus was used. Four soundings were made around the test pile outside of the zone of pile influence. Figure 11 contains these shear strength profiles and a description of the location in plan.
Shear Strength: After Pile Test
Mini-Vane Shear, Tested: 26 July 95

Figure 11. Mini-vane shear strength profile immediately proceeding pile tests.
CHAPTER 4
MODEL PILE

4.1 Similitude

Dimensional analysis for the purpose of modeling was carried out according to the principles of the Buckingham Pi theory. In this analysis, the full-scale pile commonly used by UDOT is referred to as the prototype pile, while the model for testing is referred to as the model pile.

There are seven fundamental variables (F.V.'s) of interest, and these F.V.'s contain only the two basic dimensions (B.D.'s) of force (F) and length (L), as shown in Table 1. Therefore, there are five required Pi terms (number of F.V.'s - number of B.D.'s). Since deflection (Δ) is the variable of interest, it is the nonrepeating variable, and is a function of the other four Pi terms as shown below.

\[
\left( \frac{\Delta}{D} \right) = f \left( \frac{PD^2}{EI}, \frac{L}{D}, \frac{L}{H}, \frac{CD}{EI} \right)
\]

It is sufficient to model the pile only to its point of fixity (critical length for long, counter-flexured pile behavior), a depth where the pile is essentially fixed from lateral loads. This critical length is dependent on the relative stiffness of the pile to the soil. Pile lengths deeper than this point essentially do not contribute to the lateral load response of the pile-soil system.
Table 1. Dimensional analysis variable summary

<table>
<thead>
<tr>
<th>Fundamental Variables (F.V.'s)</th>
<th>Basic Dimensions (B.D.'s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Δ (pile deflection at applied load)</td>
<td>L</td>
</tr>
<tr>
<td>P (lateral load on pile)</td>
<td>F</td>
</tr>
<tr>
<td>D (outside diameter of pile)</td>
<td>L</td>
</tr>
<tr>
<td>L (length of pile)</td>
<td>L</td>
</tr>
<tr>
<td>EI (pile stiffness)</td>
<td>F L^2</td>
</tr>
<tr>
<td>H (dist. up from ground level to load)</td>
<td>L</td>
</tr>
<tr>
<td>Su (Soil undrained shear strength)</td>
<td>F / L^2</td>
</tr>
</tbody>
</table>

This is a true model with respect to the F.V.'s of length (L), outside diameter (D), and height (H). Any one of these three F.V.'s can be selected according to any geometric constraints of the test facility. The limiting geometric constraint is that of model pile length (L). The length of the model pile was limited such that it will not extend to within three pile diameters of the bottom of the test bin in an effort to discount boundary effects. With the length established, the diameter (D) of the model pile was fixed by the Pi term (L/D). Similarly, the height (H) was fixed by the Pi term (H/L).

The cross-sectional moment of inertia (I) could then be determined from the diameter (D) and from a variety of wall thicknesses of readily available pipe. The model pile material (E) was still independent at this point. This allowed for the use of the cross-sectional moment of inertia (I), and model pile material (E) to control model pile stiffness (EI) to yield a manageable model lateral load (P). The smaller that the model pile stiffness (EI) is, the smaller (and more manageable) the model lateral load would need to
be to simulate the given prototype pile load according to the Pi term $(PD^2/EI)$.

By using a thinner wall model pile (I), and a model pile material with a lower modulus of elasticity ($E$), a smaller model load can simulate the given full-scale load. The danger of getting too low of a cross-sectional moment of inertia ($I$) and/or modulus of elasticity ($E$) is that its stability against crimping failure, and its yield stress are also reduced, respectively.

### 4.2 Model Pile Design

The model pile had geometric dimensions of length ($L$), outside diameter ($D$), height from soil to applied load ($H$), and a stiffness parameter ($EI$) as was determined by dimensional analysis to be consistent with full-scale steel pipe piles commonly used by UDOT. However, it was not essential that a strict parallelism be maintained. This was because the results of these tests will be compared to predictions made by Florida Pier and COM624P, using the model pile for input parameters. Once these programs have been investigated for characteristic Utah clay soils and piles, they can be used with greater confidence for designs in this area.

What was experimentally measured was the moment profile of the model pile for each of the lateral loads ($P$). Pile head deflection ($\Delta$) vs. lateral load ($P$), and p-y response curves were generated from the moment profiles.

The material selected for the model pile was 6061 T6 aluminum conforming to ATSM B 241. A 1-in. schedule 40, 1.315 in. (33.401 mm) outside diameter with a 0.133 in. (3.378 mm) wall thickness, was chosen from the four readily available possibilities.
according to the dimensional analysis, as shown in Figure 12 (D vs. T). To establish the fundamental variables of the model pile, the $P_i$ terms of the model pile were set equal to the $P_i$ term of the prototype pile. The fundamental variables of the prototype pile were known from what is typically used by the UDOT for steel pipe piles.

Figure 13 (D vs. L) shows that the chosen size most effectively utilized the 4-ft (1.22 m) depth of test soil available by requiring a length of embedment of 3.72 ft (1.13 m). Figure 14 (D vs. $P$) shows the lateral load required for this chosen pile was 212.7 lb (0.946 kN). The test soil can be established as the prototype soil, because the undrained shear strength ($S_u$) is not a function of the soil stress, as it would have been if the test soil had been a sand or gravel (i.e., the undrained shear strength need not be scaled).

**Figure 12.** Outside diameter vs. thickness for model pile design.
Outside Diam.(OD) vs. Embed.Length (L)

6061 T6 Aluminum Model

![Graph](image)

- **OD vs. L from Dimensional Analysis**
- **Sizes Close to Exact Model**
- **Size Selected for Model**

Figure 13. Outside diameter vs. length of embedment for model pile design.

Outside Diam. (OD) vs. Lat.Load (P)

6061 T6 Aluminum Model

![Graph](image)

- **OD vs. P from Dimensional Analysis**
- **Sizes Close to Exact Model**
- **Size Selected for Model**

Figure 14. Outside diameter vs. lateral load for model pile design.
Design of the model pile, through dimensional analysis, had set the ratio of basic dimensions of length \( (L) \) and Force \( (F) \) to follow, and a summary of prototype and model pile properties is shown in Table 2.

\[
L_{\text{prototype}} = 9.70L_{\text{model}} \quad F_{\text{prototype}} = 98.5F_{\text{model}}
\]

An analysis run on COM624P shows that the point of fixicity (critical length for a long, counterflexured pile) was under 3.33 ft (1.016 mm). The maximum stress was only about one third of the aluminum's yield stress, so a yield failure was not expected, nor did one occur. With an OD/T ratio of only 8.5, a crimping failure was not expected, nor did one occur.

### 4.3 Model Pile Manufacture

#### 4.3.1 Strain gage installation

Fourteen diametrically opposite pairs of bonded resistance type strain gages, 28 gages total, were mounted along the inside surface of the aluminum model pile without splitting the model pile. The 14 strain gage pairs were mounted at the intervals as shown in Figure 15.

The strain gages were procured from Micro Measurements, a subsidiary of the Vishay Group, and were shipped out of Raleigh, North Carolina. The type of gage used is designated as CEA-13-250UW-120. The sequence of installation steps was as follows:

1) degrease inside of tube

2) polish inside with steel wool and acid conditioner
3) flush with neutralizer and dry air
4) insert installation tool and apply strain gage with M bond AE-10 adhesive.

Table 2. Prototype and model pile properties

<table>
<thead>
<tr>
<th>Property</th>
<th>UDOT Pile (Steel)</th>
<th>Prototype Pile (Steel)</th>
<th>Model Pile (Aluminum)</th>
<th>Prot. / Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Δ pile deflection</td>
<td>non-repeating var. of interest</td>
<td>non-repeating variable of interest</td>
<td>non-repeating variable of interest</td>
<td>9.70</td>
</tr>
<tr>
<td>P lateral load</td>
<td>20,000 lb (88.90 kN)</td>
<td>25,072 lb (111.52 kN)</td>
<td>266.7 lb (1.186 kN)</td>
<td>94.01</td>
</tr>
<tr>
<td>D out. diam. of pile</td>
<td>12.75 in. (324 mm)</td>
<td>12.75 in. (324 mm)</td>
<td>1.315 in. (33.4 mm)</td>
<td>9.70</td>
</tr>
<tr>
<td>I pile embed. length</td>
<td>432.00 in. (10,973 mm)</td>
<td>431.84 in. (10,969 mm)</td>
<td>44.52 in. (1,130 mm)</td>
<td>9.70</td>
</tr>
<tr>
<td>EI pile stiffness</td>
<td>8.100<em>10^9 lb</em>in.^2 (55,850 MPa)</td>
<td>8.098<em>10^9 lb</em>in.^2 (55,836 MPa)</td>
<td>915,510 lb*in.^2 (6.31 MPa)</td>
<td>8.845</td>
</tr>
<tr>
<td>E modulus of elasticity</td>
<td>29.0*10^6 lb/in.^2 (200 MPa)</td>
<td>29.0*10^6 lb/in.^2 (200 MPa)</td>
<td>10.0*10^6 lb/in.^2 (69 MPa)</td>
<td>2.90</td>
</tr>
<tr>
<td>I x-sec. mom. of inertia</td>
<td>279.3 in.^4 (1.163 x 10^-4 m^4)</td>
<td>279.0 in.^4 (1.160x10^-4 m^4)</td>
<td>0.09155 in.^4 (3.635 x 10^-8 m^4)</td>
<td>8.853</td>
</tr>
<tr>
<td>H load height above grade</td>
<td>60.00 in. (1,524 mm)</td>
<td>58.20 in. (1,478 mm)</td>
<td>6.00 in. (152 mm)</td>
<td>9.70</td>
</tr>
<tr>
<td>Su undrained shear stren.</td>
<td>4.56 lb/in.^2 (31.45 kPa)</td>
<td>4.56 lb/in.^2 (31.45 kPa)</td>
<td>4.56 lb/in.^2 (31.45 kPa)</td>
<td>1.00</td>
</tr>
</tbody>
</table>
The inside surface of the pile was first etched with dilute phosphoric acid to remove any manufacturing residue, grease, or scale. A rubber stopper was inserted into one end of the pile and then about 3.05 in.³ (50 ml) of the dilute acid was poured into the pile. A steel wool wrapped rod was then inserted into the pile. The rod was then moved in and out of the pile several times while being spun at high speed with an electric drill. This procedure ensures that the entire interior surface had been thoroughly acid etched. The pile was rinsed with water and the operation was repeated with an ammonia water neutralizing solution. The pipe was again rinsed and then dried with air. At the conclusion of these operations, the inside of each pipe had a polished and clean finish.
When etching and neutralizing was completed, a steel scribing tool was used to scratch straight reference lines 6 in. (152.4 mm) long on the outside surface at the ends of the pipes. These reference lines would be used during the gage mounting procedure to assure that the gages were being circumferentially aligned properly within the piles.

The lead wire for the gages was a flat, two strand, unshielded type. All lead wires to gages on one side of a pile utilized red and white colored insulation, while all the lead wires to gages on the opposite side of the pile utilized black and white wire insulation. In this manner a gage is first designated as "red" or "black" depending on which side of the pile it is on, followed by a number "1 through 14" depending on which level the gage is on ("1" being closest to the pile head, with "14" being closest to the pile tip). The length of lead wire for each gage was made such that 7.0 ft (2.13 m) extended out of the pile head for connecting purposes. Both ends of the lead wires were stripped of insulation and tinned for good connection to the gages and data acquisition system.

Before installation of the gages into the pile, the lead wires had to be attached to the strain gages. A gage was positioned on a Teflon-covered mounting board where rosin core solder buttons were melted onto the two strain gage connecting tabs. Then the lead wire ends were soldered onto the gage by pressing the wire leads against the solder buttons with the tip of the soldering iron until the buttons and leads melted together. The connection points were then waterproofed with an acrylic sealant applied with a brush. The gages and lead wires were all tested with a volt meter to check for any bad connections.
The 14 strain gage pairs were then mounted without splitting the model pile. This was accomplished by means of the specialty tool designed and machined in the USU Civil Engineering machine shop, and is illustrated in Figure 16.

In preparation for mounting, a gage pair was first affixed to the mounting tool. Teflon strips covered the rubber-coated wings on the tools so that the gages would not be glued to the installation tools wings during mounting. Reference lines were drawn on these Teflon surfaces to aid in gage alignment. Short strips of cellophane tape were used to tack the gages in place -- a strip over the wires just where they met the gage, and another strip holding the tip of the gage down. As small amount of the gage tip was covered with the tape as possible so that the surface area of the gage available for bonding to the pile was maximized. The potential for unbonding, and/or peeling of the gage does exist at this location on the gage. The lead wires for both gages, designated "red" and "black," were then taped to the front of the installation tool.

Figure 16. Strain gage installation tool.
A long slender rod was slid through the pile tip, and a string attached to the rod was then drawn through the pile as the rod was withdrawn. The lead wires were pulled through the pile tip with the string until the wire emerged from the pile head. The gage wires were then pulled through the pile as the installation tool was being inserted through the pile tip.

M-bond AE-10 epoxy was prepared and smeared onto the gages ready for installation. The installation tool was then inserted into the pile by means of two telescoping rods (see Figure 17). The outer rod holds the tool at the required location, both depth inside the pile and circumferential orientation. The inner rod then engages and turns the screw threads, which draws in the wedge assembly. Spacers on this outer rod maintains the assembly centered inside the pile in order to keep the freshly coated epoxy on the gages from wiping along the inside of the pile during installation. The wedge assembly being drawn inward forces the wings apart. The two wings press the strain gages against the inside surface of the tube, and maintains this pressure while the epoxy adhesive cures. The purpose of the rubber band is simply to collapse the wings after the epoxy is cured and the wedge is subsequently withdrawn. After the epoxy had cured, excess slack was gently pulled out of lead wires and they were fixed to the inside surface of the pile head with a butyl rubber adhesive. One concern that arose during calibration and testing was that as the pile was flexed, the lead wires could possible become too taught and pull away from the gage, or perhaps cause a debond of the gage.

An epoxy was chosen for the bonding agent over cyanoacrylate (M-bond 200) for two reasons. First, the AE-10 epoxy will remain pliable for years, while the M-bond
would start to become brittle after a couple of months, making the gages essentially inoperable at any significant strain levels. Second, the AE-10 epoxy has a set time of 6 hours at normal room temperatures while the M-bond 200 tends to flash set, making the installation process even more precarious and unforgiving.

4.3.2 Pile head and lead wire shielding

Once all the gages and lead wires had been installed, the top of the pile was sealed. A latex rubber stopper was inserted into the top of the pile, with a slight compressional fit, and pushed approximately 0.75 in. (19.05 mm) past the top of the pile.
This latex rubber stopper serves only as a filler. Next, 100% silicone caulking was placed in the remaining space in the top of the pile, making sure to work it in and around the protruding lead wires for a completely air-tight seal. Silicone was selected for the sealer, because silicone is impervious to the nitrogen purging gas, as described in the section to follow.

The pile heads were machined, also from 6061 T6 aluminum, with approximately 0.03 in. (0.762 mm) compression fit relative to the outside diameter of the top of the piles. The lead wires were bundled together inside a single length of electrically insulative shrink wrap, then fished through the opening in the pile head. The pile head was then driven onto the top of the pile with a rubber mallet. Silicone caulking was then injected through the opening in the pile head until all the space between the top of the pile and the pile head was filled. The silicone was carefully worked in and around the protruding lead wires to form a completely air-tight seal. The pile head was thus environmentally sealed to the pile top.

The bundled lead wires were fished through braided stainless steel hose wrap, with the end of this hose wrap camped to the pile head with a stainless steel hose clamp. The hose wrap was then drawn out tight against the wire bundle, and a ground wire was soldered to the end of this hose wrap. The end of the hose wrap was secured near the end of the wire bundle with electrical tape.

The lead wires, while not individually electrically shielded due to lack of space inside the pile, were electrically shielded from the environment by the steel braided wrap and the aluminum pile itself. Electrical shielding is crucial when making these extremely
minute electrical measurements, because any electromagnetic transient fields from the surrounding environment, such as those produced from lighting transformers, could induce current in unshielded wires. Any electromagnetic transients will be absorbed by this shielding system, and discharged to the grounding system.

4.3.3 Dried nitrogen purge and tip installation

After all the gages and pile head were installed, the piles were purged with dry nitrogen gas through the bottom end of the pile, and then the end was sealed off with a pile tip. This was done to safeguard against moisture condensing inside the piles.

The nitrogen supply line was run through an in-line cartridge containing dryerite crystals (a common desiccant used for this purpose). As the nitrogen runs through this cartridge, the crystals will absorb any moisture out of the nitrogen. The small diameter, flexible supply line was inserted through the bottom of the pile, held in upside down position, until it was within a few inches of the pile head. As nitrogen is slightly more dense than the surrounding air, even more so when cooled by the adiabatic blow down process of releasing it from the pressurized tank, the purging process is more effective when done from the bottom upward.

The nitrogen was released into the pile in this matter at a rate sufficient to hear and feel the flow escaping out the pile for approximately 10 minutes, and then the flow rate was slowed as the supply line was slowly withdrawn from the pile over a period of approximately 5 minutes. The volume of nitrogen used during purging was many times the volume of the pile itself, insuring that any moisture had been driven out.
The pile tips, with a greased sealing O-ring, were inserted into the tip of the pile immediately proceeding the dried nitrogen purging, while maintaining the pile in an upside down position. The pile tip was machined from 6061 T6 aluminum with a groove cut around the outside diameter for a face sealing, viton rubber O-ring. The O-ring was well greased with petroleum jelly before installation. The tip profile was flat and at a right angle relative to the pile axis. This closed-ended model pile is similar to a plugged or closed-ended prototype pile, where a soil wedge develops and is driven in front of the prototype pile.
CHAPTER 5

DATA ACQUISITION and INSTRUMENTATION

5.1 Data Acquisition Hardware and Transducers

The data acquisition system consisted of a computer, a data-logger, and a multiplexer. The LVDT transducers were processed directly by the data-logger while the strain gage signals were switched through the multiplexer, and then to the data-logger. A common Wheatstone bridge, with a gage in a dummy pile for temperature compensation, lies between the multiplexer and data logger to condition the strain gage signals. Figure 18 shows a schematic of this hardware arrangement.

Figure 18. Data acquisition system.
5.1.1 Computer, data-logger, and multiplexer

The computer used for testing was an IBM compatible PC, comparable to a 386 model. A data-logger interface card was installed. The computer was used to process and download the operating instructions through the data-logger and store the digitized transducer readings uploaded back through the data-logger.

The data logger used was a Campbell Scientific 21X data-logger. This data-logger was adequate for Phase I static load testing. However, the data-logger may need to be upgraded for dynamic testing, depending largely on what multiplexer is used for dynamic testing.

The multiplexer used was a Campbell Scientific AM 416 multiplexer. This multiplexer operates with an automated, mechanical switching system. Dynamic testing undertaken in the future will most certainly require a faster, digital multiplexer.

5.1.2 Common Wheatstone bridge and dummy pile

A Wheatstone bridge arrangement is necessary to condition the resistance type, foil strain gage signals for measurement. The bridge arrangement used was a temperature compensated quarter bridge arrangement. Figure 19 is a schematic of this bridge arrangement.

The relationship between strain and the measured output voltage is expressed in the following relationship:
Figure 19. Wheatstone bridge used (quarter bridge arrangement).

\[ \varepsilon = \frac{V_{out} (R_{active} + R_{dummy})^2}{V_{source} S R_{active} R_{dummy}} \]

where:
- \( \varepsilon \) = strain (variable of interest, dimensionless)
- \( V_{source} \) = source input voltage (constant at 0.500 volts)
- \( V_{out} \) = output voltage (variable, in volts)
- \( S \) = strain gage factor (constant at 2.11, dimensionless)
- \( R_{source} \) = resistance of active gage (variable, in ohms)
- \( R_{dummy} \) = resistance of dummy gage (variable, in ohms)
As the pile was strained, the foil strain gage bonded to the pile surface was also
strained causing its resistance to change. Strain was directly proportional to these minute
amounts of change in the strain gage resistance. The data logger monitored the change in
strain gages resistance by measuring the voltage across terminals "H" and "L", while
supplying a constant voltage of 0.500 volts across the terminals "E" and "≡".

An important aspect of this bridge arrangement is that initially the bridge is
"balanced," meaning that the output voltage across terminals H and L is nearly zero
relative to ground. Thus the rage of the data logger in measuring this output voltage can
be set to utilize the full extent of this range with respect to the maximum output voltage
expected across the bridge. In this way the sensitivity of the data logger can be
maximized to our specific application. If the data logger were to simply measure the
resistance of the gage alone, without a Wheatstone bridge, the nominal resistance of 120
ohm would have been many orders of magnitude larger than the change in resistance.
The change in resistance would have been lost in the data logger output voltage range that
would have been necessary to measure the original resistance.

The precision of conventional resistors is insufficient to balance the bridge output
voltage to a near zero value. Precision resisters were used in arms "E to H" and "H to ≡"
of the Wheatstone bridge. These resisters have a nominal resistance of 120 ohm and a
tolerance of ± 0.0120 ohm (± 0.01%).

A dummy pile with a single pair of strain gages was used in the Wheatstone
bridge arrangement, denoted as \( R_{dummy} \). The purpose of the dummy pile strain gages was
to maintain temperature compensation in the bridge arrangement. The dummy pile gages
were the same type as the model pile gages. These gages were installed inside the dummy pile approximately centered in elevation with respect to the soil mass around the pile during testing. The dummy pile was manufactured with the exact same processes that were used to manufacture the model piles.

As temperature changes may occur in the test pile during testing, thermally induced strains would result in the model pile. This would cause a change in the voltage of point "H" in the bridge relative to ground. This voltage change at point "H" would also change exactly the same amount relative to point "L" of the bridge in this case, since point "L" remains fixed relative to ground by the two fixed, 120-ohm precision resistors. This would be incorrectly interpreted as strains resulting from the loading of the model pile.

The dummy pile is thus needed for compensating of temperature induced strains in the quarter bridge arrangement. The dummy pile is subjected to the same thermal environment as the model pile. Thus, any thermally induced strain (and subsequent change in gage resistance value) of the model pile is going to be matched by an equal thermally induced value in the dummy pile. Thus, point "H" will experience no change in voltage relative to ground (or also to point "L" in this case) due to the thermally induced strains. In this manner the bridge is temperature compensated, and any change in voltage across points "H" and "L" is due to only load induced strains.

5.1.3 Strain gage selection

The strain gages selected were Micro Measurement resistance-type foil gages, and
had a gage factor of 2.11. The selection of the strain gages was based on the following criteria, which correspond to Micro Measurements strain gage designation of CEA-13-250UW-120:

1) "CE" specifies a flexible gage with cast polymide backing and encapsulation featuring large copper coated solder tabs providing optimum capability and durability for direct lead wire attachment.

2) "A" specifies a constantan alloy in self-temperature compensated form.

3) "13" specifies the self temperature compensation number and is the approximate thermal expansion coefficient of the aluminum, from which the model pile is made, in ppm/°F. This gage is specifically designed to be self temperature compensating with regards to differential thermal expansion between the gage and the aluminum piles. That is to say that the gage will experience the same temperature induced contractions or expansions. This is important, because any differences in the thermal contraction or expansion between the gage and the aluminum pile would incorrectly be interpreted as very significant amounts of strain changes in the model pile. This is a different phenomenon than the temperature compensating characteristics of the Wheatstone bridge as discussed in the previous section.

4) "250" specifies the active gage length in mils, 0.001 in. (0.0254 mm).

