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# A COMPARISON OF LOAD TEST DATA AND PREDICTED BEHAVIOR OF AUGER CAST PILES IN LAYERED SOILS

by

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# ABSTRACT

The use of auger cast-in-place (ACIP) pilings is very common in Florida; however, there is a significant degree of uncertainty in determining the actual capacity of the pilings, especially when the pilings are installed through layers of cohesive soils. Therefore, there is a need to improve upon the existing methods of predicting the behavior of ACIP piles in layered soils. As a result, the primary objective of this study is to determine if a significant difference exists between the accepted methods of pile load test analysis. Provided a significant difference is noted, the secondary objective would be to determine if an improvement could be made to enhance the existing empirical relationships used to predict pile behavior in layered soils.

In order to accomplish these objectives, this study presents an evaluation of some of the most commonly used methods for predicting ACIP pile capacity based upon the results of actual field load tests. Data from twenty-five load tests were analyzed using popular methods and statistical analyses were preformed to determine and evaluate the data. These evaluations were utilized to explore correlations between predicted behavior and actual results.

Based upon the results of this study, there is no statistically significant difference between the load test analyses methods examined. As a result, no improvement to the existing methods of predicting ACIP pile behavior in layered soils may be recommended at this time, and further research in this subject matter is recommended.

This body of work is dedicated to Mary Elizabeth Hudson

## ACKNOWLEDGMENTS

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## TABLE OF CONTENTS

# LIST OF FIGURES

## LIST OF TABLES

## LIST OF ACRONYMS/ ABBREVIATIONS

| | |
|---|---|
| AASHTO | American Association of State Highway Transportation Officials |
| ACIP | Auger cast-in-place piling |
| ASTM | American Society for Testing and Materials |
| CFA | Continuous Flight Auger |
| CPT | Cone Penetrometer Test |
| FHWA | United States Department of Transportation - Federal Highway Administration |
| IBC | International Building Code |
| ICC | International Code Council |
| NAVFAC | Naval Facilities Engineering Command |
| SPT | Standard Penetration Test |
| SPSS | Statistical Package for the Social Sciences |
| USCS | Unified Soil Classification System |

# CHAPTER 1: INTRODUCTION

The installation of pilings to support structures has been a common construction practice for thousands of years. There are historical examples of this type of construction found in several cultures including but not limited to the ancient Egyptians in North Africa, the Terramare in northern Italy, nomadic tribes of Vietnam, and early Scandinavians (Fitchen, 1986). Where ever there are people that want to access areas where the local conditions are not conducive for on-grade construction, or the loads are significantly large, pilings have been used. Piles can be created from a variety of materials including steel, timber, concrete, and even composites. Furthermore, there are several methods of installing piles, including jetting, driving, drilling, and vibratory placement.

For the purposes of this study, the discussion, review and analysis are concerned with and limited to auger cast-in-place, (ACIP) grout piles. ACIP piles are also referred to in some circles as continuous flight augered piles, (CFA); nevertheless, for this body of work they shall be designated ACIP piles. Although there are records indicating the use of ACIP piles in Texas in the 1950's, (McCleland, 1996), a method for installing auger cast-in-place pilings in the United States was first patented in 1968. O'Neill and Reese (1999), indicate that the expansive clays in the San Antonio area prompted the development of drilled piles in Texas. Since that time the use of ACIP piles has increased a great deal, but not to the extent that one might expect. The hesitance to use ACIP piles as a foundation solution is often linked to a perceived lack of quality control, and there is no doubt that quality control is crucial to obtain an end product that meets the design performance requirements. According to Brown, Thompson, & Nichols (2006), the requirement for an on-site engineer to monitor the installation, record the rate of grout placement, and to take samples of the grout for compressive strength tests is a necessity. On the other hand, according to Van Impe, Van Impe, and Verstraeten (1998), ACIP piles can be utilized in a variety of applications, including areas with low overheads, locations with noise and vibration restrictions, or in situations where relatively quick installation is required. Yet perhaps the greatest benefit of ACIP piles is that they are not limited by a pre-assumed length. Figure 1 depicts a typical ACIP pile detail, and one may easily visualize how the length could be adjusted. Therefore, if an undesirable or unanticipated condition is encountered in the field the job does not come to a halt while new piles are ordered.

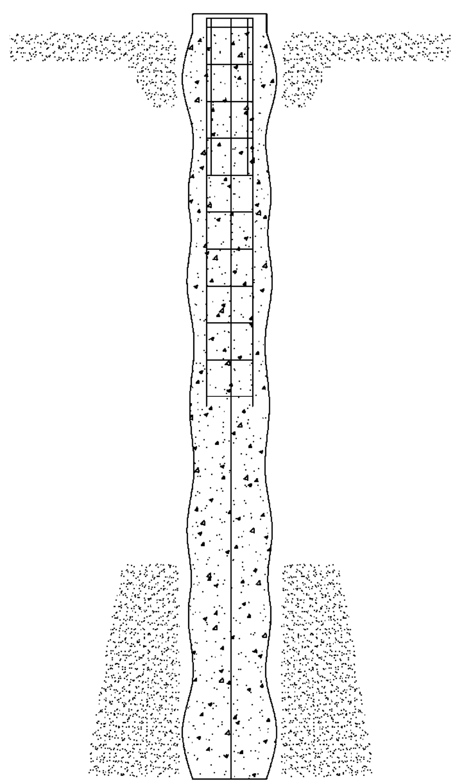

Figure 1: Typical ACIP Pile Detail

In that light, design engineers utilize field data from geotechnical investigations to produce calculations based upon accepted methods in order to predict anticipated capacity and settlement limitations for a given load on a pile. All piles rely on skin friction, end bearing, or a combination of the two in order to achieve the required compressive capacity to support the applied loads. However, the determination of the capacity based upon a given soil profile can produce a variety of results depending upon the method of analysis. This study will seek to determine the best-fit correlation between several of the popular methods of predicting pile capacity and load test data interpretation methods.

Ultimately, the best method to determine the true capacity and actual settlement from an applied load is an actual load test; unfortunately, load tests are very expensive. Moreover, the designer needs to have a high degree of confidence that the specification for the pile will provide the anticipated capacity with the required factor of safety, and stay within the tolerance limit of the specified settlement when the load test is conducted. In foundation design, the basic concern is how large does the foundation need to be to safely support the load without settling beyond a specified limit. Granted in the case of relatively light structures in high wind zones, the governing factor in foundation design may well be the required uplift resistance capacity of the foundation. Nonetheless, in the case of ACIP piles the compressive loads will often exceed the uplift loads by an order of magnitude. Therefore, where compressive loads are concerned, the capacity is often governed by how much the foundation actually settles.

## 1.1 Research Objective

The objective of this study is to compare the anticipated design values based upon soil parameters from geotechnical reports, boring logs, and field engineer test pile installation logs, with actual load test data from test piles. Based upon the information collected, this study will focus on comparing the predicted capacity and to a limited degree the anticipated settlement at a given load to the results of actual load tests. The actual load test will also be analyzed by popular methods and a best-fit correlation will be determined. Ideally, an empirical relationship will be drawn from the comparison between the anticipated capacity verses the results of the load test data.