5) "UW" specifies a normal grid and tab geometry.

6) "120" specifies the nominal resistance in ohms of the strain gage.
An excitation time of 0.01 seconds per gage was used with a delay of 0.01 seconds between each gage excitation. This cycle was repeated three times in order to obtain an average of these three readings for each time increment.

5.1.4 LVDT displacement transducers

Two linear variable differential transducers (LVDT) were procured from RDP Electrosense Inc., of Pottstown, Pennsylvania. One LVDT transducer was used in the first series of tests, and a second LVDT transducer was added for the second series of tests. The first transducer was a model no. LDC 3000C, serial no. 760 with 6 in. (152.4 mm) of total travel inside the linear measurement range, a sensitivity of 0.76 volt/in. (0.0299 volt/mm), and a linearity of 0.12%. The second transducer was a model no. LDC 1000C, serial no. 1461 with 2 in. (50.8 mm) of total travel inside the linear range, a sensitivity of 2.24 volt/in. (0.0882 volt/mm), and a linearity of 0.26%.

It was planned for these transducers to provide boundary conditions of pile head deflection and slope. The pile head slope would provide a constant of integration when integrating the measured pile moment distribution (obtained through the strain gage readings) to obtain the pile slope distribution. The pile head deflection would then provide a constant of integration when integrating the pile slope distribution to obtain the pile deflection distribution. Pile load distribution is obtained by double differentiation of the moment distribution. The p-y curves are obtained when this pile deflection distribution is plotted against the pile load distribution at discrete levels of the pile below soil grade. This would provide the two constants of integration when backing out p-y
curves from the test data, as discussed in Section 8.3.1, as well as directly providing the pile head deflection versus lateral load characteristics.

Subsequent analysis of the strain gage data makes assumptions of the boundary conditions, in lieu of using faulty LVDT readings. The assumptions that were made are that the pile displacement and slope at the tip are zero. These appear to be a valid assumptions, particularly for the lower load levels, as discussed in Section 8.3.

5.2 Data Acquisition Software

The data logger controls the data acquisition system by providing the following: switching instructions to the multiplexer, precision controlled input voltages to the transducers, precise readings measurements of the output voltages from the transducers, and digitizing and uploading the information to the computer. The set of instructions for the data logger to carry out these processes is first written on the computer, then downloaded to the data logger. The software used was Campbell Scientific's PC208 data logger support software, version 6306-08 (1988). The software is specifically designed and written to be used with Campbell Scientific's equipment.

5.3 Electrical Grounding

Grounding for the data acquisition system was provided by the model pile itself. The data logger ground was connected to the multiplexer ground through the shielding wrap around the common wires between these two instruments. The ground from the multiplexer was then connected to the aluminum pile through the shielding wrap around
the strain gage wires, as discussed in Section 4.3.2. The pile was in contact with the test soil when performing the load tests, and to the large steel supports when performing the calibrations of the model piles. The shielding ensured that any electromagnetic transients that may have been present were discharged to ground. An electrically stable reference ground was essential when making these minute voltage measurements.
CHAPTER 6
MODEL PILE CALIBRATION

6.1 Overview

The raw data consists of the ratio of output voltage to input voltage for each strain gage. An initial strain gage reading was established under the no-load condition. This is subtracted from subsequent readings and the corresponding strain is calculated using the previous equation.

When possible the strains and corresponding stresses were calculated from the averaged values of the strain gage pairs. The single gage Wheatstone bridge circuitry provided redundancy in that separate readings are made at each gage location. During the calibration process it became apparent that several gages were not functioning. In these instances, only a single gage was used to determine strain at that particular pile location.

6.2 Methodology

The model piles were loaded as simply supported beams, and the output moments (as determined from the strain gage readings) were compared to calculated moments obtained theoretically from elastic beam flexure equations. A linear regression was performed to obtain a calibration factor for each individual strain gage that would modify the measured moments to match the calculated moments. Table 3 shows the load increments used for each of the load configurations, as will be discussed in the section to
follow. Note that readings were also taken as the load was decreased in the exact same number of load steps and load increments.

Table 3. Calibration load increments

<table>
<thead>
<tr>
<th>Load Increment #</th>
<th>S.S. Beam Centered Load (lb)</th>
<th>S.S. Beam Symmetric Loads (lb/each side)</th>
<th>Axial Compression (lb total)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Change</td>
<td>Total</td>
<td>Change</td>
</tr>
<tr>
<td>1</td>
<td>12.4513</td>
<td>12.4513</td>
<td>12.4513</td>
</tr>
<tr>
<td>2</td>
<td>10.8906</td>
<td>23.3419</td>
<td>10.8906</td>
</tr>
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(1) Axial Compression included for reference only, data not used in calibration
Three readings were made at each load increment, both as the load was increased and as the load was decreased. The three readings were taken at each load increment to verify that there was no localized creep occurring in the loaded pile due to yielding, and/or electronic drift in the data acquisition system. In all cases the multiple readings at a given load were at the same value, taking into account the resolution of the data acquisition system.

At the end of a load cycle, the pile was rotated 180 degrees and incrementally loaded and unloaded again. In this manner each gage could be loaded in tension and compression during both of the load configurations for calibration purposes.

There were 612 calibration data points obtained for each gage. This takes into account two load configurations, two load cycles for each configuration, 51 load increments per load cycle, and three readings taken per load increment.

Axial compression tests were also performed. The axial data provides confirmation that the calibration data from the simply supported beam configurations were correct. However, the compression test data points were not used in the linear regression to obtain the calibration constants, as these data points would have provided a bias towards the lower end of the calibration curve, and represented no data in tension. The single model pile lateral load test produced only moment magnitudes; therefore, calibration comprised only moment-induced strains.

6.3 Load Configurations and Stress Range

Two simply supported beam configurations were used in calibration as shown in
Figure 20: first, as a simply supported beam with a concentrated load in the middle of the pile, which produced a moment distribution that peaked at this center location; second, as a simply supported beam with two symmetric concentrated loads, each 14.625 in. (371.48 mm) out along the pile from the two supports, which produced a constant moment distribution between these two locations. A picture of a pile undergoing calibration is shown in Figure 21.

The pile was cycled twice in each of the two beam configurations. The load cycle was run with the red gages along the top (compression) side of the pile. The pile was rotated $180^\circ$ and the load cycle run again with the black gages along the top side of the pile. In this manner, each gage was subjected to tension and compression stresses during both load cases ranged between 0 and 253 lbs. The associated stresses were approximately 70% of the yield stresses for both cases.

Two Loading Configurations for Calibration

- **A** - Concentrated Loads @ Center
- **B** - Two Concentrated Loads equidistant from supports

Both load cases ranged between 0 and 253 lbs. The associated stresses were approximately 70% of the yield stresses for both cases.

Figure 20. Calibration loading configurations.
the calibration. The pile was also loaded axially in compression; however, this data was not used in the calibration. Exactly the same data acquisition system and software was used during the calibration loading and the lateral load testing.

The range of stress imposed at each strain gage level at the gage location on the inside surface of the model pile during the calibration loading, axial compression loading, and lateral load testing (as will be discussed in Chapter 7) are summarized below in Table 4. The maximum stress at the outside surface of the pile was then calculated and shown in Figure 22 in terms of percentage of the 40,000 lb/in.² yield stress of the aluminum.

The calibration loads were chosen such that the resulting stresses during calibration remained less than the yield stress of the aluminum. The calibration stresses are only slightly less than the load testing stresses for the first three gage levels. The test
stresses rise above the calibration stresses to a maximum difference of 20% at gage level 5. The calibration stresses exceed the load test stresses for gage levels 8 through 14.

The significance of this was that the measured stresses during the load testing were determined using extrapolation of the calibration curves in regions of the pile where the calibration stresses are less than the load test stresses. This is of little concern here for

Table 4. Maximum stress of the pile during calibration and load testing

<table>
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<tr>
<th>Strain Gage Level</th>
<th>S.S. Beam Centered Load (lb/in.²)</th>
<th>S.S. Beam Sym. Loads (lb/in.²)</th>
<th>Axial Comp. (lb/in.²)</th>
<th>Lateral Load Testing (lb/in.²)</th>
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two reasons. These maximum load test stresses are above the calibration stresses for only the last two load increments, and the extreme linearity of the calibration curves (as will be discussed in the section to follow).

6.4 Linear Regression Analysis for Calibration Factors

The measured stress was plotted against the calculated stress. These plots are made for each individual gage, and are the basis for the calibration factors. A regression analysis was performed using the measured stress as the dependent variable, and the calculated stress as the independent. A calibration factor (regression coefficient) was then
determined, which modified the measured stresses to match the calculated stresses. Appendix A contains all the calibration curves, and their respective regression analysis. A typical calibration curve is shown in Figure 23. Table 5 summarizes all calibration factors, while Table 6 summarizes the least squares fit from the linear regressions.

6.5 Faulty Gages and Discrepancies

Three gages were clearly malfunctioning due to damage during the gage installation and pile fabrication processes. These gages were Red 14 of Pile #1, Black 2 and Red 3 of Pile #2. A typical calibration curve of a faulty gage is shown in Figure 24. The sporadic nature of calibration data from these gages conclusively shows that these gages did not adhere well, or completely, to the inside surface of the model pile, or are in some other way damaged.

![Figure 23. Typical calibration curve of a functional gage.](image-url)
Table 5. Calibration factors from linear regression

| Strain Gage Level | Pile #1 | | | Pile #2 | | | Pile #3 | | |
|------------------|---------|------------------|------------------|------------------|------------------|------------------|
|                  | Red Gage | Black Gage | Red Gage | Black Gage | Red Gage | Black Gage |
| 1                | 1.102521 | 1.046606 | 1.237991 | 1.668774 | 1.180641 | 1.178496 |
| 2                | 1.080720 | 1.276563 | 1.593702 | defective | 0.778442 | 0.757919 |
| 3                | 1.609657 | 1.056098 | defective | 5.092250 | 1.213571 | 1.178935 |
| 4                | 1.868258 | 1.070517 | 2.525873 | 1.617731 | 3.521735 | 2.590897 |
| 5                | 1.208909 | 1.179533 | 1.841858 | 2.160248 | 1.207909 | 1.334116 |
| 6                | 1.700617 | 1.606956 | 1.764209 | 1.082606 | 1.181468 | 1.052159 |
| 7                | 1.115865 | 2.998602 | 1.380346 | 1.094614 | 2.277032 | 1.619471 |
| 8                | 1.522933 | 2.230372 | 1.089462 | 4.640968 | 1.387754 | 1.068382 |
| 9                | 5.512844 | 4.876580 | 1.056193 | 1.098781 | 1.097796 | defective |
| 10               | 1.985588 | 4.738567 | 3.340756 | 1.066895 | defective | 1.078507 |
| 11               | 2.666807 | 1.974350 | 1.027480 | 1.039251 | 1.102342 | 2.383938 |
| 12               | 1.097181 | 4.254809 | 1.209876 | 1.078021 | 1.176358 | 1.115227 |
| 13               | 2.672136 | 1.231854 | 1.063086 | 0.972797 | defective | 7.118745 |
| 14               | defective | 2.335818 | 2.496892 | 1.059323 | 1.018469 | 1.022141 |

In instances where one gage of a gage pair level was faulty, the moment was still measured using the one functional gage since the gages were measured independently in quarter bridges. Instead of using the average strain of a gage pair in calculating the moment, only strain of the one functioning gage was used. The assumption that must be made was that the opposite gage has a strain level equal to but opposite in sign of the functioning gage. This influence of this assumption was mitigated by the fact that this
Table 6. Calibration standard error of coefficients from least squares fit

<table>
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<tr>
<th>Strain Gage Level</th>
<th>Pile # 1 Red Gage</th>
<th>Strain Gage Level</th>
<th>Pile # 1 Black Gage</th>
<th>Pile # 2 Red Gage</th>
<th>Pile # 2 Black Gage</th>
<th>Pile # 3 Red Gage</th>
<th>Pile # 3 Black Gage</th>
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The same assumption was also made in the performance of the linear regression to obtain the calibration constants. However, data at these locations were suspect and were considered slightly less reliable due to this lack of redundancy.

The measured stresses were compared to the calculated response of the pile. There are various possible explanations as to why the measured stresses do not exactly match the calculated stresses. These possibilities include, but may not be limited to the following:
During analysis of the load test data from the first series of tests performed using Test Pile #2, two levels of strain gages appeared to be nonfunctional. These nonfunctional levels were 2 and 10. Their values did not agree with the trend of the other strain gage levels, and were not used in the moment curve fitting analysis as will be discussed in Sections 8.2 and 8.3. Level 2 was suspect even before testing occurred due to Black 2 of the gage pair being completely nonfunctional. The reason for level 10 being nonfunctional remains unknown.
7.1 Procedure

7.1.1 Configuration and loading

Lateral load tests on the model pile were performed in general accordance with ASTM D 3966-81. This standard designates the applied lateral load in terms of the design load. Table 7 shows the lateral loads applied, and the time duration for each. The maximum model pile lateral load was 266.7 lb (1.186 kN), which is equivalent to a full scale prototype pile lateral load of 25,072 lb (111.52 kN). The model pile lateral load was applied 6 in. (152.4 mm) up from the soil surface, which corresponds to a distance of 58.2 in. (1,374 mm) for the full-scale prototype pile.

The load mechanism was a simple metal-framed arm with a sheave on a bearing center to minimize any frictional drag. This metal arm was clamped rigidly to the exterior end of the ribbed metal test bin. This arm could be adjusted in all three translational and rotational axes, such that proper alignment of the lateral load was made. Although trivial for static testing, energy losses for dynamic testing may be significant.

Measurements were made at the beginning, middle, and end of each load increment. The data acquisition system took a reading at every transducer three times for every measurement. This redundancy provided back-up readings in case any one or two of the three data sets were somehow in error. Of additional benefit, an average of these readings was made in an effort to reduce the effects of any possible random noise.
Table 7. Lateral loads applied and duration during model testing

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<th>Load Increment #</th>
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<td>4</td>
<td>20</td>
<td>+32.7</td>
<td>146.9</td>
</tr>
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<td>+32.6</td>
<td>179.5</td>
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<td>+43.6</td>
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<td>10</td>
<td>-65.3</td>
<td>125.1</td>
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<td>11</td>
<td>10</td>
<td>-61.7</td>
<td>63.4</td>
</tr>
<tr>
<td>12</td>
<td>NA</td>
<td>-63.4</td>
<td>0</td>
</tr>
</tbody>
</table>

7.1.2 Model pile installation

The model piles were pushed by hand into the test soil in a static manner. During the entire installation process templates were used to ensure that the pile was plumb, and a ball bearing was used to transfer the downward thrust to the pile to ensure that no torque was applied to the pile head. Figure 25 shows a picture of the pile installation. A change in pile orientation during installation could create a small gap between the pile and soil that would allow displacement to occur before the soil resistance was mobilized.
The templates were two rigid, hard plastic plates. These plates were each comprised of two pieces, each with a hemi-circle cut out. Four inset screws held the two pieces together such that the two hemi-circles would mate to form a hole through the template. The diameter of this hole was made the same size as the outside diameter of the piles. Thus, once the pile was installed, the plate was split into its two components and removed from around the pile.

The pile was installed in a radial orientation such that the plane containing the strain gage pairs extended out towards the smaller 3-ft (0.91 m) wide test bin sides. In this orientation, the maximum amount of soil would be present on the active and passive
sides of the test pile. Once installed, minor adjustments were made to the load mechanism position to insure that the lateral loading axis would lie in the same plane containing the strain gage pairs, and perpendicular to the unloaded pile. Any misalignment would cause a lower strain reading than what would be actually induced. This error would be small, however, because it would be proportional to the sine of any small angle in error.

7.2 Test Series

The test series consisted of three cycles of loading and unloading the model pile. The direction of loading was reversed each time a new cycle of loading began. The three load cycles were conducted on 25 and 26 July 1995 are labeled Test #1, Test #2, and Test #3 chronologically. Test #1 was a zero load cycle test, Test #2 has the direction of lateral loading reversed and may be considered 1/2 of a load cycle, and Test #3 was again loaded in the same direction as Test #1 and was one complete load cycle.
CHAPTER 8

RESULTS

8.1 Florida Pier and COM624P Computer Models

Predictions of pile behavior under the lateral test loads were made using Florida Pier and COM624P. Both software packages were used to predict the behavior of the pile during Test #1 (0 load cycle), and Test #3 (1 complete load cycle). In all cases, the soil profile was modeled using Matlock's (1970) soft clay criteria below the water table, which is the most appropriate and common soil model used for this test soil type. Table 8 below summarizes the soil parameters used, while Table 2 (shown previously) summarizes the pile properties.

Table 8. Input soil parameters used for Florida Pier and COM624P computer modeling

<table>
<thead>
<tr>
<th>Property</th>
<th>Florida Pier</th>
<th>COM624P</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Matlock (1970) soft clay below water table</td>
<td>Matlock (1970) soft clay below water table</td>
</tr>
<tr>
<td>Eff. Unit Weight (lb/in.³)</td>
<td>$6.9 \times 10^{-2}$</td>
<td>$6.9 \times 10^{-2}$</td>
</tr>
<tr>
<td>Su (lb/in.²)</td>
<td>4.562</td>
<td>4.562</td>
</tr>
<tr>
<td>E50</td>
<td>$2.0 \times 10^{-2}$</td>
<td>$2.0 \times 10^{-2}$</td>
</tr>
<tr>
<td>Shear Mod. (lb/in.²)</td>
<td>122</td>
<td>NA</td>
</tr>
<tr>
<td>Poisson's Ratio</td>
<td>0.5</td>
<td>NA</td>
</tr>
<tr>
<td>Piez. Elev. (in.)</td>
<td>2.0</td>
<td>2.0</td>
</tr>
<tr>
<td>Phi (degrees)</td>
<td>NA</td>
<td>0.0</td>
</tr>
</tbody>
</table>
Since the same soil model was used for both software packages on a single pile, Florida Pier results were nearly identical to COM624P results, as would be expected. Presentation of the results of these two software packages is made concurrently with the presentation of the model pile test results in the sections to follow.

8.2 Measured Data, Moment Profiles

The moments were measured at each strain gage pair location for which a calibration constant could be obtained. The pile head displacement readings made by the LVDTs were in error and assumptions were made in the analysis section to provide boundary conditions. Appendix B contains all moment profiles (moment vs. depth), moment paths (moment vs. lateral load), and tabular data for all three pile tests at the beginning, midpoint, and end of the load increments. Note that during subsequent curve fitting and analysis, strain gage levels 2 and 10 were found to be defective, and data from these levels were not used for analysis.

The moment distributions along the pile length at the end of each time interval for Test #1 are shown in Figure 26. The readings taken at the end of the time increment are the most significant, for at this point the soil creep has essentially come to a stop. The legend on this figure indicates the applied lateral load for each of the curves shown. Increasing loads are shown as UP and the unloading is indicated by DOWN. The moment curves indicate a logical smooth moment distribution along the pile length. The results also indicate a point of fixity advancing down the pile as the load is increased. The lateral loads up to those in the mid-range cause long pile behavior; the pile undergoes
Figure 26. Test #1 moment distribution, end of time interval.

A moment reversal (counter-flexured). The lateral loads approaching the maximum value causes short pile behavior; the pile is no longer fixed at its base.

The moment versus lateral loading relationships, that occurred during both the loading and unloading of the pile, are shown for strain gage levels 1, 5, 9, and 12 for Test #1 in Figures 27 through 30, respectively. In all these figures, soil creep during the time interval of any particular load can be seen. Furthermore, this creep can be seen to slow drastically during the time interval.

Figure 27 demonstrates the completely elastic behavior of the aluminum pile, as this level is above the soil at +2.6875 in. (+68.263 mm), and the stresses are well within the elastic limit of the aluminum pile. Figure 28 shows a distinct difference in moment
Moment vs. Lateral Load
Test #1: Level #1, Depth = +2.6875 in.

Figure 27. Test #1, gage level 1 moment path.

Moment vs. Lateral Load
Test #1: Level #5, Depth = -14.5625 in.

Figure 28. Test #1, gage level 5 moment path.
Figure 29. Test #1, gage level 9 moment path.

Figure 30. Test #1, gage level 12 moment path.
during the loading and unloading segments of the pile test; however, as this level is still relatively close to the soil surface at -14.5625 in. (-369.989 mm), the soil allows the pile in this section to return to a nearly stress free state once the load cycle is completed. Figure 29 shows how the deeper sections of the pile at -29.5625 in. (-750.956 mm) are held by the soil in a state of residual stress due to the plastic yield of the soil. Figure 30, from the deepest sections of the pile at -40.8125 in. (-1,036.638 mm), shows the point of fixicity advancing down the pile as the load is increased.

8.3 Curve Fitting and Analysis

In order to analyze the pile test data, a mathematical curve fit had to be made of the obtained moment profiles. Once this was accomplished the moment curves were double integrated (using assumed boundary conditions) to determine the pile displacement profiles, and double differentiated to obtain the soil loads on the pile. The p-y curves were then determined at various depths by comparing the soil load to the pile displacement during the load cycle at these various depths.

Mathcad V6.0 was used to accomplish the curve fitting and analysis. Appendix C contains the curve fitting and analysis for Tests 1, 2, and 3, as well as the program that was written to accomplish this.

8.3.1 Pile slope and pile displacement profiles

The moment curves were integrated in order to determine the pile slope profiles; the pile slope profiles were then integrated to determine the pile displacement profiles.
Boundary conditions of Zero pile slope, and Zero pile displacements were assumed at the pile toe in every case. These integrations are governed by the following equations:

\[ M = EI \frac{ds}{dx} = EI \frac{d^2 y}{dx^2} \quad \text{and} \quad s = \frac{dy}{dx} \]

where:
- \( M \) = moment
- \( s \) = pile slope
- \( y \) = pile displacement
- \( x \) = depth (coordinates along Pile)
- \( E \) = modulus of elasticity of the pile
- \( I \) = cross-sectional moment of inertia of the pile

The curve fitting technique utilized on the moment profiles for the purpose of integration was a simple cubic spline. Figure 3 illustrates this technique for the lateral load of 266.7 lb (1.186 kN) during Test #1. Minor perturbations in the slope of the spline function as it is forced through all of the data points have a negligible effect on the integral value (i.e., the area under the moment profile); of more importance for this technique is that the curve does pass through all of the data points. Similarly, any error introduced by the assumption of zero boundary conditions at the pile toe is negligible compared to the total integral value. The integrations were performed numerically using a step size of 0.01 in. (0.254 mm) over the entire length of the model pile.

8.3.2 Pile shear profiles and soil load

The moment curves were differentiated in order to determine the pile shear profiles; the pile shear profiles were then differentiated to determine the net soil reaction per unit length of pile (passive earth pressures minus the active earth pressures). These differentials are governed by the following equations:
Figure 31. Cubic spline of moment data, and integrations to yield pile slope and pile displacement.
\[ V = \frac{dM}{dx} \quad \text{and} \quad p = \frac{dV}{dx} = \frac{d^2 M}{dx^2} \]

where:
- \( M \) = pile moment
- \( V \) = pile shear
- \( p \) = net soil reaction
- \( x \) = depth (coordinates along Pile)

Various curve fitting techniques were tried on the moment profiles for the purpose of differentiation. The techniques tried were cubic spline functions ranging from first order to third order continuity at the interface of the spline function, high order polynomial functions of the entire moment profile, and floating third and fourth order polynomial fits for neighborhoods ranging from five to seven data points. The curve fitting technique that proved most conducive to double differentiation was a floating cubic fit, with a neighborhood of five data points. Figure 32 illustrates this curve fitting technique, while Figure 33 illustrates the differentiations of this moment data for the lateral load of 266.7 lb during test #1.

Minor perturbations in the slope, and change in slope of the moment curves would compound the error as the moment curve was differentiated twice and pieced together to form soil loads. The curve fits with higher orders and complexity introduced more perturbations in slope and change in slope as these curve fits were forced closer to the data points. While the lower order and simpler floating cubic fits did not as closely model the moment data points themselves (as was desirable with the integration techniques), they did substantially quiet down the erratic changes in slope. Further, the fit of these cubic polynomials at the extremes of the neighborhoods is inconsequential, as only the midpoints of these neighborhoods were evaluated, then pieced together.
Test #1: Load 8 = 266.6574 lb

Moment vs. Depth (Floating Curve Fit)

Figure 32. Floating cubic fit of moment data.
Figure 33. Differentiations of moment data to yield pile shear and soil load.
8.4 Comparison of Test Results to Florida Pier and COM624P

Comparisons were made of the test results to the computer predictions made with both Florida Pier and COM624P. These comparisons were broken up into the categories of pile moment profiles, pile deflection profile and pile head displacement, soil load profiles, and p-y curves. Appendix D contains the complete set of these comparisons.

8.4.1 Pile moment profiles

The moment profiles for Test #1 (0 load cycle) are a close match to Florida Pier and COM624P in every case, with the experimentally determined moments developing a little less quickly with depth than was predicted with both computer simulations. The moment profiles for Test #3 (1 load cycle) have significant differences from the predicted responses.

When the maximum moment is compared to that which was predicted for each load case, as is shown in Figure 34, again it is evident that Test #1 is a very close match to the predicted values, while Test #3 has significant differences from that which was predicted. The computer simulations underpredict the maximum moment until a lateral load of approximately 195 lb (0.867 kN) is reached, at which point the computer simulations overpredict the maximum moment.

Figure 35 shows an approximate depth at which the maximum moments are developed, compared to where they were predicted to occur. Again, Test #1 closely matches what was predicted, with the test data tending to have the maximum moment slightly deeper than what was predicted. Test #3 maximum moment depths are constant,
while the predicted maximum moment depths start shallower than the actual, then drop to deeper depths at the higher lateral loads.

**Figure 34.** Maximum moment comparisons.

**Figure 35.** Depth of maximum moment comparisons.
8.4.2 Pile deflection profile and pile head displacement

The pile head displacements are shown in Figure 36, and follow the same trends as the maximum moments comparisons that were made in the previous section. Slightly more pile head displacement occurred in Test #1 (0 load cycles) than was predicted; however, the trend was the same. For Test #3 (1 load cycle), the computer simulations underpredicted the pile head displacement until a lateral load of approximately 230 lb (1.023 kN) was reached. At this point the computer simulation overpredicted the pile head displacement by increasingly large amounts.

Pile Head Displacement Comparison

![Graph showing pile head displacement comparison](image)

Figure 36: Pile head displacement comparisons.
8.4.3 Soil load profiles

For Test #1 (0 load cycles), the actual soil loads trended in profile as was predicted; the maximum net soil resistances peaked at approximately the same depth. However, these soil resistances did not mobilize as quickly as predicted and the actual soil resistance values are generally slightly more than 2/3 of the predicted values. For Test #3 (1 load cycle), the soil resistance developed at deeper sections of the pile at the beginning of the load cycle. At a load of approximately 180 lb (0.801 kN), the predicted and actual soil resistances were at equal depths, and at increasingly greater lateral loads the predicted soil loads developed at increasingly deeper sections of the pile.

8.4.4 p-y curves

Comparisons of p-y curves are made for the first five strain gage levels below the soil surface, as deeper sections of the pile did not experience enough deflection to produce meaningful p-y curves. Both Florida Pier and COM624P computer models used Matlock's p-y curves for soft clay below the water table. Although substantial differences existed between Test #1 (0 load cycles) and that which was predicted, the general trends are similar during this load cycle. However, much greater differences occurred in the comparison of Test #3 (1 load cycle) to that which was predicted. Figures 37 and 38 show the predicted and observed p-y curves at depths of -7.06 in. and -10.81 in., respectively.

The actual p-y curves obtained for Test #1 (0 load cycles) had less of a slope (i.e., a lower soil lateral modulus) than Matlock's model at the lower displacements of the
P-Y Curves
Depth = -7.06 in.

Figure 37. p-y curves at depth = -7.06 in.

P-Y Curves
Depth = -10.81 in.

Figure 38. p-y curves at depth = -10.81 in.
curve until the ultimate soil resistance (according to Matlock's model) was reached. After this point Matlock's model has a zero slope maintained at this ultimate value, where the actual soil resistance continues to increase gradually towards this ultimate value predicted. The soil resistance for any given load below the predicted ultimate value was approximately 60% of that which was predicted. This gap closed to approximately 67% of the predicted soil load at the end of the p-y curve.

Test #2 (1/2 of a load cycle) is included in the p-y curves for comparison, and was produced when the lateral load direction was reversed and cycled. Test #2 trended close to that which was obtained in Test #1; the soil loads in Test #2 are only slightly less than those of Test #1. There exists a much greater difference between Test #2 and Test #3 in both magnitude and trend.

The p-y curves obtained for Test #3 had an extremely small slope (particularly at the beginning of the curve) compared to the predicted curve. The soil resistance continued to increase in value in a nearly linear nature, even after the predicted curve had begun to decrease after reaching its ultimate value. Test #3 results are much lower than Matlock's model, and clearly do not trend the same.
CHAPTER 9

SUMMARY AND CONCLUSIONS

9.1 Overview

Phase I of this research accomplished all the objectives, as stated in Section 1.3. The test facility, and methodology have proven to be a viable, economic way of conducting model tests of laterally loaded piles in clay. The discussions in the sections to follow summarize these objectives, how they were met, conclusions that can be drawn from them, and recommendations for future testing.

9.2 Test Facility

The test facility proved to be of sufficient scale to effectively model lateral loading of pile foundations. The depth of the test cell is sufficient to model the prototype piles under study with a prototype to model geometric ratio of 9.70. The test cell is sufficiently large in plan view to negate any boundary effects, and to conduct multiple single pile tests or study of pile groups on a limited basis. The test cell is small enough that it could be transported to other locations via a tractor trailer. There are shake tables in existence and in operation that could accommodate the entire test cell, if research should proceed in that direction.

9.3 Consolidation of the Test Soil

The main focus of the laboratory consolidation process was to provide a test soil
that was homogeneous, with a common stress history, and no inclusions or macro-fabric structure. The facility, with modifications made during Phase I, was relatively successful in accomplishing this.

An important component to the hydraulic system was added at the onset of the project. This was a hydraulic accumulator that allowed for a nearly constant overburden pressure to be maintained while allowing the load plate to "follow" the consolidating soil downward. The hydraulic system is capable of maintaining overburden pressures well in excess of any pressure potentially needed for soil studies.

9.4 Model Pile and Data Acquisition System

The model pile performed admirably, and proved itself to be an effective instrument. This was brought about by constant vigilance to detail during pile manufacture, calibration, and testing. The pile is an environmentally sealed system, as well as being electrically shielded. The pile components are made of air-craft grade structural aluminum, and the strain gages are mounted internally. As a result, the model pile is a very durable instrument. Even so, careful consideration of the effect of the extreme amount of stress that the pile undergoes during calibration and testing may need to be more thoroughly investigated.

The data acquisition system was adequate for one pile undergoing static loading, but will need to be totally upgraded when dynamic testing is pursued. This is especially true of the multiplexer, which is an old mechanical switching system that is wholly incapable of the speed and number of channels that will be required for dynamic testing.
The LVDT transducers were installed into the data acquisition system incorrectly, and did not produce directly measured pile head displacement data. However, assumptions of the boundary conditions of zero pile slope and displacement at the pile toe were used with the introduction of very little error, especially at the lower and mid-range load levels. The pile head displacements were calculated by means of double integration of the moment profiles, using these assumed boundary conditions.

One aspect of the data acquisition hardware that may warrant investigation is the dummy pile that is providing temperature compensation for the system. This dummy pile contains only one set of strain gages to provide temperature compensation for all 14 sets of strain gages in the test pile. This means that the dummy pile strain gages are excited electrically 14 times more frequently that the test pile gages. For static testing with slow cycles rates this is of little concern; however, the much higher sampling rates necessary in dynamic testing may cause the output of the dummy pile to drift relative to the test pile. It may become necessary to use a one-for-one dummy gage to test pile gage in order to negate this effect.

9.5 Load Testing Results and Comparisons to Florida Pier and COM624P

The measurements of the strain gages during testing directly yielded the moment profile of the model pile. The moment profiles showed a smooth, logical profile as was expected. The curve fitting and analysis of these moment profiles, using Mathcad V6.0, proved to be challenging, but successful in the end.
The double integrations of the moment profile to produce the deflection profiles were easily accomplished using a cubic spline technique. The integrations were insensitive to minor perturbations in the fitted spline of the moment data, and to the assumed boundary conditions at the pile toe. This insensitivity is attributed to the very small amount of area change under these curves due to minor perturbations in the fitted spline, or errors in the assumed boundary conditions.

The double differentiation of the moment profile to produce the soil load profile on the pile, however, proved to be extremely sensitive to any minor perturbations in the curve fit to the moment profile. For this reason, it was found that a floating cubic fit, using a neighborhood of five data points, worked best. This lower order floating fit was able to quiet down this extreme sensitivity to slope, while still effectively determining the slope and change in slope of the moment profile at the midpoint of each neighborhood.

Florida Pier results were nearly identical to COM624P results since the same soil model was used for both software packages on a single pile. The soil model used in these computer simulations was Matlock's (1970) for soft clay below the water table. This soil model proved to be marginal for Test #1 (0 load cycles), and inadequate for Test #3 (one load cycle).

For Test #1 the actual soil resistance did not develop as quickly as predicted, nor did the soil resistance reach the same ultimate value as did the model (only about 60% of what was predicted). However, the trend was generally the same as the soil model. This means that the actual pile head displacement was slightly higher than predicted, but still in good general agreement.
For Test #3, the soil model was unrepresentative of what was measured, both in trend and magnitude. The actual soil resistance developed dismally slow compared to the predicted values. Further, the actual soil resistance continued to rise slowly, in a nearly linear fashion, where the model predicts that the soil resistance should decrease once a portion of the original ultimate strength is reached. As a result, the actual pile head displacements were on the order of two to three times greater than was predicted until near the end of the load cycle. Near the end of the load cycle, the predicted pile head displacements became greater than the measured displacements by increasingly larger amounts. All these effects are easily attributed to the soil gapping that occurred between the soil and the pile, as was apparent near the soil surface. This gap is produced through a combination of soil plastic yield, and possible hydraulic scour during the first load cycle. The need for a cyclic loading soil model that takes this soil gapping phenomenon into account has been recognized in the literature, and will need to be formulated for these computer simulations to produce accurate predictions for cyclic loading.

9.6 Recommendations for Future Studies

The liner's ability to help relieve the side shear stress should perhaps be enhanced with the use of a lubricant (such as Teflon or silicone grease) between liner layers, or more layers of the liner. Careful consideration of any lubricant used would have to be given to determine if contamination of the soil by the lubricant were to occur, would there be an effect on the soil structure.
Since the drainage layer has a much higher friction to soil than the impermeable liner layer, it may be worthwhile in future consolidation efforts to limit the full surface coverage of drainage layers to the top and bottom surfaces. This may help to alleviate any possible detrimental side shear, as discussed in Section 3.1.3. This would come at the cost of consolidation time, but the longest drainage path would still be only approximately 2.5 ft (0.762 m), now defined by a horizontal plane midway between the upper and lower surfaces and extending out to the boundary of the impermeable liner. Of course, small flow conduits would have to extend from the bottom drainage layer to the top surface to relieve the water pressure at the bottom of the cell.

The full-size cone penetrometer was too large compared to the scale of the test soil bed. A mini-cone penetrometer would be better scaled to test the soil without risking as much disturbance to the test soil. A better means of estimating the lateral modulus is needed. The full-size pressuremeter planned for use was abandoned due to risk of extreme disturbance to the test soil. Triaxial laboratory testing may be considered for this, or for in situ testing a dilatometer may be more effective.

Future pile construction should give careful consideration to the amount of tension put into the lead wires, and the means by which they are attached to the inside surface of the pile. Also the use of a metal epoxy to secure the pile head to the pile, instead of this compression fit, may be beneficial. The driving of the pile head onto the pile could conceivably damage the gages. Inspection, maintenance, and repair of the mounted gages is virtually impossible due to the confined space inside the pile. Thus there is no need to have a removable pile tip. Future pile construction should also employ
a metal epoxy to permanently seal the tip to the pile. The metal epoxy used would need
to be electrically conductive, as an intimate and continuous electrical contact between the
pile head and pile must be maintained for the purpose of electrical shielding.

The same set of dummy pile gages is used for all 14 pairs of model pile gages. For
every reading made on the model pile gages, 14 readings were made on the dummy pile
gages. With 14 times more energy being run through the dummy pile gages, there is the
chance that differential resistance changes may have been induced between the dummy
pile gages and the model pile gages. This risk of increased energy absorption by the
dummy pile gages would become more significant as the frequency of gage excitation and
measurement is increased, such as may be the case with future dynamic testing.

However, for Phase I static testing, these differential changes were considered to be
insignificant. Furthermore, the excellent heat conductivity of the aluminum piles (which
the gages are adhered to), and a relatively long break period between reading cycles
ensure quick dissipation of any heat generated by gage excitation into the large soil mass.

Future testing will be considerably more complex in configuration, and will most
likely make use of actual displacement measurements. The reasons for the LVDT
malfunctions will most likely need to be further investigated and corrected, or other
methods or instruments used.

Load cells to measure the lateral load being applied to the pile will need to be
integrated into instrumentation. For this static testing, the lateral loading is determined by
carefully weighing the dead weights that are suspended, and recording which ones are
used during each reading. However, when dynamic testing is performed in the future,
load cells will be necessitated by the nature of this testing. Load cells are readily commercially available, or one could be designed and manufactured at the USU facilities.

Pressure transducers to measure the in situ pore water and/or soil pressures during testing may be developed. This may be of particular interest when evaluating the dynamic soil properties during the dynamic testing to be conducted in the future. Also, the parameters of pile, soil, and/or atmospheric temperature may become of interest in future testing. Transducers for these parameters are also readily available.

The system to contain the initial overfill of soil, wood falsework, was adequate, but sometimes cumbersome to work with, in, and around. Further the wood falsework allowed for a change in cross-sectional area from the region of overfilled soil above the test cell to the area below the top level of the test cell. This was thought to cause differential consolidation in the soil mass at first attempts at consolidation, and was later corrected by trimming this soil above the shoulders of the test cell away, and replacing this emptied volume with bricks behind the impermeable liner. Also, the hydraulic yoke extensions to allow for extra height of the hydraulic rams above the overfill soil level, while allowing great versatility and self-alignment, could be designed to be more user friendly.

All six sides of the test soil mass were in contact with the geocomposite drainage layer. While this was effective in expediting the soil consolidation process, it may be advantageous to have only the top and bottom surfaces in contact with the drainage layer. Of course a water pressure escape tube would be needed to relieve the water from the bottom surface. This simple, one-dimensional consolidation arrangement may be
advantageous over the three-dimensional arrangement used for various reasons:

1) Soil strata at any given depth would experience the same pore pressure during consolidation, rather than the soil closest to the side of the cell being relieved of pore pressure first.

2) Less shear stress would exist between the soil and the sides of the test cell, which provides a more homogeneous soil during the consolidation process. Perhaps more impermeable liner layers, or lubricant between liner layers would reduce this shear stress even further.

3) The measured consolidation rate could be compared to predicted rates calculated in a straightforward manner. Pressure transducers could be embedded within the soil mass to directly measure pore pressure dissipation for comparison to predicted rates as consolidation is occurring.
LITERATURE CITED


Appendix A. Model Pile Calibration Curves
Figure A1. Calibration of pile # 1.
Pile # 1 - Red Gage # 2
Cal.Fact.=1.08072  Error=0.000307

Pile # 1 - Black Gage # 2
Cal.Fact.=1.276563  Error=0.000676
Pile # 1 - Red Gage # 3
Cal. Fact. = 1.609657  Error = 0.000947

Pile # 1 - Black Gage # 3
Cal. Fact. = 1.056098  Error = 0.001184
Pile # 1 - Red Gage # 4
Cal. Fact. = 1.868258  Error = 0.001007

Calculated Stress (psi)

Pile # 1 - Black Gage # 4
Cal. Fact. = 1.070517  Error = 0.000692

Calculated Stress (psi)
Pile # 1 : Red Gage # 5
Cal. Fact. = 1.208909   Error = 0.000261

Pile # 1 - Black Gage # 5
Cal. Fact. = 1.179533   Error = 0.000678
Pile #1 - Red Gage #6
Cal. Fact. = 1.700617  Error = 0.00862

Pile #1 - Black Gage #6
Cal. Fact. = 1.506956  Error = 0.000198
Pile #1 - Red Gage #7
Cal. Fact. = 1.115865  Error = 0.00336

Pile #1 - Black Gage #7
Cal. Fact. = 2.998602  Error = 0.000313

Gage Suspected as Defective
Pile #1 - Red Gage #8
Cal. Fact. = 1.522933  Error = 0.000741

Pile #1 - Black Gage #8
Cal. Fact. = 2.230372  Error = 0.001906
Pile #1 - Red Gage #9
Cal. Fact. = 5.512844  Error = 0.001684

Pile #1 - Black Gage #9
Cal. Fact. = 4.87685  Error = 0.000255
Pile #1 - Red Gage #10
Cal. Fact. = 1.985588  Error = 0.000942

Pile #1 - Black Gage #10
Cal. Fact. = 4.738567  Error = 0.00395

- Gage Suspected as Defective
Pile #1: Red Gage #11
Cal. Fact. = 2.666807  Error = 0.009893

Pile #1: Black Gage #11
Cal. Fact. = 1.97435  Error = 0.000306

<< Diagrams of stress calculations for Red and Black Gages #11 show a close alignment with the measured stress, indicating minimal error. Red Gage is suspected as defective. >>
Pile #1 - Red Gage #12
Cal. Fact. = 1.097181  Error = 0.000306

Pile #1 - Black Gage #12
Cal. Fact. = 4.254809  Error = 0.000300

Gage Suspected as Defective
Pile #1: Red Gage #13
Cal. Fact. = 2.672136   Error = 0.001649

Pile #1 - Black Gage #13
Cal. Fact. = 1.231854   Error = 0.000619

Gage Suspected as Defective
Pile #1 - Red Gage #14
Defective Gage

Pile #1 - Black Gage #14
Cal.Fact.=2.335818  Error=0.003504

- Gage Suspected as Defective
Figure A2. Calibration of pile #2.
Pile #2 - Red Gage # 2
Cal.Fact.=1.237991   Error=0.001436

Pile # 2 - Black Gage # 2
Defective Gage
Pile #2 - Red Gage #3
Defective Gage

Pile #2 - Black Gage #3
Cal.Fact.=5.09225  Error=0.001216

Gage Suspected as Defective
Pile #2 - Red Gage #4
Cal. Fact. = 2.525873   Error = 0.001262

Pile #2 - Black Gage #4
Cal. Fact. = 1.617731   Error = 0.000956
Pile # 2 - Red Gage # 5
Cal.Fact.=1.841858   Error=0.000880

Pile # 2 - Black Gage # 5
Cal.Fact.=2.160248   Error=0.001104
Pile # 2 - Red Gage # 6
Cal. Fact. = 1.764209    Error = 0.000967

Pile # 2 - Black Gage # 6
Cal. Fact. = 1.082606    Error = 0.000559
Pile # 2 - Red Gage # 7
Cal.Fact.=1.380346   Error=0.000717

Pile # 2 - Black Gage # 7
Cal.Fact.=1.094614   Error=0.000548
Pile # 2 - Red Gage # 8
Cal. Fact. = 1.089462  Error = 0.000651

Pile # 2 - Black Gage # 8
Cal. Fact. = 4.640968  Error = 0.002017

Gage Suspected as Defective
Pile # 2 - Red Gage # 10
Cal. Fact. = 3.340756  Error = 0.006028

Pile # 2 - Black Gage # 10
Cal. Fact. = 1.066895  Error = 0.005026

* Gage Suspected as Defective
Pile #2 - Red Gage #11
Cal. Fact. = 1.02748  Error = 0.000272

Pile #2 - Black Gage #11
Cal. Fact. = 1.03925  Error = 0.000370
Pile # 2 - Red Gage # 12
Cal. Fact. = 1.209876   Error = 0.000490

Pile # 2 - Black Gage # 12
Cal. Fact. = 1.039251   Error = 0.000552
Pile # 2 - Red Gage # 13
Cal.Fact.=1.063086   Error=0.000645

Pile # 2 - Black Gage # 13
Cal.Fact.=0.972797   Error=0.000930
Figure A3. Calibration of pile #3.
Pile #3 - Red Gage #1
Cal. Fact. = 1.180641  Error = 0.001722

Pile #3 - Black Gage #1
Cal. Fact. = 1.178496  Error = 0.001086
Pile # 3 - Red Gage # 2
Cal. Fact. = 0.778442   Error = 0.000353

Pile # 3 - Black Gage # 2
Cal. Fact. = 0.757919   Error = 0.000188
Pile # 3 - Red Gage # 3
Cal.Fact. = 1.213571  Error = 0.000548

Pile # 3 - Black Gage # 3
Cal.Fact. = 1.178935  Error = 0.000383
Pile #3 - Red Gage #4
Cal. Fact. = 3.521735   Error = 0.001573

Pile #3 - Black Gage #4
Cal. Fact. = 2.590892   Error = 0.003941
Pile # 3 - Red Gage # 6
Cal. Fact. = 1.181468  Error = 0.000245

Pile # 3 - Black Gage # 6
Cal. Fact. = 1.052159  Error = 0.000316
Pile #3 - Red Gage #7
Cal. Fact. = 2.277032   Error = 0.000402

Pile #3 - Black Gage #7
Cal. Fact. = 1.619471   Error = 0.001132
Pile #3 - Red Gage #8
Cal. Fact. = 1.387754  Error = 0.000418

Pile #3 - Black Gage #8
Cal. Fact. = 1.068382  Error = 0.000334
Pile # 3 - Red Gage # 9
Cal. Fact. = 1.097796  Error = 0.000269

Pile # 3 - Black Gage # 9
Cal. Fact. = 12.43201  Error = 0.004074

○ Gage Suspected as Defective
Pile # 3 - Red # 10
Cal. Fact. = 28.11559  Error = 0.00285

Pile # 3 - Black Gage # 10
Cal. Fact. = 1.078507  Error = 0.000376

Gage Suspected as Defective
Pile # 3 - Red Gage # 11
Cal. Fact. = 1.102342  Error = 0.000315

Pile # 3 - Black Gage # 11
Cal. Fact. = 2.383938  Error = 0.000387
Pile #3 - Red Gage #12
Cal Fact.=1.176358  Error=0.000599

Pile #3: Black Gage #12
Cal Fact.=4.254809  Error=0.0003

Gage Suspected as Defective
Pile #3 - Red Gage #13
Cal. Fact. = 21.66431  Error = 0.004999

Pile #3: Black Gage #13
Cal. Fact. = 7.118745  Error = 0.001698

● Gage Suspected as Defective
Pile #3 - Red Gage #14
Cal. Fact. = 1.108469  Error = 0.003812

Pile #3: Black Gage #14
Cal. Fact. = 1.022141  Error = 0.003439

- Gage Suspected as Defective
Appendix B. Test Data
Figure B1. Test #1 moment profiles.
Moment Profile: Pile Test #1
Midpoint of Time Interval

Depth (in) vs. Moment (in\*kips)

- 41.6 lb UP
- 74.3 lb UP
- 114.2 lb UP
- 146.9 lb UP
- 179.5 lb UP
- 223.1 lb UP
- 255.8 lb UP
- 266.7 lb UP
- 190.4 lb DOWN
- 125.1 lb DOWN
- 63.4 lb DOWN
Moment Profile: Pile Test #1

Beginning of Time Interval

Moment Profile:

- 41.6 lb UP
- 74.3 lb UP
- 114.2 lb UP
- 146.9 lb UP
- 179.5 lb UP
- 223.1 lb UP
- 255.8 lb UP
- 266.7 lb UP
- 190.4 lb DOWN
- 125.1 lb DOWN
- 63.4 lb DOWN
- 0 lb DOWN
Figure B2. Test #1 moment vs. lateral load.
Moment vs. Lateral Load
Test #1: Level #3, Depth= -7.0626 in

Moment vs. Lateral Load
Test #1: Level #4, Depth= -10.8125 in
Moment vs. Lateral Load
Test #1: Level #5, Depth = -14.5625 in.