The correlation between anticipated capacity and load test data is important to enable designers to better determine the required diameter and depth for proposed ACIP pilings. As the demand increases for dwellings particularly in coastal areas, more structures will be built that will require piles for support. Moreover, the areas previously bypassed by developers, such as areas with significant layers of cohesive soils, will likely become more desirable for construction purposes. Furthermore, the United States Department of Transportation - Federal Highway Administration, FHWA, in a 2006 article praising the merits of on ACIP piles in the construction and repair of transportation foundation projects, stated:

> "…continuous flight augured piles can be installed quickly and inexpensively and are a viable foundation alternative to driven piles…[and] are a good deep foundation solution in areas that are environmentally sensitive or require minimal disturbance to human activity." (FHWA, 2006)

According to Hoback and Rujipakorn (2004), the construction of drilled piles in unstable soil is difficult because the soil has the potential to contaminate the pile. The tendency for this type of contamination is greatly reduced with ACIP since the concrete grout is installed under pressure as the auger is removed from the hole as opposed to pouring concrete into a pre-drilled cavity.

With construction costs escalating, and the number of competing design firms increasing, the designer is forced perhaps more than ever before, to remain cognizant of the necessity of economic feasibility with their design. ACIP piles are a viable alternative to other deep foundation alternatives, but there is a need to refine the design process to make accurate performance predictions. Therefore, a desire of this effort is to determine the most reliable method for predicting capacity and to determine if the analysis indicates than an enhancement to that method is appropriate.

## 1.2 Research Approach

The basic approach taken for this study was to gather and review existing geotechnical reports and load test data for projects where ACIP piles were utilized. An additional requirement was that the subsurface profiles needed to include layers of cohesive material. For the purposes of this study cohesive layers are required to comprise a minimum of 25 percent of the strata in which the ACIP pile is installed. The original research intention was to search for ACIP piles placed and founded in predominately cohesive soils. However, fulfilling the requirement for tests founded on predominately cohesive material became quite a challenge. The reports and test data that were so generously provided were not typically for ACIP piles that were actually founded in cohesive soils. As a general practice in this area of the country the drillers will penetrate the cohesive layers until a suitable sand layer is reached.

Therefore, the data collected was filtered to 25 samples where the ACIP piles were installed in layered soils with a minimum of 25 percent cohesive material. The actual data used contained layered soils with clay content ranging from a minimum of 26% to a maximum of 52% of the individual test pile soil strata. The geotechnical reports were evaluated and the generalized soil profiles were determined. Then calculations were preformed to determine the anticipated capacity and settlement tolerance of the specified ACIP pile. The test pile data were then evaluated with one of a few accepted methods, namely, the Davisson Offset Limit, the Chin-Kondner Extrapolation, the Five Percent method, and the Army Corps of Engineers' procedure. Both the Davisson Offset Limit and the Chin-Kondner Extrapolation are approved methods for the 2003 edition of the International Building Code. Finally, a statistical analysis of the results between the anticipated behavior calculations and the load test data interpretation calculations was completed.

# CHAPTER 2: LITERATURE REVIEW

Before embarking upon a study of ACIP piles, a review of pertinent information is required. This chapter will provide a brief review of several topics related to the design, installation, capacity, testing, and analysis of ACIP piles. Since this study is related to ACIP piles installed in layered soils, the review will begin with a discussion of the properties and characteristics of both sand and clay.

## 2.1 Soil Properties

The substance commonly referred to as soil is actually a composite of organic matter and various minerals. Soil may include particles of various types and sizes of sand, clay, organic compounds, and sediments, (Holtz & Kovacs, 1981).

> According to Craig (1999): "…the destructive process in the formation of soil from rock may be either physical or chemical. The physical process produces particles that retain the same composition as the parent rock…the chemical process results in changes from the parent rock due to the action of water…resulting in the formation of groups of crystalline particles of the colloidal size (0.002 mm) known as clay minerals."

There are two major soil classification systems currently in use in this country, the *Unified Soil Classification System* (USCS), and the *American Association of State Highway Transportation Officials* (AASHTO). As the name indicates the AASHTO system is the standard for transportation sub-grade applications, and the USCS is the standard for foundation applications. The USCS method is specified in ASTM D2487-06.

### 2.1.1 Sand

The USCS classifies soils as either coarse or fine grained, with coarse grains designated as those that will retain more than half of the material on a No. 200 sieve. Sands are further distinguished as those having a greater percentage passing through a No. 4 sieve. Sands are then subdivided further into categories depending upon the distribution of the grain size, and how much material passes through a No. 200 sieve. The gradation of a specific sample refers to the distribution of particle sizes in the sample. A well graded soil produces a smooth concave curve across the range of particles sizes when plotted on a graph (Craig, 1999). The AASHTO system classifies soils by one of eight groups with granular material designated by groups A1 through A-3; however, group A-2 may have significant levels of clays and silts (Das, 2005).

### 2.1.2 Clay

Clays are distinguished by the USCS as fine grained soils that more than half the material sample passes through a No. 200 sieve. Fine grained soils are divided into two categories, silts and clays, based upon their Liquid Limit designation. Fine grained solids, silts and clays, are then subdivided based upon the level of organics and their respective plastic limit. The liquid limit, (LL), represents the water content where the behavior of a soil changes from that of a plastic to that of a liquid. The plastic limit, (PL), is the water content where soil starts to exhibit plastic behavior. The plasticity index (PI), is defines as the difference between the liquid limit and the plastic limit (Das. 2005). The designations are based upon tests developed by Swedish scientist A. Atterberg, and later standardized by Terzaghi and Casagrande (Holtz & Kovacs, 1981). With the AASHTO system fine grained soils fall into groups A-4 through A-7. Like the USCS system groups A-4 through A-7 as well as the subgroups of A-2, are delineated by using the Atterberg system.

## 2.2 Soil Boring and Sampling

When a geotechnical engineering firm is directed to conduct a site exploration for a proposed structure, a design of the foundation is typically recommended. One of the standard approaches in subsurface investigations is for the engineer to review of the preliminary drawing from the project architect, civil and structural engineer to determine the type and size of the structure as well as the location of proposed drainage and architectural features. In addition, the geotechnical engineer will have discussions with the structural engineer to determine the anticipated loads, factor of safety, and foundation type. The engineer will then make a visit to the site and determine the best location for soil borings to be made.

Soil boring involves drilling into the earth recording the level of the water table if encountered, and taking samples of the soil at various intervals. The Standard Penetration Test, (SPT), is the most common type of subsurface testing in this region of the county. However, the use of the Cone Penetrometer test, (CPT), is becoming more popular. The major drawback of the CPT is the inability to obtain an actual soil sample, yet many firms are opting for the CPT due to the speed and cost savings as compared to SPT testing. Nevertheless, all of the tests conducted that were retained in the data sample for this effort were conducted using the SPT method.

The SPT method is the field test on samples collected by a split-spoon sampler, and is this most common method used for obtaining soil samples (Das 2007). As the name implies, a split-spoon sampler is constructed out of steel tubing that splits in two along the length of the shaft. A coupling is used to connect the tube to the drilling rod. Test samples are typically taken continuously for the upper ten feet of the boring. Afterwards the boring is typically drilled in intervals of five feet and then the drilling apparatus is extracted and the split tube is inserted into the hole. The tube is lowered into the bottom of the test hole and is driven with a 140 pound hammered in three six inch intervals. The number of blows with the hammer are added together for the second two intervals and that determines the standard penetration number, N, at that depth, (Das, 2007).