Moment vs. Lateral Load
Test #1: Level #6, Depth = -18.3125 in.
Moment vs. Lateral Load
Test #1: Level #7, Depth = -22.0625 in

Moment vs. Lateral Load
Test #1: Level #8, Depth = -25.8125 in
Moment vs Lateral Load
Test #1: Level #9, Depth= -29.5625 in

Moment vs. Lateral Load
Test #1: Level #10, Depth= -33.3125 in
Moment vs. Lateral Load
Test #1: Level #11, Depth = -37.0625 in

Moment vs. Lateral Load
Test #1: Level #12, Depth = -40.8125 in
Moment vs. Lateral Load
Test #1: Level #13, Depth= -44.5625 in

Moment vs. Lateral Load
Test #1: Level #14, Depth= -48.3125 in
Moment Profile: Pile Test #2
End of Time Interval

Figure B3. Test #2 moment profiles.
**Moment Profile: Pile Test #2**

**Midpoint of Time Interval**

![Graph showing moment profile with various force values at different depths.](image_url)
Moment Profile: Pile Test #2

Beginning of Time Interval

Moment Profile:

-41.6 lb UP
-74.3 lb UP
-114.2 lb UP
-146.9 lb UP
-179.5 lb UP
-223.1 lb UP
-255.8 lb UP
-266.7 lb UP
-190.4 lb DOWN
-125.1 lb DOWN
-63.4 lb DOWN
-0 lb DOWN
Moment vs. Lateral Load
Test #2: Level #1, Depth= +2.6875

Moment vs. Lateral Load
Test #2: Level #2, Depth= -3.3125 in

Figure B4. Test #2 moment vs. lateral load.
Moment vs. Lateral Load

Test #2: Level #3, Depth = -7.0626 in

Moment vs. Lateral Load

Test #2: Level #4, Depth = -10.8125 in
Moment vs. Lateral Load
Test #2: Level #5, Depth= -14.5625 in

Moment vs. Lateral Load
Test #2: Level #6, Depth= -18.3125 in
Moment vs. Lateral Load
Test #2: Level #7, Depth= -22.0625 in

Moment vs. Lateral Load
Test #2: Level #8, Depth= -25.8125 in
Moment vs Lateral Load
Test #2: Level #9, Depth = -29.5625 in

Moment vs Lateral Load
Test #2: Level #10, Depth = -33.3125 in
Moment vs. Lateral Load
Test #2: Level #11, Depth = -37.0625 in

Moment vs. Lateral Load
Test #2: Level #12, Depth = -40.8125 in
Moment vs. Lateral Load
Test #2: Level #13, Depth = -44.5625 in

Moment vs. Lateral Load
Test #2: Level #14, Depth = -48.3125 in
Figure B5. Test #3 moment profiles.
Moment Profile: Pile Test # 3

Midpoint of Time Interval

Depth (m)

Moment (in\(\times\)kips)

- 41.6 lb UP
- 74.3 lb UP
- 114.2 lb UP

- 146.9 lb UP
- 179.5 lb UP
- 223.1 lb UP

- 255.8 lb UP
- 266.7 lb UP
- 190.4 lb DOWN

125.1 lb DOWN
- 63.4 lb DOWN
Moment Profile: Pile Test # 3
Beginning of Time Interval

- 41.6 lb UP
- 74.3 lb UP
- 114.2 lb UP

- 146.9 lb UP
- 179.5 lb UP
- 223.1 lb UP

- 255.8 lb UP
- 266.7 lb UP
- 190.4 lb DOWN

- 125.1 lb DOWN
- 63.4 lb DOWN
- 0 lb DOWN
Figure B6. Test #3 moment vs. lateral load.
Moment vs. Lateral Load
Test #3: Level #3, Depth = -7.0626 in

Moment vs. Lateral Load
Test #3: Level #4, Depth = -10.8125 in
Moment vs. Lateral Load
Test #3: Level #5, Depth = -14.5625 in

Moment vs. Lateral Load
Test #3: Level #5, Depth = -18.3125 in
Moment vs. Lateral Load
Test #3: Level #7, Depth= -22.0625 in

Moment vs. Lateral Load
Test #3: Level #8, Depth= -25.8125 in
Moment vs Lateral Load
Test #3: Level #9, Depth= -29.5625 in

Moment vs. Lateral Load
Test #3: Level #10, Depth= -33.3125 in
Moment vs. Lateral Load
Test #3: Level #11, Depth = -37.0625 in

Moment vs. Lateral Load
Test #3: Level #12, Depth = -40.8125 in
Moment vs. Lateral Load
Test #3: Level #13, Depth = -44.5625 in

Moment vs. Lateral Load
Test #3: Level #14, Depth = -48.3125 in
Appendix C. Curve Fitting and Analysis
Format of Variable Names:

Load Increment Number:
Ranges from L0 (0 lbs.)
up to L8 (266.67 lbs.)
down to L11 (63.4 lbs.)

Identifies Type:
M = Moment (in’Kips)
V = Shear (Kips)
P = Soil Load (Kips/in)
S = Pile Slope (rad)
Y = Pile Displacement (in)

Specific Identifiers,
Counters, Functions,
etc.

Depth Level:
Ranges from D0 (+6.0 in)
to D13 (48.3 in)
Enter in Moment Profiles from Test Results to be Analyzed

Notes: The first level (0), is at lateral load location
Level (2) is a DEFECTIVE GAGE location, and will be overwritten with Moment Value at soil surface.
Level (11) from Moment Profile Test Results is a DEFECTIVE GAGE
and was not entered into the table below.

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Enter in Moment Profiles from Test Results to be Analyzed

Notes: The first level (0), is at lateral load location Level (2) is a DEFECTIVE GAGE location, and will be overwritten with Moment Value at soil surface. Level (11) from Moment Profile Test Results is a DEFECTIVE GAGE and was not entered into the table below.

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<tr>
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</table>
Enter in Moment Profiles from Test Results to be Analyzed

Notes: The first level (0), is at lateral load location.

Level (2) is a DEFECTIVE GAGE location, and will be overwritten with Moment Value at soil surface.

Level (11) from Moment Profile Test Results is a DEFECTIVE GAGE and was not entered into the table below.

\[ j = 0.13 \]

Test #3: Moment Data

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</table>
Define Depth Neighborhoods for Floating Fit

\[ n = 0.4 \]
\[ n_{\text{lin}} = 0.2 \]
\[ n_{\text{inter}} = 0.1 \]
\[ D_{\text{inter}} = \text{Depth}_{\text{inter}} \]
\[ D_{\text{lin}} = 0 \]
\[ D_{\text{nlin}} = 6 \]
\[ D_{\text{ninter}} = 2.6875 \]
\[ D_{\text{nlin}} = 10.8125 \]
\[ D_{\text{ninter}} = 14.5625 \]
\[ D_{\text{nlin}} = 18.3125 \]
\[ D_{\text{ninter}} = 22.0625 \]
\[ D_{\text{nlin}} = 25.8125 \]
\[ D_{\text{ninter}} = 29.5625 \]
\[ D_{\text{nlin}} = 33.3125 \]

Define Depth Neighborhoods (inches) for Floating Fit

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<th>Depth</th>
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<td>10.8125</td>
<td>114.2038</td>
</tr>
<tr>
<td>14.5625</td>
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<tr>
<td>18.3125</td>
<td>179.5316</td>
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<tr>
<td>22.0625</td>
<td>223.0945</td>
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<td>25.8125</td>
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<td>266.6574</td>
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<tr>
<td>33.3125</td>
<td>190.4223</td>
</tr>
</tbody>
</table>

Global Order of Curve Fit

\[ N = 3 \]

Pile Properties

\[ E = 1.9 \times 10^4 \]

\[ I = 0.087343 \]

Overwrite Index 2 (Bad Data Point) of All Moment Data Arrays with the Calculated Moment Intercept Values

\[ M_1 \text{data} = \text{intercept} \left( D_{\text{inter}}, M_1 \text{data} \right) \]
\[ M_2 \text{data} = \text{intercept} \left( D_{\text{inter}}, M_2 \text{data} \right) \]
\[ M_3 \text{data} = \text{intercept} \left( D_{\text{inter}}, M_3 \text{data} \right) \]
\[ M_4 \text{data} = \text{intercept} \left( D_{\text{inter}}, M_4 \text{data} \right) \]
\[ M_5 \text{data} = \text{intercept} \left( D_{\text{inter}}, M_5 \text{data} \right) \]
\[ M_6 \text{data} = \text{intercept} \left( D_{\text{inter}}, M_6 \text{data} \right) \]
\[ M_7 \text{data} = \text{intercept} \left( D_{\text{inter}}, M_7 \text{data} \right) \]
\[ M_8 \text{data} = \text{intercept} \left( D_{\text{inter}}, M_8 \text{data} \right) \]
\[ M_9 \text{data} = \text{intercept} \left( D_{\text{inter}}, M_9 \text{data} \right) \]
\[ M_{10} \text{data} = \text{intercept} \left( D_{\text{inter}}, M_{10} \text{data} \right) \]
\[ M_{11} \text{data} = \text{intercept} \left( D_{\text{inter}}, M_{11} \text{data} \right) \]

Lateral Loads (lb)

\[ j = 0 \ldots 11 \]

\[ j = 0 \]

\[ j = 1 \ldots 10 \]

\[ j = 11 \]

Construct Array of Moment Values at Defectice Gage Locations for Graphical Purposes Only

\[ \text{M1}_{\text{error}} = \text{M2}_{\text{error}} = \text{M3}_{\text{error}} = \text{M4}_{\text{error}} = \text{M5}_{\text{error}} = \text{M6}_{\text{error}} = \text{M7}_{\text{error}} = \text{M8}_{\text{error}} = \text{M9}_{\text{error}} = \text{M10}_{\text{error}} = \text{M11}_{\text{error}} = \]

\[ \text{M1}_{\text{error}} = \text{M2}_{\text{error}} = \text{M3}_{\text{error}} = \text{M4}_{\text{error}} = \text{M5}_{\text{error}} = \text{M6}_{\text{error}} = \text{M7}_{\text{error}} = \text{M8}_{\text{error}} = \text{M9}_{\text{error}} = \text{M10}_{\text{error}} = \text{M11}_{\text{error}} = \]

\[ \text{M1}_{\text{error}} = \text{M2}_{\text{error}} = \text{M3}_{\text{error}} = \text{M4}_{\text{error}} = \text{M5}_{\text{error}} = \text{M6}_{\text{error}} = \text{M7}_{\text{error}} = \text{M8}_{\text{error}} = \text{M9}_{\text{error}} = \text{M10}_{\text{error}} = \text{M11}_{\text{error}} = \]

\[ \text{M1}_{\text{error}} = \text{M2}_{\text{error}} = \text{M3}_{\text{error}} = \text{M4}_{\text{error}} = \text{M5}_{\text{error}} = \text{M6}_{\text{error}} = \text{M7}_{\text{error}} = \text{M8}_{\text{error}} = \text{M9}_{\text{error}} = \text{M10}_{\text{error}} = \text{M11}_{\text{error}} = \]
<table>
<thead>
<tr>
<th>Depth No.</th>
<th>Depth No.</th>
<th>Depth No.</th>
<th>Depth No.</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 2</td>
<td>for L1 = 41.6038 lb</td>
<td>L2 = 74.25844 lb</td>
<td>L3 = 114.2038 lb</td>
</tr>
<tr>
<td>3</td>
<td>ML1D00to2data_nlin = M1data_nlin</td>
<td>ML2D00to2data_nlin = M2data_nlin</td>
<td>ML3D00to2data_nlin = M3data_nlin</td>
</tr>
<tr>
<td>4</td>
<td>ML1D4data_n = M1data_n + 1</td>
<td>ML2D4data_n = M2data_n + 2</td>
<td>ML3D4data_n = M3data_n + 3</td>
</tr>
<tr>
<td>5</td>
<td>ML1D5data_n = M1data_n + 2</td>
<td>ML2D5data_n = M2data_n + 3</td>
<td>ML3D5data_n = M3data_n + 4</td>
</tr>
<tr>
<td>6</td>
<td>ML1D6data_n = M1data_n + 3</td>
<td>ML2D6data_n = M2data_n + 4</td>
<td>ML3D6data_n = M3data_n + 5</td>
</tr>
<tr>
<td>7</td>
<td>ML1D7data_n = M1data_n + 4</td>
<td>ML2D7data_n = M2data_n + 5</td>
<td>ML3D7data_n = M3data_n + 6</td>
</tr>
<tr>
<td>8</td>
<td>ML1D8data_n = M1data_n + 5</td>
<td>ML2D8data_n = M2data_n + 6</td>
<td>ML3D8data_n = M3data_n + 7</td>
</tr>
<tr>
<td>9</td>
<td>ML1D9data_n = M1data_n + 6</td>
<td>ML2D9data_n = M2data_n + 7</td>
<td>ML3D9data_n = M3data_n + 8</td>
</tr>
<tr>
<td>10</td>
<td>ML1D10data_n = M1data_n + 7</td>
<td>ML2D10data_n = M2data_n + 8</td>
<td>ML3D10data_n = M3data_n + 9</td>
</tr>
<tr>
<td>11 to 13</td>
<td>ML1D11to13data_n = M1data_n + 8</td>
<td>ML2D11to13data_n = M2data_n + 9</td>
<td>ML3D11to13data_n = M3data_n + 9</td>
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</table>

L4 = 146.8594 lb
L5 = 179.5316 lb
L6 = 223.0945 lb
Define Moment Neighborhoods for Floating Fits

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<th>Depth No.</th>
<th>L7 = 255.7667 lb</th>
<th>L8 = 266.6574 lb</th>
<th>L9 = 190.4223 lb</th>
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<td>ML7D0to2data __nlin = M7data __nlin</td>
<td>ML8D0to2data __nlin = M8data __nlin</td>
<td>ML9D0to2data __nlin = M9data __nlin</td>
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<td>ML7D3data __n = M7data __n+1</td>
<td>ML8D3data __n = M8data __n+1</td>
<td>ML9D3data __n = M9data __n+1</td>
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<td>4</td>
<td>ML7D4data __n = M7data __n+2</td>
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<tr>
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<td>ML7D5data __n = M7data __n+3</td>
<td>ML8D5data __n = M8data __n+3</td>
<td>ML9D5data __n = M9data __n+3</td>
</tr>
<tr>
<td>6</td>
<td>ML7D6data __n = M7data __n+4</td>
<td>ML8D6data __n = M8data __n+4</td>
<td>ML9D6data __n = M9data __n+4</td>
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<td>ML9D7data __n = M9data __n+5</td>
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<td>8</td>
<td>ML7D8data __n = M7data __n+6</td>
<td>ML8D8data __n = M8data __n+6</td>
<td>ML9D8data __n = M9data __n+6</td>
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<td>9</td>
<td>ML7D9data __n = M7data __n+7</td>
<td>ML8D9data __n = M8data __n+7</td>
<td>ML9D9data __n = M9data __n+7</td>
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<td>11 to 13</td>
<td>ML7D11to13data __n = M7data __n+9</td>
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<tr>
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<tr>
<td>11 to 13</td>
<td>ML10D11to13data __n = M10data __n+9</td>
<td>ML11D11to13data __n = M11data __n+9</td>
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</table>
Perform Floating Regression Fits

\[ \begin{align*}
L_1 &= 41.0008 \text{ lb} \\
L_2 &= 74.25644 \text{ lb} \\
L_3 &= 114.2038 \text{ lb} \\
L_4 &= 146.8594 \text{ lb} \\
L_5 &= 179.5316 \text{ lb} \\
L_6 &= 223.0945 \text{ lb}
\end{align*} \]

<table>
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<td>6</td>
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<td>8</td>
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<td>10</td>
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<td>11-13</td>
<td>11-13</td>
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<td>CL1D5</td>
<td>CL1D6</td>
<td>CL1D7</td>
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<td>CL1D9</td>
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<td>(= \text{regress}(D0to2, ML1D0to2data, N))</td>
<td>(= \text{regress}(D3, ML1D3data, N))</td>
<td>(= \text{regress}(D4, ML1D4data, N))</td>
<td>(= \text{regress}(D5, ML1D5data, N))</td>
<td>(= \text{regress}(D6, ML1D6data, N))</td>
<td>(= \text{regress}(D7, ML1D7data, N))</td>
<td>(= \text{regress}(D8, ML1D8data, N))</td>
<td>(= \text{regress}(D9, ML1D9data, N))</td>
<td>(= \text{regress}(D10, ML1D10data, N))</td>
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Perform Floating Regression Fits

L7 = 255.7667 lb

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<tr>
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<tr>
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<td>11 to 13</td>
<td>CL7D11 to 13</td>
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L8 = 266.6574 lb

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<tr>
<td>11 to 13</td>
<td>CL8D11 to 13</td>
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L9 = 190.4223 lb

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<td>11 to 13</td>
<td>CL9D11 to 13</td>
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L10 = 125.089 lb

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<td>11 to 13</td>
<td>CL10D11 to 13</td>
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L11 = 63.3712 lb

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L10 = 63.3712 lb

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<tr>
<td>11 to 13</td>
<td>CL12D11 to 13</td>
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<td>CL12D10</td>
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</table>

L11 = 63.3712 lb
Moment from Curve Fitting
(in kips)

Depth No. | L1 = 41.6068 lb | L2 = 74.25644 lb
--- | --- | ---
0 to 2 | \( \sum_{n=1}^{N+1} C L1D_{2\cdot n+2} \cdot D^{n-1} \) | \( \sum_{n=1}^{N+1} C L2D_{2\cdot n+2} \cdot D^{n-1} \)
3 | \( \sum_{n=1}^{N+1} C L1D_{3\cdot n+2} \cdot D^{n-1} \) | \( \sum_{n=1}^{N+1} C L2D_{3\cdot n+2} \cdot D^{n-1} \)
4 | \( \sum_{n=1}^{N+1} C L1D_{4\cdot n+2} \cdot D^{n-1} \) | \( \sum_{n=1}^{N+1} C L2D_{4\cdot n+2} \cdot D^{n-1} \)
5 | \( \sum_{n=1}^{N+1} C L1D_{5\cdot n+2} \cdot D^{n-1} \) | \( \sum_{n=1}^{N+1} C L2D_{5\cdot n+2} \cdot D^{n-1} \)
6 | \( \sum_{n=1}^{N+1} C L1D_{6\cdot n+2} \cdot D^{n-1} \) | \( \sum_{n=1}^{N+1} C L2D_{6\cdot n+2} \cdot D^{n-1} \)
7 | \( \sum_{n=1}^{N+1} C L1D_{7\cdot n+2} \cdot D^{n-1} \) | \( \sum_{n=1}^{N+1} C L2D_{7\cdot n+2} \cdot D^{n-1} \)
8 | \( \sum_{n=1}^{N+1} C L1D_{8\cdot n+2} \cdot D^{n-1} \) | \( \sum_{n=1}^{N+1} C L2D_{8\cdot n+2} \cdot D^{n-1} \)
9 | \( \sum_{n=1}^{N+1} C L1D_{9\cdot n+2} \cdot D^{n-1} \) | \( \sum_{n=1}^{N+1} C L2D_{9\cdot n+2} \cdot D^{n-1} \)
10 | \( \sum_{n=1}^{N+1} C L1D_{10\cdot n+2} \cdot D^{n-1} \) | \( \sum_{n=1}^{N+1} C L2D_{10\cdot n+2} \cdot D^{n-1} \)
11 to 13 | \( \sum_{n=1}^{N+1} C L1D_{13\cdot n+2} \cdot D^{n-1} \) | \( \sum_{n=1}^{N+1} C L2D_{13\cdot n+2} \cdot D^{n-1} \)
<table>
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<th>Depth No.</th>
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<tr>
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<td>$L_3 = 114.2038 \text{ lb}$</td>
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<td>0.0 to 2</td>
<td>$ML_{300\text{to}2}(D) = \sum_{n=1}^{N+1} CL_{300\text{to}2} \cdot n + 1$</td>
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<td>$ML_{303}(D) = \sum_{n=1}^{N+1} CL_{303} \cdot n + 1$</td>
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<td>$ML_{304}(D) = \sum_{n=1}^{N+1} CL_{304} \cdot n + 1$</td>
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<td>$ML_{305}(D) = \sum_{n=1}^{N+1} CL_{305} \cdot n + 1$</td>
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<tr>
<td>6</td>
<td>$ML_{306}(D) = \sum_{n=1}^{N+1} CL_{306} \cdot n + 1$</td>
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<tr>
<td>7</td>
<td>$ML_{307}(D) = \sum_{n=1}^{N+1} CL_{307} \cdot n + 1$</td>
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<tr>
<td>8</td>
<td>$ML_{308}(D) = \sum_{n=1}^{N+1} CL_{308} \cdot n + 1$</td>
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<tr>
<td>9</td>
<td>$ML_{309}(D) = \sum_{n=1}^{N+1} CL_{309} \cdot n + 1$</td>
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<tr>
<td>10</td>
<td>$ML_{310}(D) = \sum_{n=1}^{N+1} CL_{310} \cdot n + 1$</td>
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<tr>
<td>11 to 13</td>
<td>$ML_{311\text{to}13}(D) = \sum_{n=1}^{N+1} CL_{311\text{to}13} \cdot n + 1$</td>
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### Moment from Curve Fitting

**Depth No.**

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<th>L5 = 179.5316 lb</th>
<th>L6 = 223.0945 lb</th>
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<td>( ML500to2(D) = \sum_{n=1}^{N+1} CL500to2 \cdot D^n - 1 )</td>
<td>( ML600to2(D) = \sum_{n=1}^{N+1} CL600to2 \cdot D^n - 1 )</td>
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<tr>
<td>3</td>
<td>( ML53(D) = \sum_{n=1}^{N+1} CL53 \cdot D^n - 1 )</td>
<td>( ML63(D) = \sum_{n=1}^{N+1} CL63 \cdot D^n - 1 )</td>
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<tr>
<td>4</td>
<td>( ML54(D) = \sum_{n=1}^{N+1} CL54 \cdot D^n - 1 )</td>
<td>( ML64(D) = \sum_{n=1}^{N+1} CL64 \cdot D^n - 1 )</td>
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<tr>
<td>5</td>
<td>( ML55(D) = \sum_{n=1}^{N+1} CL55 \cdot D^n - 1 )</td>
<td>( ML65(D) = \sum_{n=1}^{N+1} CL65 \cdot D^n - 1 )</td>
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<td>( ML56(D) = \sum_{n=1}^{N+1} CL56 \cdot D^n - 1 )</td>
<td>( ML66(D) = \sum_{n=1}^{N+1} CL66 \cdot D^n - 1 )</td>
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<td>( ML57(D) = \sum_{n=1}^{N+1} CL57 \cdot D^n - 1 )</td>
<td>( ML67(D) = \sum_{n=1}^{N+1} CL67 \cdot D^n - 1 )</td>
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<td>8</td>
<td>( ML58(D) = \sum_{n=1}^{N+1} CL58 \cdot D^n - 1 )</td>
<td>( ML68(D) = \sum_{n=1}^{N+1} CL68 \cdot D^n - 1 )</td>
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<tr>
<td>9</td>
<td>( ML59(D) = \sum_{n=1}^{N+1} CL59 \cdot D^n - 1 )</td>
<td>( ML69(D) = \sum_{n=1}^{N+1} CL69 \cdot D^n - 1 )</td>
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<td>10</td>
<td>( ML510(D) = \sum_{n=1}^{N+1} CL510 \cdot D^n - 1 )</td>
<td>( ML610(D) = \sum_{n=1}^{N+1} CL610 \cdot D^n - 1 )</td>
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<tr>
<td>11 to 13</td>
<td>( ML511to13(D) = \sum_{n=1}^{N+1} CL511to13 \cdot D^n - 1 )</td>
<td>( ML611to13(D) = \sum_{n=1}^{N+1} CL611to13 \cdot D^n - 1 )</td>
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## Moment from Curve Fitting (in'kips)

<table>
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<tr>
<th>Depth No.</th>
<th>Equation</th>
<th>Value</th>
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<tbody>
<tr>
<td>0 to 2</td>
<td>[ M_{L7}(D) = \sum_{n=1}^{N+1} C_{L7D2} \cdot D^{n-1} ]</td>
<td>[ L_7 = 255.7667 \text{ lb} ]</td>
</tr>
<tr>
<td>3</td>
<td>[ M_{L7}(D) = \sum_{n=1}^{N+1} C_{L7D3} \cdot D^{n-1} ]</td>
<td>[ L_7 = 255.7667 \text{ lb} ]</td>
</tr>
<tr>
<td>4</td>
<td>[ M_{L7}(D) = \sum_{n=1}^{N+1} C_{L7D4} \cdot D^{n-1} ]</td>
<td>[ L_7 = 255.7667 \text{ lb} ]</td>
</tr>
<tr>
<td>5</td>
<td>[ M_{L7}(D) = \sum_{n=1}^{N+1} C_{L7D5} \cdot D^{n-1} ]</td>
<td>[ L_7 = 255.7667 \text{ lb} ]</td>
</tr>
<tr>
<td>6</td>
<td>[ M_{L7}(D) = \sum_{n=1}^{N+1} C_{L7D6} \cdot D^{n-1} ]</td>
<td>[ L_7 = 255.7667 \text{ lb} ]</td>
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<td>7</td>
<td>[ M_{L7}(D) = \sum_{n=1}^{N+1} C_{L7D7} \cdot D^{n-1} ]</td>
<td>[ L_7 = 255.7667 \text{ lb} ]</td>
</tr>
<tr>
<td>8</td>
<td>[ M_{L7}(D) = \sum_{n=1}^{N+1} C_{L7D8} \cdot D^{n-1} ]</td>
<td>[ L_7 = 255.7667 \text{ lb} ]</td>
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<td>9</td>
<td>[ M_{L7}(D) = \sum_{n=1}^{N+1} C_{L7D9} \cdot D^{n-1} ]</td>
<td>[ L_7 = 255.7667 \text{ lb} ]</td>
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<tr>
<td>10</td>
<td>[ M_{L7}(D) = \sum_{n=1}^{N+1} C_{L7D10} \cdot D^{n-1} ]</td>
<td>[ L_7 = 255.7667 \text{ lb} ]</td>
</tr>
<tr>
<td>11 to 13</td>
<td>[ M_{L7}(D) = \sum_{n=1}^{N+1} C_{L7D11} \cdot D^{n-1} ]</td>
<td>[ L_7 = 255.7667 \text{ lb} ]</td>
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<td>14</td>
<td>[ M_{L7}(D) = \sum_{n=1}^{N+1} C_{L7D12} \cdot D^{n-1} ]</td>
<td>[ L_7 = 255.7667 \text{ lb} ]</td>
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<td>[ M_{L7}(D) = \sum_{n=1}^{N+1} C_{L7D13} \cdot D^{n-1} ]</td>
<td>[ L_7 = 255.7667 \text{ lb} ]</td>
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### Moment from Curve Fitting

$(\text{in} \times \text{kips})$

<table>
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<tr>
<th>Depth No.</th>
<th>$L_9 = 190.4223 \text{ lb}$</th>
<th>$L_{10} = 125.0800 \text{ lb}$</th>
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<tbody>
<tr>
<td>0 to 2</td>
<td>$ML_{9D0to2}(D) = \sum_{n=1}^{N+1} CL_{9D0to2} n_{+2} D^{n-1}$</td>
<td>$ML_{10D0to2}(D) = \sum_{n=1}^{N+1} CL_{10D0to2} n_{+2} D^{n-1}$</td>
</tr>
<tr>
<td>3</td>
<td>$ML_{9D3}(D) = \sum_{n=1}^{N+1} CL_{9D3} n_{+2} D^{n-1}$</td>
<td>$ML_{10D3}(D) = \sum_{n=1}^{N+1} CL_{10D3} n_{+2} D^{n-1}$</td>
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<tr>
<td>4</td>
<td>$ML_{9D4}(D) = \sum_{n=1}^{N+1} CL_{9D4} n_{+2} D^{n-1}$</td>
<td>$ML_{10D4}(D) = \sum_{n=1}^{N+1} CL_{10D4} n_{+2} D^{n-1}$</td>
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<td>6</td>
<td>$ML_{9D6}(D) = \sum_{n=1}^{N+1} CL_{9D6} n_{+2} D^{n-1}$</td>
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<td>7</td>
<td>$ML_{9D7}(D) = \sum_{n=1}^{N+1} CL_{9D7} n_{+2} D^{n-1}$</td>
<td>$ML_{10D7}(D) = \sum_{n=1}^{N+1} CL_{10D7} n_{+2} D^{n-1}$</td>
</tr>
<tr>
<td>8</td>
<td>$ML_{9D8}(D) = \sum_{n=1}^{N+1} CL_{9D8} n_{+2} D^{n-1}$</td>
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<td>$ML_{9D9}(D) = \sum_{n=1}^{N+1} CL_{9D9} n_{+2} D^{n-1}$</td>
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<tr>
<td>10</td>
<td>$ML_{9D10}(D) = \sum_{n=1}^{N+1} CL_{9D10} n_{+2} D^{n-1}$</td>
<td>$ML_{10D10}(D) = \sum_{n=1}^{N+1} CL_{10D10} n_{+2} D^{n-1}$</td>
</tr>
<tr>
<td>11 to 13</td>
<td>$ML_{9D11to13}(D) = \sum_{n=1}^{N+1} CL_{9D11to13} n_{+2} D^{n-1}$</td>
<td>$ML_{10D11to13}(D) = \sum_{n=1}^{N+1} CL_{10D11to13} n_{+2} D^{n-1}$</td>
</tr>
</tbody>
</table>
Moment from Curve Fitting
(in kips)

\[ L_{11} = 63.3712 \text{ lb} \]

\[ M_{L1D0to2(D)} = \sum_{n=1}^{N+1} C_{L1D0to2(n+2)} D^{n-1} \]

\[ M_{L1D3(D)} = \sum_{n=1}^{N+1} C_{L1D3(n+2)} D^{n-1} \]

\[ M_{L1D4(D)} = \sum_{n=1}^{N+1} C_{L1D4(n+2)} D^{n-1} \]

\[ M_{L1D5(D)} = \sum_{n=1}^{N+1} C_{L1D5(n+2)} D^{n-1} \]

\[ M_{L1D6(D)} = \sum_{n=1}^{N+1} C_{L1D6(n+2)} D^{n-1} \]

\[ M_{L1D7(D)} = \sum_{n=1}^{N+1} C_{L1D7(n+2)} D^{n-1} \]

\[ M_{L1D8(D)} = \sum_{n=1}^{N+1} C_{L1D8(n+2)} D^{n-1} \]

\[ M_{L1D9(D)} = \sum_{n=1}^{N+1} C_{L1D9(n+2)} D^{n-1} \]

\[ M_{L1D10(D)} = \sum_{n=1}^{N+1} C_{L1D10(n+2)} D^{n-1} \]

\[ M_{L1D11to13(D)} = \sum_{n=1}^{N+1} C_{L1D11to13(n+2)} D^{n-1} \]
<table>
<thead>
<tr>
<th>Depth No.</th>
<th>Shear from Derivative of Fitted Moment (kips) for $L_1 = 41.6008$ lb</th>
<th>Shear from Derivative of Fitted Moment (kips) for $L_2 = 74.2564$ lb</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 2</td>
<td>$VL_1D002(D) = \sum_{n=2}^{1} (n-1) \cdot CL_1D002_{n} \cdot D^{n-2}$</td>
<td>$VL_2D002(D) = \sum_{n=2}^{1} (n-1) \cdot CL_2D002_{n} \cdot D^{n-2}$</td>
</tr>
<tr>
<td>3</td>
<td>$VL_1D3(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_1D3_{n} \cdot D^{n-2}$</td>
<td>$VL_2D3(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_2D3_{n} \cdot D^{n-2}$</td>
</tr>
<tr>
<td>4</td>
<td>$VL_1D4(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_1D4_{n} \cdot D^{n-2}$</td>
<td>$VL_2D4(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_2D4_{n} \cdot D^{n-2}$</td>
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<tr>
<td>5</td>
<td>$VL_1D5(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_1D5_{n} \cdot D^{n-2}$</td>
<td>$VL_2D5(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_2D5_{n} \cdot D^{n-2}$</td>
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<tr>
<td>6</td>
<td>$VL_1D6(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_1D6_{n} \cdot D^{n-2}$</td>
<td>$VL_2D6(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_2D6_{n} \cdot D^{n-2}$</td>
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<tr>
<td>7</td>
<td>$VL_1D7(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_1D7_{n} \cdot D^{n-2}$</td>
<td>$VL_2D7(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_2D7_{n} \cdot D^{n-2}$</td>
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<tr>
<td>8</td>
<td>$VL_1D8(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_1D8_{n} \cdot D^{n-2}$</td>
<td>$VL_2D8(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_2D8_{n} \cdot D^{n-2}$</td>
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<td>9</td>
<td>$VL_1D9(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_1D9_{n} \cdot D^{n-2}$</td>
<td>$VL_2D9(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_2D9_{n} \cdot D^{n-2}$</td>
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<td>10</td>
<td>$VL_1D10(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_1D10_{n} \cdot D^{n-2}$</td>
<td>$VL_2D10(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_2D10_{n} \cdot D^{n-2}$</td>
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<tr>
<td>11 to 13</td>
<td>$VL_1D11to13(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_1D11to13_{n} \cdot D^{n-2}$</td>
<td>$VL_2D11to13(D) = \sum_{n=2}^{N+1} (n-1) \cdot CL_2D11to13_{n} \cdot D^{n-2}$</td>
</tr>
<tr>
<td>Depth No.</td>
<td>Shear from Derivative of Fitted Moment (kips)</td>
<td></td>
</tr>
<tr>
<td>-----------</td>
<td>---------------------------------------------</td>
<td></td>
</tr>
<tr>
<td>0 to 2</td>
<td>( VL_{3D002(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{3D002} \cdot n^{-2} \cdot D^{n-2} ) ( VL_{4D002(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{4D002} \cdot n^{-2} \cdot D^{n-2} )</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>( VL_{3D3(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{3D3} \cdot n^{-2} \cdot D^{n-2} ) ( VL_{4D3(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{4D3} \cdot n^{-2} \cdot D^{n-2} )</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>( VL_{3D4(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{3D4} \cdot n^{-2} \cdot D^{n-2} ) ( VL_{4D4(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{4D4} \cdot n^{-2} \cdot D^{n-2} )</td>
<td></td>
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<tr>
<td>5</td>
<td>( VL_{3D5(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{3D5} \cdot n^{-2} \cdot D^{n-2} ) ( VL_{4D5(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{4D5} \cdot n^{-2} \cdot D^{n-2} )</td>
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<td>6</td>
<td>( VL_{3D6(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{3D6} \cdot n^{-2} \cdot D^{n-2} ) ( VL_{4D6(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{4D6} \cdot n^{-2} \cdot D^{n-2} )</td>
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</tr>
<tr>
<td>7</td>
<td>( VL_{3D7(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{3D7} \cdot n^{-2} \cdot D^{n-2} ) ( VL_{4D7(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{4D7} \cdot n^{-2} \cdot D^{n-2} )</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>( VL_{3D8(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{3D8} \cdot n^{-2} \cdot D^{n-2} ) ( VL_{4D8(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{4D8} \cdot n^{-2} \cdot D^{n-2} )</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>( VL_{3D9(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{3D9} \cdot n^{-2} \cdot D^{n-2} ) ( VL_{4D9(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{4D9} \cdot n^{-2} \cdot D^{n-2} )</td>
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</tr>
<tr>
<td>10</td>
<td>( VL_{3D10(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{3D10} \cdot n^{-2} \cdot D^{n-2} ) ( VL_{4D10(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{4D10} \cdot n^{-2} \cdot D^{n-2} )</td>
<td></td>
</tr>
<tr>
<td>11 to 13</td>
<td>( VL_{3D11to13(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{3D11to13} \cdot n^{-2} \cdot D^{n-2} ) ( VL_{4D11to13(D)} = \sum_{n=2}^{N+1} (n-1) \cdot CL_{4D11to13} \cdot n^{-2} \cdot D^{n-2} )</td>
<td></td>
</tr>
</tbody>
</table>

For \( L3 = 114.2038 \text{ lb} \), the shear from the derivative of fitted moment is given by the above equations. For \( L4 = 146.8594 \text{ lb} \), similar expressions apply. This table presents the shear values for different depths, with the coefficients \( CL \) and power terms \( n^{-2} \cdot D^{n-2} \) indicating the specific contributions at each depth range.
Shear from Derivative of Fitted Moment

for $L_5 = 179.5316$ lb

\[
V_{L5D0to2} (D) = \sum_{n=2}^{N+1} (n-1) C_{L5D0n+2} D^{n-2}
\]

for $L_6 = 223.0945$ lb

\[
V_{L6D0to2} (D) = \sum_{n=2}^{N+1} (n-1) C_{L6D0n+2} D^{n-2}
\]
Shear from Derivative of Fitted Moment

for L7 = 255.7867 lb

for L8 = 266.6574 lb

\[
\begin{align*}
\text{V7D02(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL7D02_{n+2} D^{n-2} \\
\text{V8D02(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL8D02_{n+2} D^{n-2} \\
\text{V7D3(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL7D3_{n+2} D^{n-2} \\
\text{V8D3(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL8D3_{n+2} D^{n-2} \\
\text{V7D4(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL7D4_{n+2} D^{n-2} \\
\text{V8D4(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL8D4_{n+2} D^{n-2} \\
\text{V7D5(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL7D5_{n+2} D^{n-2} \\
\text{V8D5(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL8D5_{n+2} D^{n-2} \\
\text{V7D6(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL7D6_{n+2} D^{n-2} \\
\text{V8D6(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL8D6_{n+2} D^{n-2} \\
\text{V7D7(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL7D7_{n+2} D^{n-2} \\
\text{V8D7(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL8D7_{n+2} D^{n-2} \\
\text{V7D8(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL7D8_{n+2} D^{n-2} \\
\text{V8D8(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL8D8_{n+2} D^{n-2} \\
\text{V7D9(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL7D9_{n+2} D^{n-2} \\
\text{V8D9(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL8D9_{n+2} D^{n-2} \\
\text{V7D10(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL7D10_{n+2} D^{n-2} \\
\text{V8D10(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL8D10_{n+2} D^{n-2} \\
\text{V7D11(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL7D11_{n+2} D^{n-2} \\
\text{V8D11(D)} &= \sum_{n=2}^{N+1} (n-1) \cdot CL8D11_{n+2} D^{n-2}
\end{align*}
\]
Shear from Derivative of Fitted Moment (kips)

for \( L_9 = 190.4223 \) lb

\[
VL9D0\theta 2(\theta) = \sum_{n=2}^{1+1} (n-1) CL9D0\theta 2 + 2 D^{n-2}
\]

\[
VL10D0\theta 2(\theta) = \sum_{n=2}^{1+1} (n-1) CL10D0\theta 2 + 2 D^{n-2}
\]

for \( L_{10} = 41.1250890 \) lb

\[
VL9D3(\theta) = \sum_{n=2}^{N+1} (n-1) CL9D3 + 2 D^{n-2}
\]

\[
VL10D3(\theta) = \sum_{n=2}^{N+1} (n-1) CL10D3 + 2 D^{n-2}
\]

\[
VL9D4(\theta) = \sum_{n=2}^{N+1} (n-1) CL9D4 + 2 D^{n-2}
\]

\[
VL10D4(\theta) = \sum_{n=2}^{N+1} (n-1) CL10D4 + 2 D^{n-2}
\]

\[
VL9D5(\theta) = \sum_{n=2}^{N+1} (n-1) CL9D5 + 2 D^{n-2}
\]

\[
VL10D5(\theta) = \sum_{n=2}^{N+1} (n-1) CL10D5 + 2 D^{n-2}
\]

\[
VL9D6(\theta) = \sum_{n=2}^{N+1} (n-1) CL9D6 + 2 D^{n-2}
\]

\[
VL10D6(\theta) = \sum_{n=2}^{N+1} (n-1) CL10D6 + 2 D^{n-2}
\]

\[
VL9D7(\theta) = \sum_{n=2}^{N+1} (n-1) CL9D7 + 2 D^{n-2}
\]

\[
VL10D7(\theta) = \sum_{n=2}^{N+1} (n-1) CL10D7 + 2 D^{n-2}
\]

\[
VL9D8(\theta) = \sum_{n=2}^{N+1} (n-1) CL9D8 + 2 D^{n-2}
\]

\[
VL10D8(\theta) = \sum_{n=2}^{N+1} (n-1) CL10D8 + 2 D^{n-2}
\]

\[
VL9D9(\theta) = \sum_{n=2}^{N+1} (n-1) CL9D9 + 2 D^{n-2}
\]

\[
VL10D9(\theta) = \sum_{n=2}^{N+1} (n-1) CL10D9 + 2 D^{n-2}
\]

\[
VL9D10(\theta) = \sum_{n=2}^{N+1} (n-1) CL9D10 + 2 D^{n-2}
\]

\[
VL10D10(\theta) = \sum_{n=2}^{N+1} (n-1) CL10D10 + 2 D^{n-2}
\]

\[
VL9D11to13(\theta) = \sum_{n=2}^{N+1} (n-1) CL9D11to13 + 2 D^{n-2}
\]

\[
VL10D11to13(\theta) = \sum_{n=2}^{N+1} (n-1) CL10D11to13 + 2 D^{n-2}
\]
Depth No. | Shear from Derivative of Fitted Moment (kips) for $L_{11} = 63.3712$ lb

0 to 2

$V_{L1D02}(D) = \sum_{n=2}^{1+1} (n-1)C_{L1D02n+2}D^{n-2}$

3

$V_{L1D3}(D) = \sum_{n=2}^{N+1} (n-1)C_{L1D3n+2}D^{n-2}$

4

$V_{L1D4}(D) = \sum_{n=2}^{N+1} (n-1)C_{L1D4n+2}D^{n-2}$

5

$V_{L1D5}(D) = \sum_{n=2}^{N+1} (n-1)C_{L1D5n+2}D^{n-2}$

6

$V_{L1D6}(D) = \sum_{n=2}^{N+1} (n-1)C_{L1D6n+2}D^{n-2}$

7

$V_{L1D7}(D) = \sum_{n=2}^{N+1} (n-1)C_{L1D7n+2}D^{n-2}$

8

$V_{L1D8}(D) = \sum_{n=2}^{N+1} (n-1)C_{L1D8n+2}D^{n-2}$

9

$V_{L1D9}(D) = \sum_{n=2}^{N+1} (n-1)C_{L1D9n+2}D^{n-2}$

10

$V_{L1D10}(D) = \sum_{n=2}^{N+1} (n-1)C_{L1D10n+2}D^{n-2}$

11 to 13

$V_{L1D11to13}(D) = \sum_{n=2}^{N+1} (n-1)C_{L1D11to13n+2}D^{n-2}$
Depth No. for \( L_1 = 41.6008 \) lb

<table>
<thead>
<tr>
<th>Depth No.</th>
<th>Load from Derivative of Shear (kips/in) for ( L_2 = 74.2564 ) lb</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 to 2</td>
<td>( P L_{1D0002(D)} = 0 )</td>
</tr>
<tr>
<td>3</td>
<td>( \sum_{n=3}^{N+1} (n-2)(n-1)CL_{1D3_{n+2}}D^{n-3} )</td>
</tr>
<tr>
<td>4</td>
<td>( \sum_{n=3}^{N+1} (n-2)(n-1)CL_{1D4_{n+2}}D^{n-3} )</td>
</tr>
<tr>
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<td>( \sum_{n=3}^{N+1} (n-2)(n-1)CL_{1D5_{n+2}}D^{n-3} )</td>
</tr>
<tr>
<td>6</td>
<td>( \sum_{n=3}^{N+1} (n-2)(n-1)CL_{1D6_{n+2}}D^{n-3} )</td>
</tr>
<tr>
<td>7</td>
<td>( \sum_{n=3}^{N+1} (n-2)(n-1)CL_{1D7_{n+2}}D^{n-3} )</td>
</tr>
<tr>
<td>8</td>
<td>( \sum_{n=3}^{N+1} (n-2)(n-1)CL_{1D8_{n+2}}D^{n-3} )</td>
</tr>
<tr>
<td>9</td>
<td>( \sum_{n=3}^{N+1} (n-2)(n-1)CL_{1D9_{n+2}}D^{n-3} )</td>
</tr>
<tr>
<td>10</td>
<td>( \sum_{n=3}^{N+1} (n-2)(n-1)CL_{1D10_{n+2}}D^{n-3} )</td>
</tr>
<tr>
<td>11 to 13</td>
<td>( \sum_{n=3}^{N+1} (n-2)(n-1)CL_{1D11to13_{n+2}}D^{n-3} )</td>
</tr>
<tr>
<td></td>
<td>( PL_{1D11to13(D)} = \sum_{n=3}^{N+1} (n-2)(n-1)CL_{1D11to13_{n+2}}D^{n-3} )</td>
</tr>
</tbody>
</table>
Depth No. | Load from Derivative of Shear (kips / in) for L3 = 114.2038 lb | for L4 = 146.8594 lb
--- | --- | ---
0 to 2 | PL3D002(D) = 0 | PL4D002(D) = 0
3 | PL3D3(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL3D_n \cdot D^{n-3} \) | PL4D3(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL4D_n \cdot D^{n-3} \)
4 | PL3D4(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL3D_n \cdot D^{n-3} \) | PL4D4(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL4D_n \cdot D^{n-3} \)
5 | PL3D5(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL3D_n \cdot D^{n-3} \) | PL4D5(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL4D_n \cdot D^{n-3} \)
6 | PL3D6(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL3D_n \cdot D^{n-3} \) | PL4D6(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL4D_n \cdot D^{n-3} \)
7 | PL3D7(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL3D_n \cdot D^{n-3} \) | PL4D7(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL4D_n \cdot D^{n-3} \)
8 | PL3D8(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL3D_n \cdot D^{n-3} \) | PL4D8(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL4D_n \cdot D^{n-3} \)
9 | PL3D9(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL3D_n \cdot D^{n-3} \) | PL4D9(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL4D_n \cdot D^{n-3} \)
10 | PL3D10(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL3D_n \cdot D^{n-3} \) | PL4D10(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL4D_n \cdot D^{n-3} \)
11 to 13 | PL3D11 to 13(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL3D_n \cdot D^{n-3} \) | PL4D11 to 13(D) = \( \sum_{n=3}^{N+1} (n-2)(n-1) \cdot CL4D_n \cdot D^{n-3} \)
<table>
<thead>
<tr>
<th>Depth No.</th>
<th>Load from Derivative of Shear (kips/in)</th>
<th>Depth No.</th>
<th>Load from Derivative of Shear (kips/in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0to2</td>
<td>( \text{PLSD0to2(D)} = 0 )</td>
<td>0to2</td>
<td>( \text{PLSD0to2(D)} = 0 )</td>
</tr>
<tr>
<td>3</td>
<td>( \sum_{n=3}^{N+1} (n-2)(n-1) \text{CL5D}_3 \cdot D^{n-3} )</td>
<td>3</td>
<td>( \sum_{n=3}^{N+1} (n-2)(n-1) \text{CL6D}_3 \cdot D^{n-3} )</td>
</tr>
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<td>11to13</td>
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<td>( \sum_{n=3}^{N+1} (n-2)(n-1) \text{CL6D}_{11to13} \cdot D^{n-3} )</td>
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For \( L_5 = 170.3314 \text{ lb} \) and \( L_6 = 223.0945 \text{ lb} \)
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<td>$PL7D0 +2(D) = 0$</td>
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<tr>
<td>3</td>
<td>$PL7D3(D) = \sum_{n=3}^{N+1} (n-2)(n-1)CL7D3_{n+2}D^{n-3}$</td>
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<td>$PL8D0 +2(D) = 0$</td>
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<td>$PL7D5(D) = \sum_{n=3}^{N+1} (n-2)(n-1)CL7D5_{n+2}D^{n-3}$</td>
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<td>$PL7D10(D) = \sum_{n=3}^{N+1} (n-2)(n-1)CL7D10_{n+2}D^{n-3}$</td>
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Load from Derivative of Shear (kips / in)