According to Bowles (1996), common practice in analyzing a given SPT test is to utilize correlations that have been made between the N value and other properties of a given soil layer including the angle of internal friction and the unconfined compression strength. The samples obtained from the SPT are then sealed and taken to the laboratory for analysis. Depending upon the size of the particles, the laboratory will perform a sieve, and if necessary, a hydrometer analysis, and also determine the liquid and plastic limits. Finally each sample is then classified based upon the USCS or the AASHTO, system as required. In addition, the laboratory will determine the moisture and organic content, and the specific gravity of the sample. Each of the tests performed in the lab have a specific protocol specified by the American Society for Testing and Materials, (ASTM).

## 2.3 Data Evaluation

Once the sample has been evaluated in the laboratory, the geotechnical engineer reviews the information with consideration to both the structural engineer's specifications and the architect's design requirements. In some cases the structural engineer may specify a maximum allowable deflection for the pile. Another common method is for the structural engineer to calculate the anticipated service loads and then request foundation design recommendations from the project geotechnical engineer. The geotechnical engineer will then determine the best type of foundation for the site and the end use and size it accordingly. As previously stated, all of the reports selected for this study utilized ACIP piles.

As mentioned in the introduction, differences occur in the calculated capacity of a given pile in the same soil profile based upon the method chosen for evaluation. Therefore, the logical approach is to first determine the ultimate capacity by the each of the four methods chosen to provide anticipated values. The next step is to determine if a significant statistical difference is present between the various methods and the four load test interpretation methods. Likewise, the theoretical and elastic deflection is calculated for comparison.

## 2.4 Bearing Capacity Methods of a Pile

As stated in the introduction, the bearing capacity of a pile is based upon both the skin friction $Q_s$, generated with the interaction of the pile and the surrounding soil mass, and the end bearing capacity $Q_p$, of the soil strata below the tip of the pile. According to Das (2007) the general equation for ultimate bearing capacity is given as:

$$Q_{ult} = Q_s + Q_p \qquad [2.1]$$

Naturally if the pile is resting upon bedrock, the capacity of the pile becomes more like a column analysis and the contribution of skin friction may not be applicable. However, this study is conducted on ACIP test piles placed layered soils without encountering limestone, bedrock, or any hard rock. Therefore the concern is primarily with skin friction and to a lesser extent end bearing capacity generated by the pile. According to Meyerhof (1983), the bored piles have an ultimate unit point resistance of only about one-third that of similar driven piles, because driving compresses the soil strata below the tip and therefore increases the capacity.

The Corps of Engineers (1993) revealed that layered soils present a problem in determining safe bearing capacity because the soils may cause service piles to perform differently than indicated by test piles. At the time of the test, the pile may receive support from an unconsolidated cohesive layer. Then over time, this same cohesive layer may consolidate under the load, and transfer the load to another soil layer not stressed during the pile test according to Chin and Vail, (1973). Should this type of consolidation occur, the member could actually become a point bearing pile rather than a skin friction pile. In those cases unacceptable settlements may occur.

### 2.4.1 Skin Friction

Obviously the capacity of a pile has a direct relationship with the diameter of that pile, and to an even greater extent the length of the embedment. This assertion is supported by Meyerhof (1983), when he stated that the “ultimate skin friction of piles in sand…[and] clay of a given shear strength is practically independent of pile diameter.” When considering the skin friction capacity, *Qs*, of a pile the theory is to sum the surface area of embedment multiplied by the frictional resistance of the soil (Das 2007).

$$Qs = \Sigma p \Delta L f \qquad [2.2]$$

where

$p$ = the perimeter of the pile

$\Delta L$ = the length of the pile over which $p$ and $f$ are constant

$f$ = the frictional resistance of the soils at a given depth

However, the difficult part of this equation is determining the frictional resistance. In fact, there are several popular methods for calculating the frictional resistance of piles, including the $\lambda$ method and the $\beta$ methods. Regrettably, the studies that developed these methods relate more too driven piles than drilled piles, (Bowles, 1996 & Das, 2007). Therefore, this evaluation utilizes both the Naval Facilities Engineering Command (NAVFAC) method and the Coyle and Castello method for calculating skin friction capacity.

According to Das (2007) the frictional resistance of a pile may be calculated by the following relationship:

$$f = K\sigma' \tan\delta' \qquad [2.3]$$

where

$K$ = the coefficient of lateral earth pressure

$\sigma'$ = the effective stress

$\delta'$ = the effective angle of friction between the pile and the soil

Both the NAVFAC method and the Coyle and Castello method agree with the basic concept of equation [2.2]; however, there are a few differences in the method of calculating the effective angle of friction, the effective stress, and the coefficient of lateral earth pressure.

#### 2.4.1.1 NAVFAC

Based upon the method prescribed by the NAVFAC's design manual (1986), the skin friction capacity in sand is calculated by:

$$Q_s = \sum_{H=H_o}^{H=H_o+D} (K_{HC})(P_o)(\tan\delta)(s) \qquad [2.4]$$

where

$K_{HC} =$ the ratio of horizontal to vertical effective stress, given as 0.7 for drilled piles

$P_o =$ the effective vertical stress over the length of embedment

$s$ = the surface area of pile per unit length

$$\delta = 0.75\,\phi' \qquad [2.5]$$

The NAVFAC method sets the coefficient of lateral earth pressure at a set value of 0.7 for drilled concrete piles, and the effective angle of friction at $0.75\,\phi'$. Studies show that the frictional resistance increases with the depth of embedment rather linearly up to a certain depth commonly defined as *L'*. After that depth is reached, the frictional resistance remains relatively constant. In this method the length *L'* is limited to 20 times the pile diameter. According to NAVFAC (1986) the skin friction capacity in clay soils is given as:

$$Qs = c_A 2\pi R z \qquad [2.6]$$

where

$c_A =$ the adhesion between the clay and the surface of the pile in psf

$R =$ the radius of the pile

$z$ =the depth of embedment

As shown if Figure 2, the NAVFAC (1986) Design Manual provides a chart correlating recommended values of adhesion with respect to cohesion for various types of piles. For soft and very soft clays the cohesion is approximately equal to the adhesion; however as the consistency of the soil increases the adhesion does not increase at the same rate as the cohesion, such that for very stiff soils the adhesion is approximately one-third the value of the cohesion.