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<th>PL10D0to2(D)</th>
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<td>PL10D3(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL10D3_{n+2} D^{n-3}</td>
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<td>PL10D4(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL10D4_{n+2} D^{n-3}</td>
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<td>PL9D6(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL9D6_{n+2} D^{n-3}</td>
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<tr>
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<td>PL10D6(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL10D6_{n+2} D^{n-3}</td>
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<td>PL9D7(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL9D7_{n+2} D^{n-3}</td>
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<td>PL10D7(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL10D7_{n+2} D^{n-3}</td>
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<td>PL9D8(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL9D8_{n+2} D^{n-3}</td>
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<td>PL10D8(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL10D8_{n+2} D^{n-3}</td>
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<td>PL9D9(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL9D9_{n+2} D^{n-3}</td>
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<td>PL9D10(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL9D10_{n+2} D^{n-3}</td>
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<td>PL10D10(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL10D10_{n+2} D^{n-3}</td>
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<td>11 to 13</td>
<td>PL9D11to13(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL9D11to13_{n+2} D^{n-3}</td>
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<td>PL10D11to13(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL10D11to13_{n+2} D^{n-3}</td>
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Load from Derivative of Shear
(kips/in)

Depth No. for \( L_{11} = 63.3712 \text{ lb} \)

3 \( PL_{11D3}(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL_{11D3n}^2 D^{n-3} \)

4 \( PL_{11D4}(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL_{11D4n}^2 D^{n-3} \)

5 \( PL_{11D5}(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL_{11D5n}^2 D^{n-3} \)

6 \( PL_{11D6}(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL_{11D6n}^2 D^{n-3} \)

7 \( PL_{11D7}(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL_{11D7n}^2 D^{n-3} \)

8 \( PL_{11D8}(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL_{11D8n}^2 D^{n-3} \)

9 \( PL_{11D9}(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL_{11D9n}^2 D^{n-3} \)

10 \( PL_{11D10}(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL_{11D10n}^2 D^{n-3} \)

11 to 13 \( PL_{11D10 to 13}(D) = \sum_{n=3}^{N+1} (n-2)(n-1) CL_{11D10 to 13n}^2 D^{n-3} \)
## Define Shear Vectors

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Define Soil Load Vectors

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<td>PL₆₀₀₀₂ Depth₁²</td>
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<td>PL₃₀₀₀₂ Depth₁³</td>
<td>PL₄₀₀₀₂ Depth₁³</td>
<td>PL₅₀₀₀₂ Depth₁³</td>
<td>PL₆₀₀₀₂ Depth₁³</td>
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<td>PL₆₀₀₀₂ Depth₁₀⁰</td>
</tr>
</tbody>
</table>

Define Soil Load Vectors
Create CUBIC SPLINE for MOMENT DATA that can be used for NUMERICAL INTEGRATION

DEPTH and MOMENT data must be arranged in ASCENDING ORDER for the CUBIC SPLINE Function to work.

Moment Data (in*kips)

<table>
<thead>
<tr>
<th>Moment Data (in*Kips)</th>
<th>Gage Depths (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>SplineM1data = M1data</td>
<td>SplineDepth = Depth</td>
</tr>
<tr>
<td>SplineM2data = M2data</td>
<td></td>
</tr>
<tr>
<td>SplineM3data = M3data</td>
<td></td>
</tr>
<tr>
<td>SplineM4data = M4data</td>
<td></td>
</tr>
<tr>
<td>SplineM5data = M5data</td>
<td></td>
</tr>
<tr>
<td>SplineM6data = M6data</td>
<td></td>
</tr>
<tr>
<td>SplineM7data = M7data</td>
<td></td>
</tr>
<tr>
<td>SplineM8data = M8data</td>
<td></td>
</tr>
<tr>
<td>SplineM9data = M9data</td>
<td></td>
</tr>
<tr>
<td>SplineM10data = M10data</td>
<td></td>
</tr>
<tr>
<td>SplineM11data = M11data</td>
<td></td>
</tr>
</tbody>
</table>

NUMERICAL INTEGRATION

Total # of Increments

\[ d = 0, 5600 \]

Beginning Depth

\[ \text{depth}_0 = 50.0025 \]

Step Size

\[ \text{depth}_{d+1} = \text{depth}_d + 0.01 \]

Top of Pile

\[ \text{depth}_{5607} = 6.0614 \]

\[ \text{depth}_{5600} = 0 \]

\[ \text{is} = 1, 5600 \]

\[ \text{ig} = 169, 5600 \]

Cubic SPLINE of the Moment Data (in*kips)

\[ \text{SplineM1}_d = \text{interp} \text{ Ispline(SplineDepth, SplineM1data), SplineDepth, SplineM1data, depth}_d \]
\[ \text{SplineM2}_d = \text{interp} \text{ Ispline(SplineDepth, SplineM2data), SplineDepth, SplineM2data, depth}_d \]
\[ \text{SplineM3}_d = \text{interp} \text{ Ispline(SplineDepth, SplineM3data), SplineDepth, SplineM3data, depth}_d \]
\[ \text{SplineM4}_d = \text{interp} \text{ Ispline(SplineDepth, SplineM4data), SplineDepth, SplineM4data, depth}_d \]
\[ \text{SplineM5}_d = \text{interp} \text{ Ispline(SplineDepth, SplineM5data), SplineDepth, SplineM5data, depth}_d \]
\[ \text{SplineM6}_d = \text{interp} \text{ Ispline(SplineDepth, SplineM6data), SplineDepth, SplineM6data, depth}_d \]
\[ \text{SplineM7}_d = \text{interp} \text{ Ispline(SplineDepth, SplineM7data), SplineDepth, SplineM7data, depth}_d \]
\[ \text{SplineM8}_d = \text{interp} \text{ Ispline(SplineDepth, SplineM8data), SplineDepth, SplineM8data, depth}_d \]
\[ \text{SplineM9}_d = \text{interp} \text{ Ispline(SplineDepth, SplineM9data), SplineDepth, SplineM9data, depth}_d \]
\[ \text{SplineM10}_d = \text{interp} \text{ Ispline(SplineDepth, SplineM10data), SplineDepth, SplineM10data, depth}_d \]
\[ \text{SplineM11}_d = \text{interp} \text{ Ispline(SplineDepth, SplineM11data), SplineDepth, SplineM11data, depth}_d \]
<table>
<thead>
<tr>
<th>Boundry Condition of Slope (S) = 0 at Depth = -50.0025 in.</th>
<th>SLOPE (rad) of Pile from Numerical Integration of Moment Spline</th>
</tr>
</thead>
<tbody>
<tr>
<td>$S_{10} = 0$</td>
<td>$S_{11} = S_{11_{11}} - \frac{1}{E} \cdot \frac{1}{2} \cdot \text{depth} \cdot \text{depth} - 1$</td>
</tr>
<tr>
<td>$S_{20} = 0$</td>
<td>$S_{22} = S_{22_{22}} - \frac{1}{E} \cdot \frac{1}{2} \cdot \text{depth} \cdot \text{depth} - 1$</td>
</tr>
<tr>
<td>$S_{30} = 0$</td>
<td>$S_{33} = S_{33_{33}} - \frac{1}{E} \cdot \frac{1}{2} \cdot \text{depth} \cdot \text{depth} - 1$</td>
</tr>
<tr>
<td>$S_{40} = 0$</td>
<td>$S_{44} = S_{44_{44}} - \frac{1}{E} \cdot \frac{1}{2} \cdot \text{depth} \cdot \text{depth} - 1$</td>
</tr>
<tr>
<td>$S_{50} = 0$</td>
<td>$S_{55} = S_{55_{55}} - \frac{1}{E} \cdot \frac{1}{2} \cdot \text{depth} \cdot \text{depth} - 1$</td>
</tr>
<tr>
<td>$S_{60} = 0$</td>
<td>$S_{66} = S_{66_{66}} - \frac{1}{E} \cdot \frac{1}{2} \cdot \text{depth} \cdot \text{depth} - 1$</td>
</tr>
<tr>
<td>$S_{70} = 0$</td>
<td>$S_{77} = S_{77_{77}} - \frac{1}{E} \cdot \frac{1}{2} \cdot \text{depth} \cdot \text{depth} - 1$</td>
</tr>
<tr>
<td>$S_{80} = 0$</td>
<td>$S_{88} = S_{88_{88}} - \frac{1}{E} \cdot \frac{1}{2} \cdot \text{depth} \cdot \text{depth} - 1$</td>
</tr>
<tr>
<td>$S_{90} = 0$</td>
<td>$S_{99} = S_{99_{99}} - \frac{1}{E} \cdot \frac{1}{2} \cdot \text{depth} \cdot \text{depth} - 1$</td>
</tr>
<tr>
<td>$S_{100} = 0$</td>
<td>$S_{110} = S_{110_{110}} - \frac{1}{E} \cdot \frac{1}{2} \cdot \text{depth} \cdot \text{depth} - 1$</td>
</tr>
<tr>
<td>$S_{110} = 0$</td>
<td>$S_{120} = S_{120_{120}} - \frac{1}{E} \cdot \frac{1}{2} \cdot \text{depth} \cdot \text{depth} - 1$</td>
</tr>
<tr>
<td>Boundry Condition of Displacement (Y) = 0 at Depth = -50.0025 in.</td>
<td>DEFLECTION (in) of Pile from Numerical Integration of Slope of Pile (rad)</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>$Y_{1} = 0$</td>
<td>$Y_{1} = Y_{1} + \frac{S_{1}}{2} \text{ depth}<em>{1} - \text{ depth}</em>{1-1}$</td>
</tr>
<tr>
<td>$Y_{2} = 0$</td>
<td>$Y_{2} = Y_{2} + \frac{S_{2}}{2} \text{ depth}<em>{2} - \text{ depth}</em>{2-1}$</td>
</tr>
<tr>
<td>$Y_{3} = 0$</td>
<td>$Y_{3} = Y_{3} + \frac{S_{3}}{2} \text{ depth}<em>{3} - \text{ depth}</em>{3-1}$</td>
</tr>
<tr>
<td>$Y_{4} = 0$</td>
<td>$Y_{4} = Y_{4} + \frac{S_{4}}{2} \text{ depth}<em>{4} - \text{ depth}</em>{4-1}$</td>
</tr>
<tr>
<td>$Y_{5} = 0$</td>
<td>$Y_{5} = Y_{5} + \frac{S_{5}}{2} \text{ depth}<em>{5} - \text{ depth}</em>{5-1}$</td>
</tr>
<tr>
<td>$Y_{6} = 0$</td>
<td>$Y_{6} = Y_{6} + \frac{S_{6}}{2} \text{ depth}<em>{6} - \text{ depth}</em>{6-1}$</td>
</tr>
<tr>
<td>$Y_{7} = 0$</td>
<td>$Y_{7} = Y_{7} + \frac{S_{7}}{2} \text{ depth}<em>{7} - \text{ depth}</em>{7-1}$</td>
</tr>
<tr>
<td>$Y_{8} = 0$</td>
<td>$Y_{8} = Y_{8} + \frac{S_{8}}{2} \text{ depth}<em>{8} - \text{ depth}</em>{8-1}$</td>
</tr>
<tr>
<td>$Y_{9} = 0$</td>
<td>$Y_{9} = Y_{9} + \frac{S_{9}}{2} \text{ depth}<em>{9} - \text{ depth}</em>{9-1}$</td>
</tr>
<tr>
<td>$Y_{10} = 0$</td>
<td>$Y_{10} = Y_{10} + \frac{S_{10}}{2} \text{ depth}<em>{10} - \text{ depth}</em>{10-1}$</td>
</tr>
<tr>
<td>$Y_{11} = 0$</td>
<td>$Y_{11} = Y_{11} + \frac{S_{11}}{2} \text{ depth}<em>{11} - \text{ depth}</em>{11-1}$</td>
</tr>
</tbody>
</table>
Define Pile Slope Vectors
(Rad)

SL1 = S1 5600 S2 5600 S3 5600 S4 5600 S5 5600 S6 5600 S7 5600 S8 5600 S9 5600 S10 5600 S11 5600
SL2 = S1 5269 S2 5269 S3 5269 S4 5269 S5 5269 S6 5269 S7 5269 S8 5269 S9 5269 S10 5269 S11 5269
SL3 = S1 5000 S2 5000 S3 5000 S4 5000 S5 5000 S6 5000 S7 5000 S8 5000 S9 5000 S10 5000 S11 5000
SL4 = S1 4294 S2 4294 S3 4294 S4 4294 S5 4294 S6 4294 S7 4294 S8 4294 S9 4294 S10 4294 S11 4294
SL5 = S1 3919 S2 3919 S3 3919 S4 3919 S5 3919 S6 3919 S7 3919 S8 3919 S9 3919 S10 3919 S11 3919
SL6 = S1 3544 S2 3544 S3 3544 S4 3544 S5 3544 S6 3544 S7 3544 S8 3544 S9 3544 S10 3544 S11 3544
SL7 = S1 3169 S2 3169 S3 3169 S4 3169 S5 3169 S6 3169 S7 3169 S8 3169 S9 3169 S10 3169 S11 3169
SL8 = S1 2794 S2 2794 S3 2794 S4 2794 S5 2794 S6 2794 S7 2794 S8 2794 S9 2794 S10 2794 S11 2794
SL9 = S1 2419 S2 2419 S3 2419 S4 2419 S5 2419 S6 2419 S7 2419 S8 2419 S9 2419 S10 2419 S11 2419
SL10 = S1 2044 S2 2044 S3 2044 S4 2044 S5 2044 S6 2044 S7 2044 S8 2044 S9 2044 S10 2044 S11 2044
SL11 = S1 169 S2 169 S3 169 S4 169 S5 169 S6 169 S7 169 S8 169 S9 169 S10 169 S11 169

Define Pile Displacement Vectors
(Inch)

YL1 = Y1 5600 Y2 5600 Y3 5600 Y4 5600 Y5 5600 Y6 5600 Y7 5600 Y8 5600 Y9 5600 Y10 5600 Y11 5600
YL2 = Y1 5269 Y2 5269 Y3 5269 Y4 5269 Y5 5269 Y6 5269 Y7 5269 Y8 5269 Y9 5269 Y10 5269 Y11 5269
YL3 = Y1 5000 Y2 5000 Y3 5000 Y4 5000 Y5 5000 Y6 5000 Y7 5000 Y8 5000 Y9 5000 Y10 5000 Y11 5000
YL4 = Y1 4294 Y2 4294 Y3 4294 Y4 4294 Y5 4294 Y6 4294 Y7 4294 Y8 4294 Y9 4294 Y10 4294 Y11 4294
YL5 = Y1 3919 Y2 3919 Y3 3919 Y4 3919 Y5 3919 Y6 3919 Y7 3919 Y8 3919 Y9 3919 Y10 3919 Y11 3919
YL6 = Y1 3544 Y2 3544 Y3 3544 Y4 3544 Y5 3544 Y6 3544 Y7 3544 Y8 3544 Y9 3544 Y10 3544 Y11 3544
YL7 = Y1 3169 Y2 3169 Y3 3169 Y4 3169 Y5 3169 Y6 3169 Y7 3169 Y8 3169 Y9 3169 Y10 3169 Y11 3169
YL8 = Y1 2794 Y2 2794 Y3 2794 Y4 2794 Y5 2794 Y6 2794 Y7 2794 Y8 2794 Y9 2794 Y10 2794 Y11 2794
YL9 = Y1 2419 Y2 2419 Y3 2419 Y4 2419 Y5 2419 Y6 2419 Y7 2419 Y8 2419 Y9 2419 Y10 2419 Y11 2419
YL10 = Y1 2044 Y2 2044 Y3 2044 Y4 2044 Y5 2044 Y6 2044 Y7 2044 Y8 2044 Y9 2044 Y10 2044 Y11 2044
YL11 = Y1 169 Y2 169 Y3 169 Y4 169 Y5 169 Y6 169 Y7 169 Y8 169 Y9 169 Y10 169 Y11 169

Define Pile Displacement Vectors
(Inch)
Identify Gage Depth Levels from Numerical Integration Index’s

\[
\begin{align*}
&\text{depth}_{5600} = 5.9975 \\
&\text{depth}_{5269} = 2.6875 \\
&\text{depth}_{5000} = 0 \\
&\text{depth}_{4294} = -7.0625 \\
&\text{depth}_{3919} = -10.8125 \\
&\text{depth}_{3544} = -14.5625 \\
&\text{depth}_{3169} = -18.3125 \\
&\text{depth}_{2794} = -22.0625 \\
&\text{depth}_{2419} = -25.8125 \\
&\text{depth}_{2044} = -29.5625 \\
&\text{depth}_{1294} = -47.0625 \\
&\text{depth}_{919} = -40.8125 \\
&\text{depth}_{544} = -44.5625 \\
&\text{depth}_{169} = -48.3125
\end{align*}
\]

Build Displacement Vectors for P-Y Curves

\[
\begin{align*}
&YD_0 = YD_1 = YD_2 = YD_3 = YD_4 = YD_5 = YD_6 = YD_7 = YD_8 = YD_9 = YD_{10} = YD_{11} = YD_{12} = YD_{13} =
\end{align*}
\]
## Build Load Vectors for P-Y Curves

<table>
<thead>
<tr>
<th>$PD_0_j$</th>
<th>$PD_1_j$</th>
<th>$PD_2_j$</th>
<th>$PD_3_j$</th>
<th>$PD_4_j$</th>
<th>$PD_5_j$</th>
<th>$PD_6_j$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth 0</td>
<td>Depth 0</td>
<td>Depth 0</td>
<td>Depth 0</td>
<td>Depth 0</td>
<td>Depth 0</td>
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</table>

<table>
<thead>
<tr>
<th>$PD_7_j$</th>
<th>$PD_8_j$</th>
<th>$PD_9_j$</th>
<th>$PD_{10}_j$</th>
<th>$PD_{11}_j$</th>
<th>$PD_{12}_j$</th>
<th>$PD_{13}_j$</th>
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</thead>
<tbody>
<tr>
<td>Depth 0</td>
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## Test #1: Pile Displacement Vectors
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Note: The content above represents tables of data for load vectors in P-Y curves for different tests. Each column and row in the tables correspond to specific data points representing various points on the P-Y curves.
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**Displacement Vectors for P-Y Curves**

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Test 11

Displacement Vector s

for

P - Y

Curves

1. 0.0046 -0.003 -0.0006

0.0021 0.0025 0.0018 0.0006

0.0001

1. -0.0017 -0.0072 -0.0018 -0.0009

0.0002 0.0005

1. 0.0018 -0.0009

0.0001

224
Test #1: Load 1 = 41.6008 lb

Moment vs. Depth (Floating Curve Fit)

- Data Points (Used for Curve Fitting)
- Top of Pile (Used for Curve Fitting)
- Moment Linear Intercept (Used for Curve Fitting)
- Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 6.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -33.3125 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)

Figure C1. Test #1 moment, shear, load, slope, and displacements.
Test #1: Load 1 = 41.6008 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #1: Load 1 = 41.6008 lb

Moment vs. Depth (Cubic Spline)

- Data Points (Used for Cubic Spline)
- Top of Pile (Used for Cubic Spline)
- Moment Linear Intercept (Used for Cubic Spline)
- Erroneous Data Points (Not Used for Cubic Spline)
- Cubic Spline of Moment Data

Pile Slope vs. Depth

- Pile Slope from Numerical Integration of Moment Spline
- Pile Slope at Data Points
- B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- Pile Displacement from Numerical Integration of Pile Slope
- Pile Displacement at Data Points
- B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #1: Load 2 = 74,2564 lb

Moment vs. Depth (Floating Curve Fit)

Data Points (Used for Curve Fitting)

Top of Pile (Used for Curve Fitting)

Moment Linear Intercept (Used for Curve Fitting)

Erroneous Data Points (Not Used for Curve Fitting)

- Used for Depth = 0.0 in (Linear Fit)
- Used for Depth = -7.0625 in (Cubic Fit)
- Used for Depth = -10.8125 in (Cubic Fit)
- Used for Depth = -14.5625 in (Cubic Fit)
- Used for Depth = -18.3125 in (Cubic Fit)
- Used for Depth = -22.0625 in (Cubic Fit)
- Used for Depth = -25.8125 in (Cubic Fit)
- Used for Depth = -29.5625 in (Cubic Fit)
- Used for Depth = -37.6625 in (Cubic Fit)
- Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #1: Load 2 = 74.2564 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #1: Load 2 = 74.2564 lb

Moment vs. Depth (Cubic Spline)

- Data Points (Used for Cubic Spline)
- Top of Pile (Used for Cubic Spline)
- Moment Linear Intercept (Used for Cubic Spline)
- Erroneous Data Points (Not Used for Cubic Spline)
- Cubic Spline of Moment Data

Pile Slope vs. Depth

- Pile Slope from Numerical Integration of Moment Spline
- Pile Slope at Data Points
- B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- Pile Displacement from Numerical Integration of Pile Slope
- Pile Displacement at Data Points
- B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #1: Load 3 = 114.2038 lb

Moment vs. Depth (Floating Curve Fit)

- Data Points (Used for Curve Fitting)
- Top of Pile (Used for Curve Fitting)
- Moment Linear Intercepts (Used for Curve Fitting)
- Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 6.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -37.6625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #1:  Load 3 = 114.2038 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #1: Load 3 = 114.2038 lb

Moment vs. Depth (Cubic Spline)

<table>
<thead>
<tr>
<th>Xxxx</th>
<th>Data Points (Used for Cubic Spline)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Xxxx</td>
<td>Top of Pile (Used for Cubic Spline)</td>
</tr>
<tr>
<td>Xxxx</td>
<td>Moment Linear Intercept (Used for Cubic Spline)</td>
</tr>
<tr>
<td>Xxxx</td>
<td>Erroneous Data Points (Not Used for Cubic Spline)</td>
</tr>
<tr>
<td>----</td>
<td>Cubic Spline of Moment Data</td>
</tr>
</tbody>
</table>

Pile Slope vs Depth

<table>
<thead>
<tr>
<th>Xxxx</th>
<th>Pile Slope from Numerical Integration of Moment Spline</th>
</tr>
</thead>
<tbody>
<tr>
<td>Xxxx</td>
<td>Pile Slope at Data Points</td>
</tr>
<tr>
<td>Xxxx</td>
<td>B.C. of Slope = 0 (Assumed at Pile Toe)</td>
</tr>
</tbody>
</table>

Pile Lateral Displacement vs Depth

<table>
<thead>
<tr>
<th>Xxxx</th>
<th>Pile Displacement from Numerical Integration of Pile Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>Xxxx</td>
<td>Pile Displacement at Data Points</td>
</tr>
<tr>
<td>Xxxx</td>
<td>B.C. of Displacement = 0 (Assumed at Pile Toe)</td>
</tr>
</tbody>
</table>
Test #1: Load 4 = 146.8594 lb

Moment vs Depth (Floating Curve Fit)

- ○○○ Data Points (Used for Curve Fitting)
- ◊◊◊ Top of Pile (Used for Curve Fitting)
- □□□ Moment Linear Intercept (Used for Curve Fitting)
- ××× Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 0.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -33.3125 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #1: Load 4 = 146.8594 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #1: Load 4 = 146.8594 lb

Moment vs. Depth (Cubic Spline)

- ○○○ Data Points (Used for Cubic Spline)
- ○○○ Top of Pile (Used for Cubic Spline)
- □□□ Moment Linear Intercept (Used for Cubic Spline)
- ××× Erroneous Data Points (Not Used for Cubic Spline)
- ― Cubic Spline of Moment Data

Pile Slope vs. Depth

- — Pile Slope from Numerical Integration of Moment Spline
- ○○○ Pile Slope at Data Points
- □□□ B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- — Pile Displacement from Numerical Integration of Pile Slope
- ○○○ Pile Displacement at Data Points
- □□□ B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #1: Load 5 = 179.5316 lb

Moment vs. Depth (Floating Curve Fit)

Data Points (Used for Curve Fitting)
- Used for Depth = 0.0614 to 0.0 in (Linear Fit)
- Used for Depth = -7.6625 in (Cubic Fit)
- Used for Depth = -10.8125 in (Cubic Fit)
- Used for Depth = -14.6625 in (Cubic Fit)
- Used for Depth = -18.3125 in (Cubic Fit)
- Used for Depth = -22.1625 in (Cubic Fit)
- Used for depth = -25.8125 in (Cubic Fit)
- Used for Depth = -29.5625 in (Cubic Fit)
- Used for Depth = -37.0625 in (Cubic Fit)
- Used for Depth = -48.3125 in (Cubic Fit)

Top of Pile (Used for Curve Fitting)
- Used for Depth = -6.0614 to 0.0 in (Linear Fit)

Moment Linear Intercept (Used for Curve Fitting)
- Used for Depth = -7.6625 in (Cubic Fit)
- Used for Depth = -10.8125 in (Cubic Fit)
- Used for Depth = -14.6625 in (Cubic Fit)
- Used for Depth = -18.3125 in (Cubic Fit)
- Used for Depth = -22.1625 in (Cubic Fit)
- Used for Depth = -25.8125 in (Cubic Fit)
- Used for Depth = -29.5625 in (Cubic Fit)
- Used for Depth = -37.0625 in (Cubic Fit)
- Used for Depth = -48.3125 in (Cubic Fit)
Test #1: Load 5 = 179.5316 lb

Shear vs. Depth

Depth (in)

Net Soil Load vs. Depth
Test #1: Load 5 = 179.5316 lb

Moment vs. Depth (Cubic Spline)

Depth (in)

Moment (in * lbs)

Data Points (Used for Cubic Spline)
Top of Pile (Used for Cubic Spline)
Moment Linear Intercept (Used for Cubic Spline)
Erroneous Data Points (Not Used for Cubic Spline)
Cubic Spline of Moment Data

Pile Slope vs. Depth

Depth (in)
Pile Slope (rad)

Pile Slope from Numerical Integration of Moment Spline
Pile Slope at Data Points
B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

Depth (in)
Pile Displacement (in)

Pile Displacement from Numerical Integration of Pile Slope
Pile Displacement at Data Points
B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #1: Load 6 = 223,094.5 lb

Moment vs. Depth (Floating Curve Fit)

- Data Points (Used for Curve Fitting)
- Top of Pile (Used for Curve Fitting)
- Moment Linear Intercept (Used for Curve Fitting)
- Erroooous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 6.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #1: Load 6 = 223.0945 lb

Shear vs. Depth

Net Soil Load vs. Depth

Depth (in)

Shear (kip)

Net Soil Load (kip/in)

Depth (in)
Test #1: Load 6 = 223.0945 lb

Moment vs. Depth (Cubic Spline)

- Data Points (Used for Cubic Spline)
- Top of Pile (Used for Cubic Spline)
- Moment Linear Intercept (Used for Cubic Spline)
- Erroneous Data Points (Not Used for Cubic Spline)
- Cubic Spline of Moment Data

Pile Slope vs. Depth

- Pile Slope from Numerical Integration of Moment Spline
- Pile Slope at Data Points
- B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- Pile Displacement from Numerical Integration of Pile Slope
- Pile Displacement at Data Points
- B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #1:  Load 7 = 255.7667 lb

Moment vs. Depth (Floating Curve Fit)

○○○ Data Points (Used for Curve Fitting)
○○ Top of Pile (Used for Curve Fitting)
● Moment Linear Intercept (Used for Curve Fitting)
××× Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 6.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #1: Load 7 = 256.7667 lb

Shear vs. Depth

Net Soil Load vs. Depth

Depth (in)

Net Soil Load (kips / in)

Depth (in)
Test #1: Load 7 = 255.7667 lb

Moment vs. Depth (Cubic Spline)

```

Data Points (Used for Cubic Spline)
Top of Pile (Used for Cubic Spline)
Moment Linear Intercept (Used for Cubic Spline)
Erroneous Data Points (Not Used for Cubic Spline)
Cubic Spline of Moment Data

Pile Slope vs Depth

Pile Lateral Displacement vs Depth

Pile Slope from Numerical Integration of Moment Spline
Pile Slope at Data Points
B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Displacement from Numerical Integration of Pile Slope
Pile Displacement at Data Points
B.C. of Displacement = 0 (Assumed at Pile Toe)
```
Test #1: Load $8 = 266.6574$ lb

Moment vs. Depth (Floating Curve Fit)

- ○○○ Data Points (Used for Curve Fitting)
- ●●● Top of Pile (Used for Curve Fitting)
- ××× Moment Linear Intercept (Used for Curve Fitting)
- ◯◯◯ Erroneous Data Points (Not Used for Curve Fitting)
- Used for Depth = 6.0614 to 0.0 in (Linear Fit)
- Used for Depth = 7.0625 in (Cubic Fit)
- Used for Depth = 10.8125 in (Cubic Fit)
- Used for Depth = 14.5625 in (Cubic Fit)
- Used for Depth = 18.3125 in (Cubic Fit)
- Used for Depth = 22.0625 in (Cubic Fit)
- Used for depth = 25.8125 in (Cubic Fit)
- Used for Depth = 29.5625 in (Cubic Fit)
- Used for Depth = 37.0625 in (Cubic Fit)
- Used for Depth = 40.8125 to -48.3125 in (Cubic Fit)
Test #1: Load 8 = 266.6574 lb

Shear vs. Depth

Net Soil Load vs. Depth

Depth (in)
Test #1: Load $8 = 266.6574$ lb

Moment vs. Depth (Cubic Spline)

- ○○○ Data Points (Used for Cubic Spline)
- ●●● Top of Pile (Used for Cubic Spline)
- ▲ Moment Linear Intercept (Used for Cubic Spline)
- ✗✗✗ Erroneous Data Points (Not Used for Cubic Spline)
- <- Cubic Spline of Moment Data

Pile Slope vs. Depth

- Pile Slope from Numerical Integration of Moment Spline
- ●●● Pile Slope at Data Points
- ◆◆◆ B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- Pile Displacement from Numerical Integration of Pile Slope
- ●●● Pile Displacement at Data Points
- ◆◆◆ B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #1: Load 9 = 190.4223 lb

Moment vs Depth (Floating Curve Fit)

Data Points (Used for Curve Fitting)
- Top of Pile (Used for Curve Fitting)
- Moment Linear Intercept (Used for Curve Fitting)
- Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 6.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.3625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.3625 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #1: Load 9 = 190.4223 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #1: Load 9 = 190.4223 lb

Moment vs. Depth (Cubic Spline)

Depth (in)

Moment (in. * lbs)

Data Points (Used for Cubic Spline)
Top of Pile (Used for Cubic Spline)
Moment Linear Intercept (Used for Cubic Spline)
Errorous Data Points (Not Used for Cubic Spline)
Cubic Spline of Moment Data

Pile Slope vs. Depth

Pile Slope (rad)

Depth (in)

Pile Slope from Numerical Integration of Moment Spline
Pile Slope at Data Points
B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

Pile Displacement (in.)

Depth (in)

Pile Displacement from Numerical Integration of Pile Slope
Pile Displacement at Data Points
B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #1: Load 10 = 126.0890 lb

Moment vs. Depth (Floating Curve Fit)

- Data Points (Used for Curve Fitting)
- Top of Pile (Used for Curve Fitting)
- Moment Linear Intercept (Used for Curve Fitting)
- Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 0.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -33.3125 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #1: Load 10 = 125.0890 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #1: Load 10 = 125.0890 lb

Moment vs Depth (Cubic Spline)

- Data Points (Used for Cubic Spline)
- Top of Pile (Used for Cubic Spline)
- Moment Linear Intercept (Used for Cubic Spline)
- Erroneous Data Points (Not Used for Cubic Spline)
- Cubic Spline of Moment Data

Pile Slope vs. Depth

- Pile Slope from Numerical Integration of Moment Spline
- Pile Slope at Data Points
- B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- Pile Displacement from Numerical Integration of Pile Slope
- Pile Displacement at Data Points
- B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #1: Load 11 = 63.3712 lb

Moment vs. Depth (Floating Curve Fit)

- Data Points (Used for Curve Fitting)
- Top of Pile (Used for Curve Fitting)
- Moment Linear Intercept (Used for Curve Fitting)
- Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 0.0614 to 0.0 in (Linear Fit)
  - Used for Depth = 7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.3625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #1: Load 11 = 63.3712 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #1: Load 11 = 63.3712 lb

Moment vs. Depth (Cubic Spline)

- ○○○ Data Points (Used for Cubic Spline)
- ---- Top of Pile (Used for Cubic Spline)
- □□□ Moment Linear Intercept (Used for Cubic Spline)
- ××× Erroneous Data Points (Not Used for Cubic Spline)
- — Cubic Spline of Moment Data

Pile Slope vs. Depth

- — Pile Slope from Numerical Integration of Moment Spline
- ○○○ Pile Slope at Data Points
- ○○○ B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- — Pile Displacement from Numerical Integration of Pile Slope
- ○○○ Pile Displacement at Data Points
- ○○○ B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #1: P-Y Curves

Figure C2. Test #1 p-y curves.
Test #1: P - Y Curves for Locations of Large, Single Direction Displacements

P - Y Curves: Test 1

Y (in)

- Depth = -7.0625 in
- Depth = -10.8125 in
- Depth = -14.5625 in
Test #1: P - Y Curves for Locations with Small Displacements, and Small Stress Reversals

P - Y Curve’s: Test 1

Depth = -18.3125 in
Depth = -22.9025 in
Test #1: P - Y Curves for Locations with Small Displacements, and Large Stress Reversals

P - Y Curves: Test 1

- Depth = -25.8125 in
- Depth = -29.5625 in
- Depth = -37.0625 in
- Depth = -40.8125 in
- Depth = -44.5625 in
- Depth = -48.3125 in

Y (in)

P (kips/in)
Test #2: Load 1 = 41.6008 lb

Moment vs. Depth (Floating Curve Fit)

Data Points (Used for Curve Fitting)
Top of Pile (Used for Curve Fitting)
Moment Linear Intercept (Used for Curve Fitting)
Error Data Points (Not Used for Curve Fitting)
- Used for Depth = 0.0614 to 0.0 in (Linear Fit)
- Used for Depth = -0.0625 in (Cubic Fit)
- Used for Depth = -10.8125 in (Cubic Fit)
- Used for Depth = -14.5625 in (Cubic Fit)
- Used for Depth = -18.3125 in (Cubic Fit)
- Used for Depth = -22.0625 in (Cubic Fit)
- Used for depth = -25.8125 in (Cubic Fit)
- Used for Depth = -29.5625 in (Cubic Fit)
- Used for Depth = -37.0625 in (Cubic Fit)
- Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)

Figure C3. Test #2 moment, shear, load, slope, and displacements.
Test #2: Load 1 = 41.6008 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #2: Load 1 = 41.6008 lb

Moment vs. Depth (Cubic Spline)

- Data Points (Used for Cubic Spline)
- Top of Pile (Used for Cubic Spline)
- Moment Linear Intercept (Used for Cubic Spline)
- Erroneous Data Points (Not Used for Cubic Spline)
- Cubic Spline of Moment Data

Pile Slope vs. Depth

- Pile Slope from Numerical Integration of Moment Spline
- Pile Slope at Data Points
- B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- Pile Displacement from Numerical Integration of Pile Slope
- Pile Displacement at Data Points
- B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #2: Load 2 = 74.2564 lb

Moment vs. Depth (Floating Curve Fit)

- Data Points (Used for Curve Fitting)
- Top of Pile (Used for Curve Fitting)
- Moment Linear Intercept (Used for Curve Fitting)
- Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 6.6614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0023 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #2: Load 2 = 74.2564 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #2: Load 2 = 74.2564 lb

Moment vs. Depth (Cubic Spline)

Depth (in)

Data Points (Used for Cubic Spline)
Top of Pile (Used for Cubic Spline)
Moment Linear Intercep (Used for Cubic Spline)
Erroneous Data Points (Not Used for Cubic Spline)
Cubic Spline of Moment Data

Pile Slope vs. Depth

Depth (in)
Pile Slope from Numerical Integration of Moment Spline
Pile slope at Data Points
B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs Depth

Depth (in)
Pile Displacement from Numerical Integration of Pile Slope
Pile Displacement at Data Points
B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #2: Load 3 = 114.2038 lb

Moment vs. Depth (Floating Curve Fit)

- ○○○ Data Points (Used for Curve Fitting)
- □□□ Top of Pile (Used for Curve Fitting)
- △△△ Moment Linear Intercept (Used for Curve Fitting)
- ××× Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 6.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #2: Load 3 = 114.2038 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #2: Load 3 = 114.2038 lb

Moment vs. Depth (Cubic Spline)

- Data Points (Used for Cubic Spline)
- Top of Pile (Used for Cubic Spline)
- Moment Linear Intercept (Used for Cubic Spline)
- Errorneous Data Points (Not Used for Cubic Spline)
- Cubic Spline of Moment Data

Pile Slope vs. Depth

- Pile Slope from Numerical Integration of Moment Spline
- Pile Slope at Data Points
- B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- Pile Displacement from Numerical Integration of Pile Slope
- Pile Displacement at Data Points
- B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #2: Load 4 = 146.8594 lb

Moment vs. Depth (Floating Curve Fit)

- ○○○ Data Points (Used for Curve Fitting)
- □□□ Top of Pile (Used for Curve Fitting)
- ❌❌❌ Moment Linear Intercept (Used for Curve Fitting)
- ××× Erroneous Data Points (Not Used for Curve Fitting)
- Used for Depth = 6.0614 to 0.0 in (Linear Fit)
- Used for Depth = -7.6625 in (Cubic Fit)
- Used for Depth = -10.8125 in (Cubic Fit)
- Used for Depth = -14.5625 in (Cubic Fit)
- Used for Depth = -18.3125 in (Cubic Fit)
- Used for Depth = -22.0625 in (Cubic Fit)
- Used for depth = -25.8125 in (Cubic Fit)
- Used for Depth = -29.5625 in (Cubic Fit)
- Used for Depth = -37.0625 in (Cubic Fit)
- Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #2: Load 4 = 146.8594 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #2: Load 4 = 146.8594 lb

Moment vs. Depth (Cubic Spline)

- ○○○ Data Points (Used for Cubic Spline)
- □□□ Top of Pile (Used for Cubic Spline)
- ■■■ Moment Linear Intercepts (Used for Cubic Spline)
- ××× Erroneous Data Points (Not Used for Cubic Spline)
- — Cubic Spline of Moment Data

Pile Slope vs. Depth

- — Pile Slope from Numerical Integration of Moment Spline
- ○○○ Pile Slope at Data Points
- ◯◯◯ B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- — Pile Displacement from Numerical Integration of Pile Slope
- ○○○ Pile Displacement at Data Points
- ◯◯◯ B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #2: Load 5 = 179.5316 lb

Moment vs. Depth (Floating Curve Fit)

- Data Points (Used for Curve Fitting)
- Top of Pile (Used for Curve Fitting)
- Moment Linear Intercept (Used for Curve Fitting)
- Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 6.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -44.3125 in (Cubic Fit)
Test #2: Load 5 = 179.5316 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #2: Load 5 = 179.5316 lb

Moment vs. Depth (Cubic Spline)

- Data Points (Used for Cubic Spline)
- Top of Pile (Used for Cubic Spline)
- Moment Linear Intercept (Used for Cubic Spline)
- Erroneous Data Points (Not Used for Cubic Spline)
- Cubic Spline of Moment Data

Pile Slope vs. Depth

- Pile Slope from Numerical Integration of Moment Spline
- Pile Slope at Data Points
- B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- Pile Displacement from Numerical Integration of Pile Slope
- Pile Displacement at Data Points
- B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #2: Load 6 = 223.0945 lb

Moment vs. Depth (Floating Curve Fit)

Data Points (Used for Curve Fitting)
- Top of Pile (Used for Curve Fitting)
- Moment Linear Intercept (Used for Curve Fitting)
- Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 0.014 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #2: Load 6 = 223.0945 lb

Shear vs. Depth

Net Soil Load vs. Depth

Depth (in)
Test #2: Load 6 = 223.0945 lb

Moment vs. Depth (Cubic Spline)

- Data Points (Used for Cubic Spline)
- Top of Pile (Used for Cubic Spline)
- Moment Linear Intercep (Used for Cubic Spline)
- Erroneous Data Points (Not Used for Cubic Spline)
- Cubic Spline of Moment Data

Pile Slope vs. Depth

- Pile Slope from Numerical Integration of Moment Spline
- Pile Slope at Data Points
- B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- Pile Displacement from Numerical Integration of Pile Slope
- Pile Displacement at Data Points
- B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #2: Load 7 = 255.7667 lb

Moment vs. Depth (Floating Curve Fit)

- Data Points (Used for Curve Fitting)
- Top of Pile (Used for Curve Fitting)
- Moment Linear Intercept (Used for Curve Fitting)
- Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 6.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0025 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #2: Load 7 = 255.7667 lb

Shear vs. Depth

Depth (in)

Net Soil Load vs. Depth

Depth (in)
Test #2: Load 7 = 255.7667 lb

Moment vs. Depth (Cubic Spline)

- Data Points (Used for Cubic Spline)
- Top of Pile (Used for Cubic Spline)
- Moment Linear Intercept (Used for Cubic Spline)
- Erroneous Data Points (Not Used for Cubic Spline)
- Cubic Spline of Moment Data

Pile Slope vs. Depth

- Pile Slope from Numerical Integration of Moment Spline
- Pile Slope at Data Points
- B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- Pile Displacement from Numerical Integration of Pile Slope
- Pile Displacement at Data Points
- B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #2: Load $8 = 266.6574$ lb

Moment vs. Depth (Floating Curve Fit)

- Data Points (Used for Curve Fitting)
- Top of Pile (Used for Curve Fitting)
- Moment Linear Intercept (Used for Curve Fitting)
- Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 6.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -33.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #2: Load 8 = 266.6574 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #2: Load 8 = 266.6574 lb

Moment vs. Depth (Cubic Spline)

Data Points (Used for Cubic Spline)
Top of Pile (Used for Cubic Spline)
Moment Linear Intercepts (Used for Cubic Spline)
Erroneous Data Points (Not Used for Cubic Spline)
Cubic Spline of Moment Data

Pile Slope vs. Depth

Pile Slope from Numerical Integration of Moment Spline
Pile Slope at Data Points
B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

Pile Displacement from Numerical Integration of Pile Slope
Pile Displacement at Data Points
B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #2: Load 9 = 190.4223 lb

Moment vs. Depth (Floating Curve Fit)

- ○○○ Data Points (Used for Curve Fitting)
- □□□ Top of Pile (Used for Curve Fitting)
- III Moment Linear Intercept (Used for Curve Fitting)
- XXX Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 6.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #2: Load 9 = 190.4223 lb

Shear vs. Depth

Depth (in)

Net Soil Load vs. Depth

Net Soil Load (kips/in)

Depth (in)
Test #2: Load 9 = 190.4223 lb

Moment vs. Depth (Cubic Spline)

- Data Points (Used for Cubic Spline)
- Top of Pile (Used for Cubic Spline)
- Moment Linear Intercept (Used for Cubic Spline)
- Erroneous Data Points (Not Used for Cubic Spline)
- Cubic Spline of Moment Data

Pile Slope vs. Depth

- Pile Slope from Numerical Integration of Moment Spline
- Pile Slope at Data Points
- B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- Pile Displacement from Numerical Integration of Pile Slope
- Pile Displacement at Data Points
- B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #2: Load 10 = 125.0890 lb

Moment vs. Depth (Floating Curve Fit)

- ○○ Data Points (Used for Curve Fitting)
- ▲ Top of Pile (Used for Curve Fitting)
- >>> Moment Linear Intercept (Used for Curve Fitting)
- ××× Erroneous Data Points (Not Used for Curve Fitting)
- Used for Depth = 6.0614 to 0.0 in (Linear Fit)
- Used for Depth = -7.0625 in (Cubic Fit)
- Used for Depth = -10.8125 in (Cubic Fit)
- Used for Depth = -14.5625 in (Cubic Fit)
- Used for Depth = -18.3125 in (Cubic Fit)
- Used for Depth = -22.0625 in (Cubic Fit)
- Used for depth = -25.8125 in (Cubic Fit)
- Used for Depth = -29.5625 in (Cubic Fit)
- Used for Depth = -37.0625 in (Cubic Fit)
- Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #2: Load 10 = 125.0890 lb

Shear vs. Depth

Net Soil Load vs. Depth

Depth (in)
Test #2: Load 10 = 125.0890 lb

Moment vs Depth (Cubic Spline)

Moment vs Depth (Cubic Spline)

Moment (in * lb)

Depth (in)

- Data Points (Used for Cubic Spline)
- Top of Pile (Used for Cubic Spline)
- Moment Linear Intercept (Used for Cubic Spline)
- Erroneous Data Points (Not Used for Cubic Spline)
- Cubic Spline of Moment Data