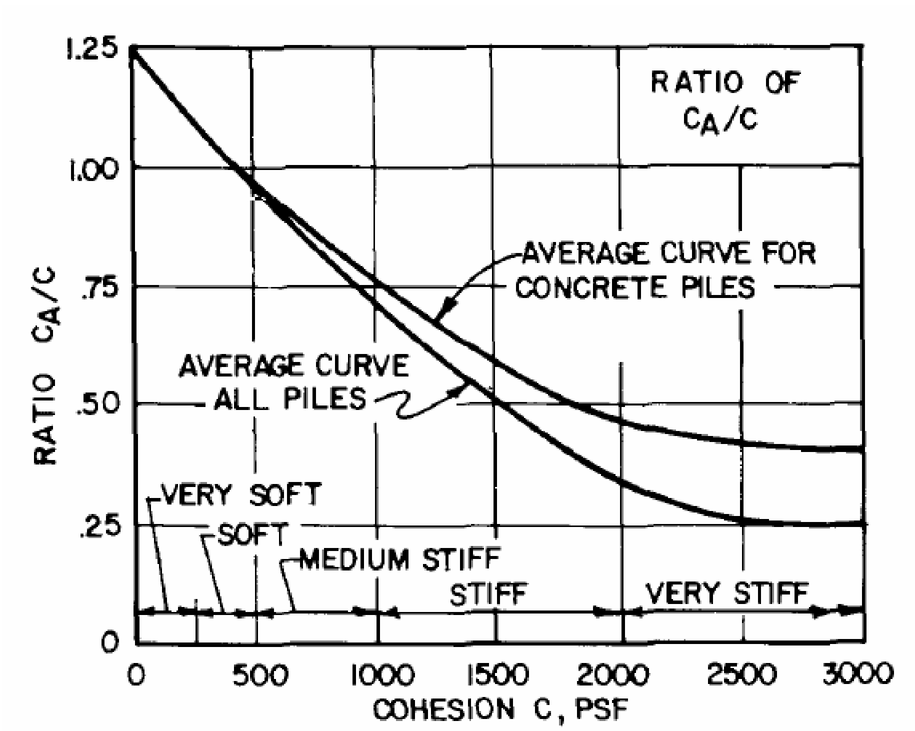


Figure 2: Recommended Values of Adhesion from NAVFAC

#### 2.4.1.2 Coyle and Castello

According to the work by Coyle and Castello in the early 1980's, the skin friction capacity in sand may be calculated as follows:

$$Q_s = f_{av} pL = (K \bar{\sigma}_o' \tan \delta') pL \qquad [2.7]$$

where

$\bar{\sigma}_o'$ = the average effective overburden pressure

$\delta'$ = soil-pile friction angle = $0.8\,\phi'$ [2.8]

$L = 15D$ [2.9]

$K$ from a published table that varies with $L/D$ [2.10]

Interestingly, in 1980 Meyerhof noted that in comparative pile load test in soft sensitive clay in Sweden (Fellenius 1955) pile taper had no significant effect on the skin friction, even when compared with the piles with an upward taper.

### 2.4.2 End Bearing

The end bearing or point capacity of a pile is classically calculated by multiplying the surface area of the tip or pile point by the bearing capacity of the soil stratum directly beneath (Das 2007).

$$Q_p = A_p q_p \qquad [2.11]$$

The basic equation for the bearing capacity is usually taken from the work of Terzaghi, who according to Holtz and Kovacs (1981) is known as the father of soil mechanics. From that point the bearing capacity equation is then modified by various factors depending upon the studies of those who came after him. According to Bowles (1996), Terzaghi's original bearing capacity equation for a circular footing is given as follows:

$$Q_p = c' N_c + qN_q + 0.3\gamma B N_\gamma \qquad [2.12]$$

where

$c'$ = the cohesion of the soil strata

$N_c$ = the cohesion factor given by $N_c = \tan\phi'(K_c + 1)$ [2.13]

$q$ = the surcharge pressure

$N_q$ = the surcharge factor given by $N_q = Kq \tan \phi'$ [2.14]

B = the diameter of the footing

$N_\gamma$ = the shape factor given by $N_\gamma = 1/2 \tan \phi' (K_\gamma \tan \phi' - 1)$ [2.15]

where

$K$ = the coefficient of lateral earth pressure

$\phi'$ = the effective angle of friction

However, most pile capacity equations simple drop the last set of terms in Terzaghi's original equation since the effect upon the capacity is insignificant. Therefore, most common difference in the various methods of determining end bearing capacity involves the calculation of the factor $N_q^*$, and is related to the dimensional characteristics of the soil below the foundation element. However, the popular methods for determining the end bearing capacity can produce varying results for the same soil strata (Bowles, 1996).

**<u>2.4.2.1 Meyerhof</u>**

G.G. Meyerhof has been quite prolific in the contributions to the field of geotechnical engineering. His research and publications are referenced in virtually every geotechnical engineering text and journal published in the past thirty years. In 1976 he published research pertaining to his determination that values for the bearing capacity factor $N_q^*$ were somewhat different from the original (Meyerhof, 1976). His basic equation for the end bearing capacity of a pile in sand is given as follows:

$$Q_p = A_p q' N_q^* \quad [2.16]$$

However, the method of calculation of the coefficients is different as shown:

$$N_q^* = e^{\pi \tan \phi'} \tan^2(45 + \frac{\phi'}{2}) \quad [2.17]$$

Meyerhof then limited his bearing capacity equation such that the result could not exceed the limit of $A_p q_t$ as follows:

$$q_t = 0.5 \rho_a N_q^* \tan \phi' \quad [2.18]$$

where

$\rho_a$ = the atmospheric pressure of 2000 psf

**<u>2.4.2.2 Vesic'</u>**

According to Hoback and Rujipakorn (2004) "in 1967 Vesic' compared the theoretical results relating to the variation of the bearing capacity of sand $N_q$, to the soil friction angle." Vesic' also proposed "that the ultimate bearing capacity of a cohesive soil is equal to $N_c^*$ multiplied by the undrained shear strength", (Hoback, 2004) and in 1975 Vesic' developed his own modification for determining the value of a pile's end bearing capacity as shown in the following equation:

$$Q_p = (c' N_c^* + \bar{\sigma}' N_\sigma^*) \quad [2.19]$$

where

$$N_c^* = (N_q^* - 1)\cot\phi' \quad [2.20]$$

and

$$N_\sigma^* = \frac{3N_q^*}{(1+2K_o)} \quad [2.21]$$

$N_\sigma^*$ is the product of *f* and a factor called the reduced rigidity index, *Irr* as follows:

$$Irr = \frac{Ir}{1+\varepsilon u Ir} \quad [2.22]$$

where

$\varepsilon u$ is the volumetric strain given by the initial volume divided by the change in volume:

$$\varepsilon u = \frac{V}{\Delta V} \quad [2.23]$$

and *Ir* is the rigidity index given by:

$$Ir = \frac{G'}{c + \bar{q}\tan\phi} \quad [2.24]$$

the value of *Irr* varies with the density of the soil and is commonly determined by taking the value or interpolating from tables.

### 2.4.2.3 Janbu

From his research on the topic of end bearing, Janbu (1976) determined that the value of $N_q^*$ should be calculated based upon the following equation:

$$N_q^* = (\tan\phi + \sqrt{1+\tan^2\phi})^2 \exp(2\psi\tan\phi) \quad [2.25]$$

where

$\psi$ = is an angle of the failure plane at the pile tip that can vary from $60^{o}$ in soft soils that compress easily to $105^{o}$ in soils that are very dense.

### 2.4.2.4 NAVFAC

The NAVFAC design manual (1986) presents a similar equation for the end bearing capacity in sand as follows:

$$Qp = P_T N_q^* A_T \quad [2.26]$$

where

$P_T$ = the effective vertical stress at the pile tip limited to $L'$=20D [2.27]

$N_q^*$ = the bearing factor from a published table

$A_T$ = the area of the pile tip

Each of the preceding methods for calculating end bearing capacity examined is typically used with sands. However, in saturated clay with $\phi' = 0$, experiments show that the value of $N_c^*$ reaches an approximate maximum value of nine when compared to the ratio of depth to width of a foundation (Bowles, 1996 and Das, 2007). Therefore, the point bearing capacity is generally taken as:

$$Q_p = N_c^* c_u A_p = 9 c_u A_p \quad [2.28]$$

where

$c_u$ = the undrained cohesion of the soil strata beneath the pile tip

Each of the aforementioned methods for determining skin friction and bearing capacity may be utilized to determine the predicted capacity of ACIP piles. However, some of the methods were derived from theory and tests on driven piles rather than actual ACIP piles. Many foundation textbooks will provide pile capacity equations that exist based upon research on drilled shafts; however, drilled shafts are commonly described as those having a diameter of about 2.5 feet, (Das, 2007). As previously stated, the maximum diameter for the ACIP piles considered in this study is 24 inches; therefore, drilled shaft equations have not been considered. Unfortunately, according to Kulhaway & Chen (2005), there is not a fundamental model specifically designed for ACIP piles.

## 2.5 Down drag forces in clay

Down drag or negative skin friction is a force that can greatly reduce the capacity of a pile. These are forces which may be applied to a pile in cohesive layers by the adjacent soil under certain conditions. According to research by Fellenius (1972) the force on piles due to the reconsolidation effect of cohesive soils can be quite large, and are greatly affected by the water table. In his study on down drag forces annual settlement for the test piles averaged only 2-3mm (0.1 in.) per year for the first 43 months. However, after a severe drought the following summer the study noted a settlement of 0.6 inches was observed (measured). According to Das (2007) down drag forces must be considered when:

- granular fill is placed over a soft cohesive layer
- cohesive fill is placed over a granular layer if the pile is driven

- when the water table is lowered since clay will consolidate and $\sigma'$ will increase

Research published by Kuwabara and Poulos (1989) in the Journal of Geotechnical Engineering indicates that down drag forces is more of concern on individual end bearing piles than on pile groups, because with individual piles, it is usually assumed that full slip will occur over the pile-soil interface (ASCE, 1989). Therefore, Fellinius (1972) recommended eliminating the contribution of the skin friction in cohesive layers from the ultimate pile capacity calculation. Current theory breaks the down drag calculation process into two equations. For cohesive fill over granular soils the following equation is recommended by Das (2007):

$$Q_n = \int_0^{H_f} (pK^{'}\gamma_f^{'} \tan\delta^{'})zdz = \frac{pK^{'}\gamma_f^{'}H_f^2 \tan\delta^{'}}{2} \qquad [2.27]$$

where

$H_f^2$ = the height of the fill

$\gamma_f^{'}$ = the effective unit weight of the soil if fill is below the water table

$K^{'}$ = the earth pressure coefficient 1-sin$\phi^{'}$

$\sigma_0^{'}$ = the soil pile friction angle

For granular fill over clay layers Das (2007) recommends:

$$Q_n = \int_0^{L_f} pf_n zdz = (pK^{'}\gamma_f^{'}H_f \tan\delta^{'})L_1 + \frac{L_1^2 pK^{'}\gamma_f^{'} \tan\delta^{'}}{2} \qquad [2.28]$$

where

$L_1$ = the neutral depth (Vesic', 1975) according to Bowles (1996):

$$L_1 = \frac{(L - H_f)}{L_1}\left[\frac{L - H_f}{2} + \frac{\gamma_f^{'} H_f}{\gamma^{'}}\right] - \frac{2\gamma_f^{'} H_f}{\gamma^{'}} \qquad [2.29]$$

However, for this study the load tests were conducted prior to fill being placed on the site, and no data were available to indicate the type or volume of anticipated fill material. Therefore, the effects of down drag forces attributing to the construction related fill could not be accurately calculated.

## 2.6 Settlement limit specifications

There are several methods that are used to predict the amount of settlement from design loads placed on a given pile, and it must be noted that failure of a pile is not simply the point at which the soil is fully mobilized or the pile material breaks down. Rather, failure for a pile is in reality the amount of settlement caused by the undesirable effect upon the structure (Ng, 2004). For that reason, the settlement limit is what the design engineer calls for in the specifications. According to the NAVFAC (1986) documents a deflection criterion is normally used to define failure of the pile. In the absence of an over-riding project specification criterion, the NAVFAC (1986) recommends using ¾ inch net settlement at twice the design load to define pile failure due to settlement.

There are several methods for interpreting the results of pile load tests with respect to settlement. However, a distinction must be made between the settlement due to the deflection of the pile and the settlement due to the compression of the soil. The deflection of the pile, or theoretical settlement, is the decrease in length due to compressive forces on the pile itself. The deflection of the soil is actually the summation of the settlement attributable to the compression of the strata in response to the transmission of the load from the pile.

### 2.6.1 Theoretical Settlement Method

One of the standard methods in evaluating predicted pile settlement is called the theoretical method from mechanics of materials, which is actually an application of Hook's Law. According to Beer, Johnston, and Wolf (2001), since the diameter of the shaft and the modulus of elasticity of the grout are theoretically constant, the predicted deflection of a pile is given by the equation:

$$\Delta = \frac{PL}{AE} \qquad [2.30]$$

where

$P$ = the applied load

$L$ = the length of embedment

$A$ = the surface area of the pile per unit length

$E$ = the modulus of elasticity of the concrete grout

This method is also used as the basis for the Davisson Offset method which will be discussed in section 2.6.1.

### 2.6.2 Five Percent Method

According to Charles Ng (2004), Terzaghi originally proposed that the ultimate capacity of a pile is the load that produces a settlement equal to ten percent of the pile diameter. Although this method has been commonly utilized by engineers in the past, subsequent research has shown that a ten percent settlement limit may exceed the acceptable limits for working loads. Therefore, a more conservative approach of limiting the allowable settlement to five percent of the pile diameter has become common (Ng, 2004)

## 2.7 Testing Methods

The book *"Load Testing of Deep Foundations"* by Crowther (1988) provides the following definitions:

- A *load* is an amount carried at one time; the weight borne up by a structure; a varying weight.
- A *test* is an examination of something's value; the method or criterion used in this examination; an event that evaluates quantities.

Therefore, a load test is a method used in the examination of the amount of weight that can be carried by a structural unit. In the case at hand, the structure is a deep foundation.

Load tests on piles maybe either static or dynamic, and there are specific ASTM standards for performing each type. Typically, the pile is loaded and the resulting settlement is recorded. For the pile tests reviewed as part of this study the test method used was the *Standard Test Method* *for Individual Piles Under Axial Compressive Load* (ASTM D 1143-81 Reapproved 1994). The standard specifically states that a qualified geotechnical engineer is required to interpret the results of the aforementioned tests “so as to predict the actual performance and adequacy of piles used in the constructed foundation.”

In addition, the standard provides several approved procedures for conducting the test including:

- Procedure A  Quick Test
- Procedure B  Maintained Test
- Procedure C Loading in Excess of Maintained Test
- Procedure D Constant Time Interval Test (optional)
- Procedure E Constant Rate of Penetration Test
- Procedure F Constant Movement Increment Test
- Procedure G Cyclic Loading Test

The load test provides information that reveals the amount of settlement or movement of the pile in response to the application of the load. According to the Corps of Engineers (1991), “a load cell should be used to measure load instead of the pressure gage on the jack because pressure gage measurements are known to be inaccurate.” The actual movement of the pile is commonly measured through the use of telltales and gauges attached to the pile while the load is applied. With this information a plot is made with the settlement on the vertical axis and the applied load on the horizontal axis. The resulting curve may then be analyzed to draw inferences regarding the capacity and settlement of the pile.