Pile Slope vs Depth

Depth (in)

- Pile Slope from Numerical Integration of Moment Spline
- Pile Slope at Data Points
- B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

Depth (in)

- Pile Displacement from Numerical Integration of Pile Slope
- Pile Displacement at Data Points
- B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #2:  Load 11 = 63.3712 lb

Moment vs. Depth (Floating Curve Fit)

Data Points (Used for Curve Fitting)

Top of Pile (Used for Curve Fitting)

Moment Linear Intercept (Used for Curve Fitting)

Erroneous Data Points (Not Used for Curve Fitting)

- Used for Depth = 6.0614 to 0.0 in (Linear Fit)
- Used for Depth = 7.0625 in (Cubic Fit)
- Used for Depth = 10.8125 in (Cubic Fit)
- Used for Depth = 14.5625 in (Cubic Fit)
- Used for Depth = 18.3125 in (Cubic Fit)
- Used for Depth = 22.0625 in (Cubic Fit)
- Used for depth = 25.8125 in (Cubic Fit)
- Used for Depth = 29.5625 in (Cubic Fit)
- Used for Depth = 33.3125 in (Cubic Fit)
- Used for Depth = 40.8125 to -48.3125 in (Cubic Fit)
Test #2: Load 11 = 63.3712 lb

Shear vs. Depth

Net Soil Load vs. Depth

Depth (in)
Test #2: Load 11 = 63.3712 lb

Moment vs. Depth (Cubic Spline)

- Data Points (Used for Cubic Spline)
- Top of Pile (Used for Cubic Spline)
- Moment Linear Intercept (Used for Cubic Spline)
- Erroneous Data Points (Not Used for Cubic Spline)
- Cubic Spline of Moment Data

Pile Slope vs. Depth

- Pile Slope from Numerical Integration of Moment Spline
- Pile Slope at Data Points
- B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- Pile Displacement from Numerical Integration of Pile Slope
- Pile Displacement at Data Points
- B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #2: P - Y Curves

P - Y Curves: Test 2

Figure C4. Test #2 p-y curves.
Test #2: P - Y Curves for Locations of Large, Single Direction Displacements

P - Y Curves: Test 2

Depth = -7.0625 in.
• Depth = -10.8125 in.
• Depth = -14.5625 in.
Test #2: P - Y Curves for Locations with Small Displacements, and Small Stress Reversals

P - Y Curves: Test 2

- Depth = 18.3125 in.
- Depth = 22.0625 in.
Test #2: P-Y Curves for Locations with Small Displacements, and Large Stress Reversals

P-Y Curves: Test 2

- Depth = -25.8125 in.
- Depth = -29.5625 in.
- Depth = -37.6625 in.
- Depth = -40.8125 in.
- Depth = -44.5625 in.
- Depth = -48.3125 in.
Test #3: Load 1 = 41.6008 lb

Moment vs. Depth (Floating Curve Fit)

Figure C5. Test #3 moment, shear, load, slope, and displacements.
Test #3: Load 1 = 41.8098 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #3: Load 1 = 41.6008 lb

Moment vs. Depth (Cubic Spline)

- ○○○ Data Points (Used for Cubic Spline)
- ○○○ Top of Pile (Used for Cubic Spline)
- □□□ Moment Linear Intercept (Used for Cubic Spline)
- ××× Erroneous Data Points (Not Used for Cubic Spline)
- — Cubic Spline of Moment Data

Pile Slope vs. Depth

- — Pile Slope from Numerical Integration of Moment Spline
- ○○○ Pile Slope at Data Points
- □□□ B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- — Pile Displacement from Numerical Integration of Pile Slope
- ○○○ Pile Displacement at Data Points
- □□□ B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #3: Load 2 = 74,2564 lb

Moment vs Depth (Floating Curve Fit)

Data Points (Used for Curve Fitting)
- Top of Pile (Used for Curve Fitting)
- Moment Linear Intercept (Used for Curve Fitting)
- Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 6.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #3: Load 2 = 74.2564 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #3: Load 2 = 74.2564 lb

Moment vs. Depth (Cubic Spline)

- ○○○ Data Points (Used for Cubic Spline)
- ○○ Top of Pile (Used for Cubic Spline)
- ■■■ Moment Linear Intercept (Used for Cubic Spline)
- ××× Erroneous Data Points (Not Used for Cubic Spline)
- - - Cubic Spline of Moment Data

Pile Slope vs. Depth

- - Pile Slope from Numerical Integration of Moment Spline
- ○○○ B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- - Pile Displacement from Numerical Integration of Pile Slope
- ○○○ B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #3: Load 3 = 114.2038 lb

Moment vs. Depth (Floating Curve Fit)

Data Points (Used for Curve Fitting)
Top of Pile (Used for Curve Fitting)
Moment Linear Intercept (Used for Curve Fitting)
Erroneous Data Points (Not Used for Curve Fitting)
- Used for Depth = 6.0614 to 0.0 in (Linear Fit)
- Used for Depth = -7.0625 in (Cubic Fit)
- Used for Depth = -10.8125 in (Cubic Fit)
- Used for Depth = -14.3625 in (Cubic Fit)
- Used for Depth = -18.3125 in (Cubic Fit)
- Used for Depth = -22.0625 in (Cubic Fit)
- Used for Depth = -25.8125 in (Cubic Fit)
- Used for Depth = -29.5625 in (Cubic Fit)
- Used for Depth = -32.3125 in (Cubic Fit)
- Used for Depth = -37.0625 in (Cubic Fit)
- Used for Depth = -40.8125 in - 48.3125 in (Cubic Fit)
Test #3: Load 3 = 114.2038 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #3: Load 3 = 114.2038 lb

Moment vs. Depth (Cubic Spline)

- ○○○ Data Points (Used for Cubic Spline)
- Top of Pile (Used for Cubic Spline)
- Moment Linear Intercept (Used for Cubic Spline)
- ××× Erroneous Data Points (Not Used for Cubic Spline)
- Cubic Spline of Moment Data

Pile Slope vs Depth

- Pile Slope from Numerical Integration of Moment Spline
- ○○○ Pile Slope at Data Points
- ××× B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs Depth

- Pile Displacement from Numerical Integration of Pile Slope
- ○○○ Pile Displacement at Data Points
- ××× B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #3: Load 4 = 146.8594 lb

Moment vs Depth (Floating Curve Fit)

- ○○○ Data Points (Used for Curve Fitting)
- □□□ Top of Pile (Used for Curve Fitting)
- □□□ Moment Linear Intercept (Used for Curve Fitting)
- ××× Erroneous Data Points (Not Used for Curve Fitting)
- Used for Depth = 6.0614 to 0.0 in (Linear Fit)
- Used for Depth = -7.0625 in (Cubic Fit)
- Used for Depth = -10.8125 in (Cubic Fit)
- Used for Depth = -14.5625 in (Cubic Fit)
- Used for Depth = -18.3125 in (Cubic Fit)
- Used for Depth = -22.0625 in (Cubic Fit)
- Used for depth = -25.8125 in (Cubic Fit)
- Used for Depth = -29.5625 in (Cubic Fit)
- Used for Depth = -33.3125 in (Cubic Fit)
- Used for Depth = -37.0625 in (Cubic Fit)
- Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
**Test #3:**  Load 4 = 146.8594 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #3: Load 4 = 146.8694 lb

Moment vs. Depth (Cubic Spline)

- ○○○ Data Points (Used for Cubic Spline)
- ○○○ Top of Pile (Used for Cubic Spline)
- ■■■ Moment Linear Intercept (Used for Cubic Spline)
- X X X Erroneous Data Points (Not Used for Cubic Spline)
- — Cubic Spline of Moment Data

Pile Slope vs. Depth

- — Pile Slope from Numerical Integration of Moment Spline
- ○○○ Pile Slope at Data Points
- ○○○ B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- — Pile Displacement from Numerical Integration of Pile Slope
- ○○○ Pile Displacement at Data Points
- ○○○ B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #3: Load 5 = 179.5316 lb

Moment vs. Depth (Floating Curve Fit)

- Data Points (Used for Curve Fitting)
- Top of Pile (Used for Curve Fitting)
- Moment Linear Intercept (Used for Curve Fitting)
- Error Data Points (Not Used for Curve Fitting)
  - Used for Depth = 6.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -12.0625 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #3: Load 5 = 179.5316 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #3: Load 5 = 179.5316 lb

Moment vs. Depth (Cubic Spline)

- Data Points (Used for Cubic Spline)
- Top of Pile (Used for Cubic Spline)
- Moment Linear Intercept (Used for Cubic Spline)
- Erroneous Data Points (Not Used for Cubic Spline)
- Cubic Spline of Moment Data

Pile Slope vs. Depth

- Pile Slope from Numerical Integration of Moment Spline
- B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- Pile Displacement from Numerical Integration of Pile Slope
- B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #3: Load 6 = 223.0945 lb

Moment vs. Depth (Floating Curve Fit)

- OOOO Data Points (Used for Curve Fitting)
- □□□ Top of Pile (Used for Curve Fitting)
- O Moment Linear Intercept (Used for Curve Fitting)
- X××X Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 6.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -32.3125 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -39.8125 to -48.3125 in (Cubic Fit)
Test #3: Load 6 = 223.0945 lb

Moment vs. Depth (Cubic Spline)

Data Points (Used for Cubic Spline)
Top of Pile (Used for Cubic Spline)
Moment Linear Intercept (Used for Cubic Spline)
Erroneous Data Points (Not Used for Cubic Spline)
Cubic Spline of Moment Data

Pile Slope vs. Depth

Pile Slope from Numerical Integration of Moment Spline
Pile Slope at Data Points
B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

Pile Displacement from Numerical Integration of Pile Slope
Pile Displacement at Data Points
B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #3: Load 7 = 255.7667 lb

Moment vs. Depth (Floating Curve Fit)

- ○○○ Data Points (Used for Curve Fitting)
- ○○○ Top of Pile (Used for Curve Fitting)
- □□□ Moment Linear Intercept (Used for Curve Fitting)
- ▢▢▢ Error Data Points (Not Used for Curve Fitting)
  - Used for Depth = 6.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -33.3125 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #3: Load 7 = 255.7667 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #3: Load 7 = 255.7667 lb

Moment vs. Depth (Cubic Spline)

Depth (in)

Pile Slope vs. Depth

Depth (in)

Pile Lateral Displacement vs. Depth

Depth (in)
Test #3: Load 8 = 266.6574 lb

Moment vs. Depth (Floating Curve Fit)

Data Points (Used for Curve Fitting)

Top of Pile (Used for Curve Fitting)

Moment Linear Intercept (Used for Curve Fitting)

Error Data Points (Not Used for Curve Fitting)

- Used for Depth = 6.0614 to 0.0 in (Linear Fit)
- Used for Depth = -7.0625 in (Cubic Fit)
- Used for Depth = -10.8125 in (Cubic Fit)
- Used for Depth = -14.5625 in (Cubic Fit)
- Used for Depth = -18.3125 in (Cubic Fit)
- Used for Depth = -22.0625 in (Cubic Fit)
- Used for Depth = -25.8125 in (Cubic Fit)
- Used for Depth = -29.5625 in (Cubic Fit)
- Used for Depth = -37.0625 in (Cubic Fit)
- Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #3: Load 8 = 266.6574 lb

Shear vs. Depth

Net Soil Load vs. Depth

Depth (in)
Test #3: Load 8 = 266.6574 lb

Moment vs. Depth (Cubic Spline)

- ○○○ Data Points (Used for Cubic Spline)
- □□□ Top of Pile (Used for Cubic Spline)
- ◯◯◯ Moment Linear Intercept (Used for Cubic Spline)
- ✗✗✗ Erroneous Data Points (Not Used for Cubic Spline)
- — Cubic Spline of Moment Data

Pile Slope vs. Depth

- — Pile Slope from Numerical Integration of Moment Spline
- ○○○ Pile Slope at Data Points
- ◯◯◯ B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- — Pile Displacement from Numerical Integration of Pile Slope
- ○○○ Pile Displacement at Data Points
- ◯◯◯ B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #3:  Load 9 = 190.4223 lb

Moment vs. Depth (Floating Curve Fit)

- Data Points (Used for Curve Fitting)
- Top of Pile (Used for Curve Fitting)
- Moment Linear Intercept (Used for Curve Fitting)
- Erroneous Data Points (Not Used for Curve Fitting)
  - Used for Depth = 6.0614 to 0.0 in (Linear Fit)
  - Used for Depth = -7.0625 in (Cubic Fit)
  - Used for Depth = -10.8125 in (Cubic Fit)
  - Used for Depth = -14.5625 in (Cubic Fit)
  - Used for Depth = -18.3125 in (Cubic Fit)
  - Used for Depth = -22.0625 in (Cubic Fit)
  - Used for Depth = -25.8125 in (Cubic Fit)
  - Used for Depth = -29.5625 in (Cubic Fit)
  - Used for Depth = -37.0625 in (Cubic Fit)
  - Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #3:  Load 9 = 190.4223 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #3: Load 9 = 190.4223 lb

Moment vs. Depth (Cubic Spline)

- ○○○ Data Points (Used for Cubic Spline)
- □□□ Top of Pile (Used for Cubic Spline)
- □□□ Moment Linear Intercept (Used for Cubic Spline)
- ××× Erroneous Data Points (Not Used for Cubic Spline)
- — Cubic Spline of Moment Data

Pile Slope vs. Depth

- — Pile Slope from Numerical Integration of Moment Spline
- ○○○ Pile Slope at Data Points
- □□□ B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- — Pile Displacement from Numerical Integration of Pile Slope
- ○○○ Pile Displacement at Data Points
- □□□ B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #3: Load 10 = 125.0890 lb

Moment vs. Depth (Floating Curve Fit)

- Data Points (Used for Curve Fitting)
- Top of Pile (Used for Curve Fitting)
- Moment Linear Intercept (Used for Curve Fitting)
- Error Data Points (Not Used for Curve Fitting)

- Used for Depth = 6.0614 to 0.0 in (Linear Fit)
- Used for Depth = -7.0625 in (Cubic Fit)
- Used for Depth = -10.8125 in (Cubic Fit)
- Used for Depth = -14.5625 in (Cubic Fit)
- Used for Depth = -18.3125 in (Cubic Fit)
- Used for Depth = -22.0625 in (Cubic Fit)
- Used for Depth = -25.8125 m (Cubic Fit)
- Used for Depth = -29.5625 in (Cubic Fit)
- Used for Depth = -37.0625 in (Cubic Fit)
- Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
**Test #3**: Load 10 = 125.0890 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #3: Load 10 = 125.0890 lb

Moment vs. Depth (Cubic Spline)

- ○○○ Data Points (Used for Cubic Spline)
- □□□ Top of Pile (Used for Cubic Spline)
- ■■■ Moment Linear Intercept (Used for Cubic Spline)
- ××× Erroneous Data Points (Not Used for Cubic Spline)
- - Cubic Spline of Moment Data

Pile Slope vs. Depth

- Pile Slope from Numerical Integration of Moment Spline
- ○○○ Pile Slope at Data Points
- □□□ B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- Pile Displacement from Numerical Integration of Pile Slope
- ○○○ Pile Displacement at Data Points
- □□□ B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #3: Load 11 = 63.3712 lb
Moment vs. Depth (Floating Curve Fit)

- ○○○ Data Points (Used for Curve Fitting)
- ●●● Top of Pile (Used for Curve Fitting)
- □□□ Moment Linear Intercept (Used for Curve Fitting)
- ××× Erroneous Data Points (Not Used for Curve Fitting)

- Used for Depth = 6.0614 to 0.0 in (Linear Fit)
- Used for Depth = -7.0625 in (Cubic Fit)
- Used for Depth = -10.8125 in (Cubic Fit)
- Used for Depth = -14.5625 in (Cubic Fit)
- Used for Depth = -18.3125 in (Cubic Fit)
- Used for Depth = -22.0625 in (Cubic Fit)
- Used for depth = -25.8125 in (Cubic Fit)
- Used for Depth = -29.5625 in (Cubic Fit)
- Used for Depth = -37.0625 in (Cubic Fit)
- Used for Depth = -40.8125 to -48.3125 in (Cubic Fit)
Test #3:  Load 11 = 63.3712 lb

Shear vs. Depth

Net Soil Load vs. Depth
Test #3: Load 11 = 63.3712 lb

Moment vs. Depth (Cubic Spline)

- Data Points (Used for Cubic Spline)
- Top of Pile (Used for Cubic Spline)
- Moment Linear Intercept (Used for Cubic Spline)
- Erroneous Data Points (Not Used for Cubic Spline)
- Cubic Spline of Moment Data

Pile Slope vs. Depth

- Pile Slope from Numerical Integration of Moment Spline
- Pile Slope at Data Points
- B.C. of Slope = 0 (Assumed at Pile Toe)

Pile Lateral Displacement vs. Depth

- Pile Displacement from Numerical Integration of Pile Slope
- Pile Displacement at Data Points
- B.C. of Displacement = 0 (Assumed at Pile Toe)
Test #3: P-Y Curves

P-Y Curves: Test 3

Figure C6. Test #3 p-y curves.
Test #3: P - Y Curves for Locations of Large, Single Direction Displacements

P - Y Curve's: Test 3

- Depth = - 7.0625 in.
- Depth = - 10.8125 in.
- Depth = - 14.5625 in.
Test #3: P - Y Curves for Locations with Small Displacements, and Small Stress Reversals

P - Y Curve's: Test 3

+ Depth = -18.3125 in

* Depth = -22.0625 in
Test #3: P - Y Curves for Locations with Small Displacements, and Large Stress Reversals

P - Y Curve's Test 3

Depth = -25.8125 in.
Depth = -29.5625 in.
Depth = -37.0625 in.
Depth = -40.8125 in.
Depth = -44.3625 in.
Depth = -48.3125 in.
Appendix D. Comparisons Between Test Data and Computer Predictions
Figure D1. Pile moment profile comparisons.
Moment Comparison
Load = 74.3 lb
Moment Comparison
Load = 114.2 lb

Depth (in)

Moment (in-kips)

- TEST # 1 (0 Load Cycle) - TEST # 3 (1 Load Cycle)
- COM624P (0 Load Cycle) - COM624P (1 Load Cycle)
- Florida Pier (0 Load Cycle) - Florida Pier (1 Load Cycle)
Moment Comparison
Load = 146.9 lb

Depth (in)

Moment (in-kips)

- TEST # 1 (0 Load Cycle)  - TEST # 3 (1 Load Cycle)
- COM624P (0 Load Cycle)  - COM624P (1 Load Cycle)
- Florida Pier (0 Load Cycle)  - Florida Pier (1 Load Cycle)
Moment Comparison
Load = 179.5 lb

- TEST # 1 (0 Load Cycle) - TEST # 3 (1 Load Cycle)
- COM624P (0 Load Cycle) - COM624P (1 Load Cycle)
- Florida Pier (0 Load Cycle) - Florida Pier (1 Load Cycle)
Moment Comparison
Load = 223.1 lb
Moment Comparison
Load = 255.8 lb
Moment Comparison
Load = 266.7 lb
Displacement Comparison
Load = 41.6 lb

Figure D2. Pile displacement profile comparisons.
Displacement Comparison
Load = 74.3 lb

Depth (in)

Pile Displacement (in)

- TEST # 1 (0 Load Cycle)  ○ TEST # 3 (1 Load Cycle)
- COM624P (0 Load Cycle)  ○ COM624P (1 Load Cycle)
- Florida Pier (0 Load Cycle)  ○ Florida Pier (1 Load Cycle)
Displacement Comparison
Load = 114.2 lb

- TEST # 1 (0 Load Cycle)  - TEST # 3 (1 Load Cycle)
- COM624P (0 Load Cycle)  - COM624P (1 Load Cycle)
- Florida Pier (0 Load Cycle)  - Florida Pier (1 Load Cycle)
Displacement Comparison
Load = 146.9 lb

- TEST # 1 (0 Load Cycle)  - TEST # 3 (1 Load Cycle)

- COM624P (0 Load Cycle)  - COM624P (1 Load Cycle)

- Florida Pier (0 Load Cycle)  - Florida Pier (1 Load Cycle)
Displacement Comparison
load = 179.5 lb

Pile Displacement (in)
Depth (in)

- Test # 1 (0 Load Cycle)
- TEST # 3 (1 Load Cycle)
- COM624P (0 Load Cycle)
- COM624P (1 Load Cycle)
- Florida Pier (0 Load Cycle)
- Florida Pier (1 Load Cycle)
Displacement Comparison
Load = 223.1 lb

![Displacement Comparison Graph](image)

- TEST # 1 (0 Load Cycle) - TEST # 3 (1 Load Cycle)
- COM624P (0 Load Cycle) - COM624P (1 Load Cycle)
- Florida Pier (0 Load Cycle) - Florida Pier (1 Load Cycle)
Displacement Comparison
Load = 255.8 lb

TEST # 1 (0 Load Cycle)  →  TEST # 3 (1 Load Cycle)
COM624P (0 Load Cycle)  →  COM624P (1 Load Cycle)
Florida Pier (0 Load Cycle)  →  Florida Pier (1 Load Cycle)
Displacement Comparison
Load = 266.7 lb

Depth (in) vs. Pile Displacement (in)

- TEST # 1 (0 Load Cycle) - TEST # 3 (1 Load Cycle)
- COM624P (0 Load Cycle) - COM624P (1 Load Cycle)
- Florida Pier (0 Load Cycle) - Florida Pier (1 load cycle)
Soil Load Comparison
Load = 41.6 lb

Figure D3. Soil load profile comparisons.
Soil Load Comparison
Load = 74.3 lb
Soil Load Comparison
Load = 114.2 lb

![Graph showing soil load comparison with different markers for test samples and locations.](Image)
Soil Load Comparison
Load = 146.9 lb

TEST # 1 (0 Load Cycle)  TEST # 3 (1 Load Cycle)
COM624P (0 Load Cycle)  COM624P (1 Load Cycle)
Florida Pier (0 Load Cycle)  Florida Pier (1 Load Cycle)
Soil Load Comparison
Load = 179.5 lb
Soil Load Comparison
Load = 223.1 lb

- TEST # 1 (0 Load Cycle)
- TEST # 3 (1 Load Cycle)
- COM624P (0 Load Cycle)
- COM624P (1 Load Cycle)
- Florida Pier (0 Load Cycle)
- Florida Pier (1 Load Cycle)
Soil Load Comparison
Load = 255.8 lb
Soil Load Comparison
Load = 266.7 lb

- TEST # 1 (0 Load Cycle) - TEST # 3 (1 Load Cycle)
- COM624P (0 Load Cycle) - COM624P (1 Load Cycle)
- Florida Pier (0 Load Cycle) - Florida Pier (1 Load Cycle)
Figure D4. $p$-$y$ curve comparisons.
P-Y Curves
Depth = -10.81 in.

![Graph showing P-Y curves with different test cycles and load conditions.](image-url)
P-Y Curves
Depth = -14.56 in.
P-Y Curves
Depth = -18.31 in.

Soil Load (lb/in.)

Displacement (in.)

- TEST # 1 (0 Load Cycle)
- TEST # 2 (1/2 Load Cycle)
- TEST # 3 (1 Load Cycle)
- Matlock (0 Load Cycle)
- Matlock (1 Load Cycle)
P-Y Curves
Depth = -22.06 in.

Soil Load (lb/in.)

Displacement (in.)

- TEST # 1 (0 Load Cycle)
- TEST # 2 (1/2 Load Cycle)
- TEST # 3 (1 Load Cycle)
- Matlock (0 Load Cycle)
- Matlock (1 Load Cycle)