## 2.8 Test Pile Installation

Once the geotechnical engineer has reviewed the data, and completed his preliminary recommendations, the test and anchor piles must be installed. Typically the projects reviewed utilized four anchor piles to one test pile. The piles are all ACIP and the installation is monitored for quality control by a representative of the geotechnical engineering firm responsible for the design. The quality control inspector is most often an engineer intern; but in some cases an actual professional engineer will be on site for the installation. Figure 3 shows the actual installation of an ACIP pile.

The grout used to construct the pile must be sampled for strength test evaluations. The pump delivering the grout must be calibrated, and the installation must be monitored to ensure the pile is cast according to the design specifications. According to the IBC (2003),

> "Concrete pumping pressures shall be measured and maintained high enough at all times to offset hydrostatic and lateral earth pressures. Concrete volumes shall be measured to ensure that the volume of the concrete placed in each pile is equal to or greater than the theoretical volume of the hole created by the auger. Where the installation process of any pile is interrupted or a loss of concreting pressure occurs, the pile shall be redrilled to 5 feet below the elevation of the tip of the auger when the installation was interrupted or concrete pressure was lost or reformed. "

Figure 3: ACIP Installation

The rate of grout delivery and the speed at which the auger is extracted is critical to the production of a pile with uniform diameter. Without a consistent regulated delivery of grout the pile may have large variations in the cross sectional area along the length of the shaft. Once the test piles have been installed in accordance with the specifications, they are allowed to cure for at least seven days before the actual field load tests are conducted.

## 2.9 Test Analysis Methods

There are several available methods for analyzing pile load test results. According to Crowther (1988), the rules for acceptance should be defined prior to the evaluation of the test. The engineer must be familiar with the local codes and any governing specifications. In some cases the deflection sets the limit and in some cases the intensity of the applied load controls. Designations such as “failure” or “ultimate load” are subjective unless they are predefined. For this study four methods were chosen for comparison. Those methods are the Davisson Offset Method, the Chin-Kondner Method, the Five Percent of the Pile Diameter Method, and the Corps of Engineers method.

### 2.9.1 Davisson Offset Method

The Davisson method is the most widely used method of evaluation in use today, and is the defacto standard. Davisson has proven to be conservative, yet fair, and results in acceptable settlement. (IBC, 2005). This method starts with the theoretical settlement equation [2.30] and basically adds an empirical offset obtained through experimentation. The offset line in conjunction with the loading plot is observed to determine a failure load. According to Corps of Engineers (1991), the equation is given as follows:

$$\Delta = \frac{PL}{AE} + (0.15 + \frac{D}{120}) \qquad [2.31]$$

The resulting displacement is plotted against the applied load in tons, on the same graph with a plot of the actual settlement verses the applied load. As shown if Figure 4, a pile's bearing capacity failure $Qf$ is defined as the point of intersection between the actual measured load test deflection and the Davisson Offset line. Fellenius (2001) notes:

> '…the Offset Limit Load is not necessarily the ultimate load. The method is based on the assumption that capacity is reached at a certain small toe movement and tries to estimate that movement by compensating for the stiffness (length and diameter) of the pile. It was developed by correlating—to one single criterion—a large number subjectively determined pile capacities for a data base of pile loading tests. It is primarily intended for test results from driven piles tested according to quick methods."

The results of the Davisson Offset method for each of the pile tests evaluated are provided in Appendix B. A factor of safety of at least two (2) must be applied to determine the allowable working load. Moreover, the resulting deflection must be compared to the original specifications for the project to ensure compliance with design of the engineer of responsible charge for the project.

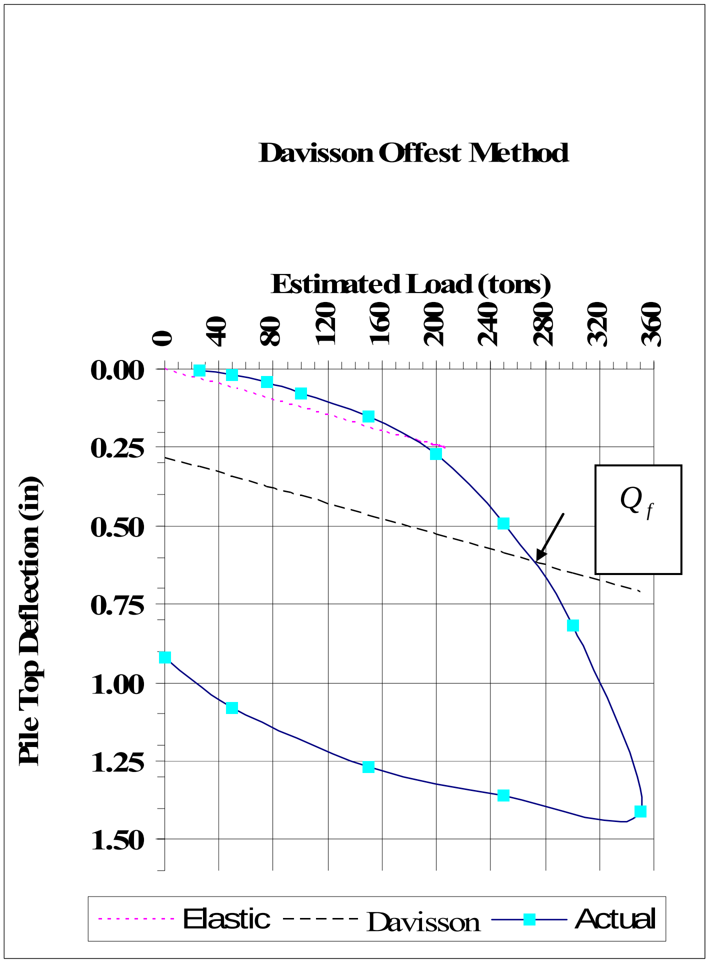


Figure 4: Davisson Offset Example Plot (Ref. TP-9)

### 2.9.2 Chin-Kondner Extrapolation

The Chin-Kondner Extrapolation method is somewhat convoluted in the approach to determining the capacity of a pile. According to Roscoe, Dic and Mice (1984), Vesic' noted that "shaft friction is mobilized at small settlement (6 to 10 mm) and that end bearing is not fully mobilized until much greater settlements of up to 30% of the base diameter of the pile occur." With that in mind, Chin came up with a method to separate the contribution of skin friction and end bearing from load test data. His method “assumes a relationship between the applied load (P) and settlement (∆) is hyperbolic.” (Roscoe et al., 1984). Therefore, the deflection from the applied load can be plotted on a horizontal axis against that same deflection divided by the applied load on a vertical axis. As shown in Figure 5, the resulting plot typically takes the form of a line with two distinct breaks with the initial portion relating to skin friction capacity and the second portion related to the ultimate bearing capacity. The reciprocal of the slope of the portion of the plot after the initial break is calculated to determine the ultimate capacity of the pile. The second portion of the line is also extended to the vertical axis at the break point to determine the y-intercept. To calculate the load for a given settlement the following equation and typical plot are used:

$$Q_x = \frac{x}{mx + c} \qquad [2.32]$$

where:

$x$ = the settlement in inches

$m$ = the slope of the second portion of the line

$c$ = the y-axis intercept

The results of the Chin-Kondner Extrapolation for each of the test piles evaluated are provided in Appendix B.

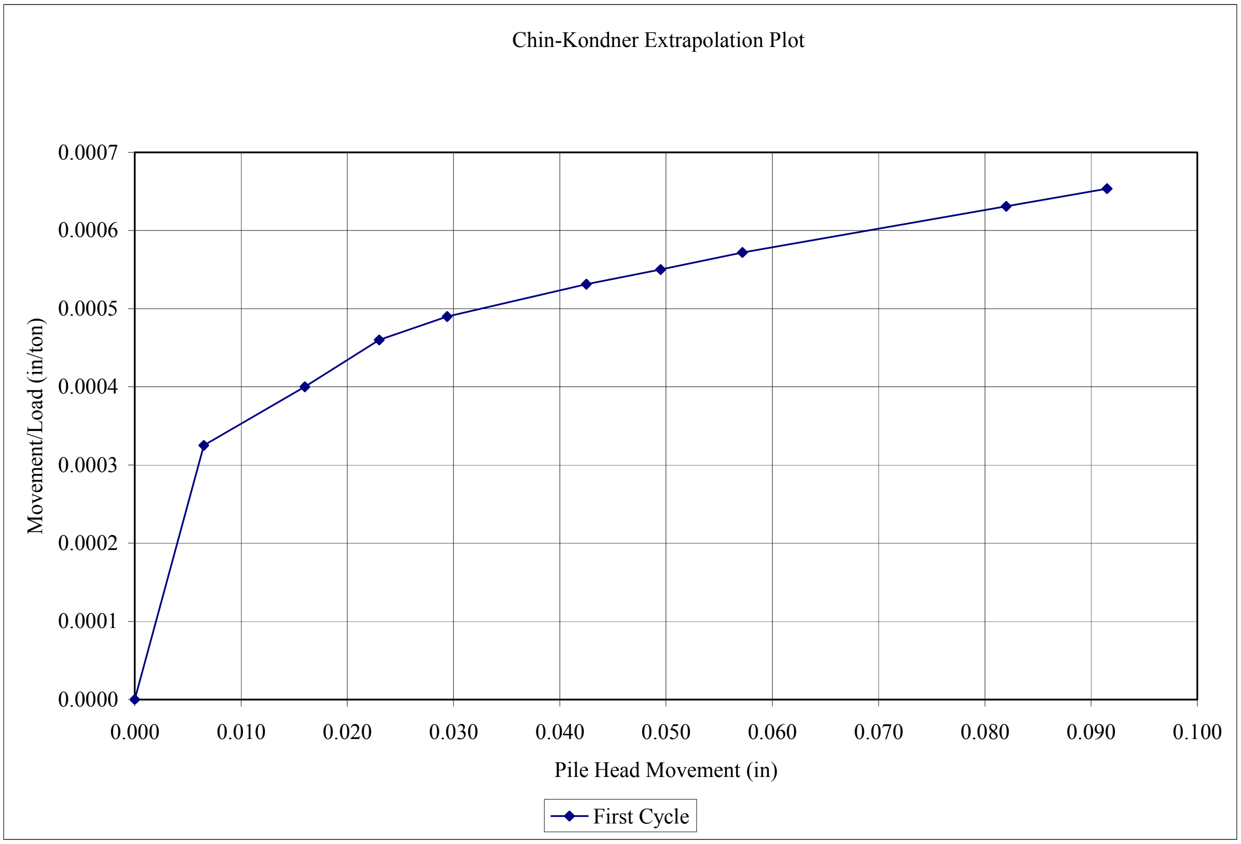


Figure 5: Chin-Kondner Plot

### 2.9.3 Chin-Kondner Extrapolation at Five Percent

As noted in section 2.6.2, the Five Percent method limits the allowable settlement to five percent of the actual pile diameter. In order to make use of this particular method, it is convenient to employ the Chin-Kondner Extrapolation and back solve for the ultimate load. This utilization of the Chin-Kondner method effectively reduces the capacity and will therefore reduce any inherent overstatement. The value of the ultimate load at five percent of the pile diameter is provided in Appendix B and may be calculated with the following relationship:

$$Q_{0.05} = \frac{0.05D}{0.05Dm + c} \qquad [2.33]$$

where

$D$ = the diameter of the pile in inches

$m$ = the slope of the second portion of the line

$c$ = the y-axis intercept

### 2.9.4 The Corps of Engineers Method

The Corps of Engineers (1991) method also makes use of plots to determine the ultimate capacity of a pile from load test data. This method utilizes graphical interpretation of a combination of three other methods. In this method the actual pile head movement in inches is shown on the vertical axis and the applied load in tons is shown on the horizontal axis. The curve resulting from the loading and subsequent unloading of the pile determines the shape of the plot. A line is then drawn from the point of one-quarter inch settlement on the vertical axis until it intersects the deflection curve. Then a vertical line is drawn from the point of intersection to the point of maximum loading on the horizontal axis. Likewise, a line is drawn from the point where the settlement curve exhibits a considerable change in slope to the corresponding load on the vertical axis. Similarly; the location that best identifies the point where the loading verses settlement plot has a slope of 0.01 inch per ton is noted and the corresponding load on the horizontal axis is determined. According to the Corps of Engineers 1991 manual:

> "…the average of the three loads determined in this manner would be considered the ultimate axial capacity of the pile. If one of these three procedures yields a value that

differs significantly from the other two, judgment should be used before including or excluding this value from the average. A suitable factor of safety should be applied to the resulting axial pile capacity."

Figure 6 shows and example of the Army Corps method where the ultimate load and settlement given by the procedure are approximately 134 tons and 0.27 inches respectively. The results of the Army Corps of Engineers method for each of the test piles evaluated are provided in Appendix B.

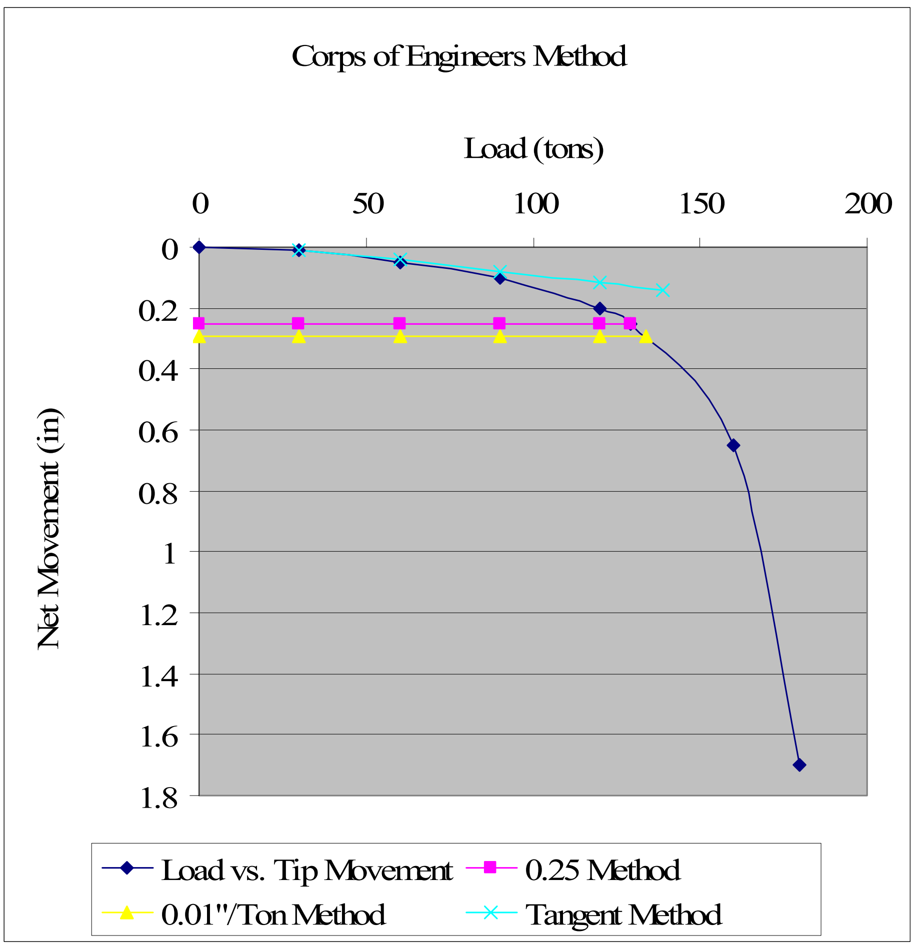


Figure 6: Army Corps Method Plot

# CHAPTER 3 METHODOLOGY

This chapter will discuss the overall methodology of the project from the load test interpretations to the actual analysis and the results. The geotechnical reports reviewed and the field measurements of the deflections resulting from the applied loads during the actual load tests were provided by MACTEC Engineering, Inc. on the condition that the site specific information would not be revealed.

The test piles investigated had diameters varying from 14 inches to 20 inches, and were installed in layered soils to depths varying from 38 feet to 98 feet. Each individual report was reviewed and the information was utilized to calculate anticipated pile capacity, and to predict anticipated pile settlement. The calculations for ultimate capacity are based upon several of the most common methods, including the NAVFAC, Meyerhof, Janbu, and Vesic'. Each of these methods is used to provide the end bearing capacity of the test piles. In addition, the NAVFAC method and the Coyle and Castillo method are utilized to calculate the skin friction capacity as described in the previous chapter. The actual percent of clay in the layered soils is provided in Table 1 as follows:

Table 1: Percent of Clay in Test Samples

| **Percent of Clay** | **Quantity** |
|---|---|
| 0-25 | 0 |
| 26-30 | 7 |
| 31-35 | 5 |
| 36-40 | 6 |
| 41-45 | 5 |
| >45 | 2 |
| Total | 25 |

## 3.1 Test Interpretation

The methods of interpretation utilized are the Davisson Offset Limit, the Chin-Kondner Extrapolation, the Five Percent method, and the Corps of Engineers method. The results for each of the test methods are compiled in spreadsheet format using *Microsoft Excel*. One note of interest is that in several cases the pile load tests did not continue to increase the applied load to actual failure of the pile. Rather, the piles were loaded to exceed the design load by 200% and then the test was terminated. This method is acceptable according to the ASTM standard; however, as shown in Figure 7, some of the tests do not provide an actual Davisson failure load $Q_f$, simply because the curve resulting from a plot of deflection of the pile head in inches verses the applied load in tons does not intercept the Davisson Offset line.

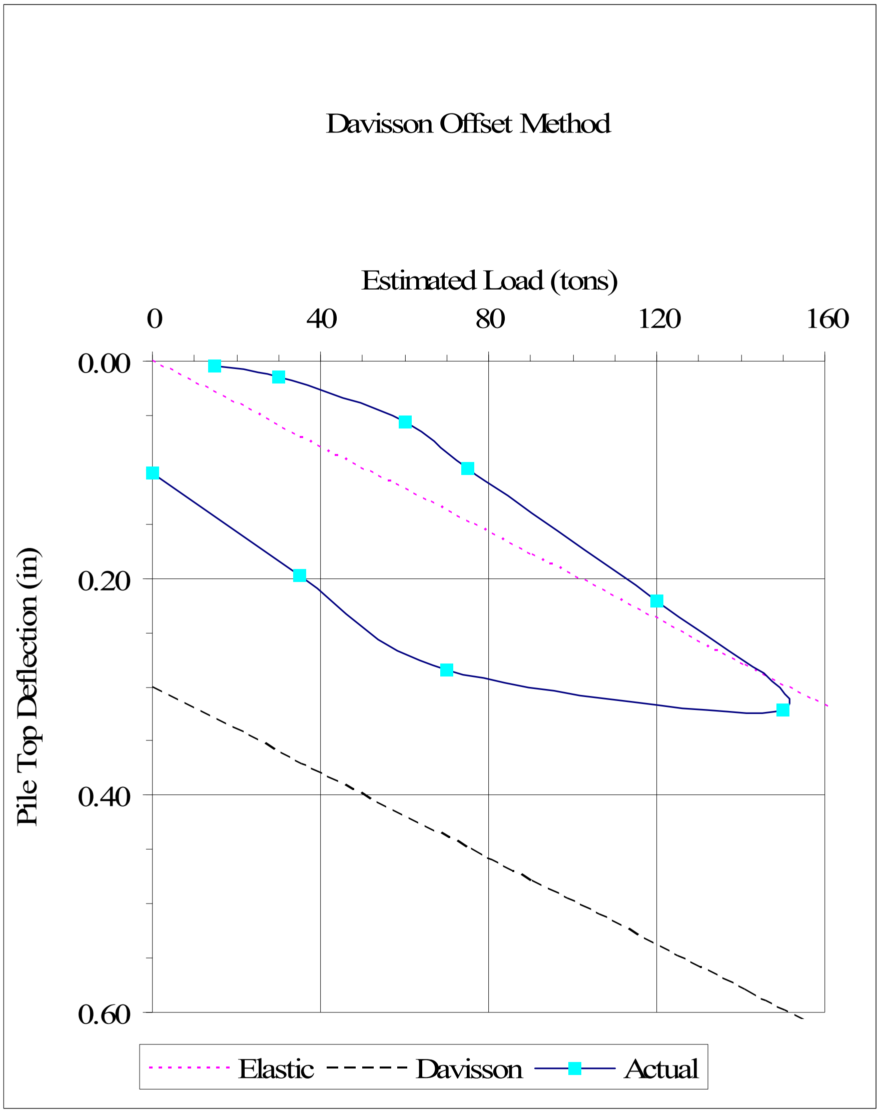


Figure 7: Load Test –vs.- Deflection Curve and Plot of Davisson Offset Line (Ref. TP-15)

In these cases the Davisson failure load is taken as the maximum load applied during the test. Likewise, in some cases the viability of the Corps of Engineers method is affected by the termination of the applied load prior to the point of failure or significant deflection. Figure 7 shows how the early termination of the increase in load does not provide for a complete loading verses deflection curve from which to draw a tangent. Furthermore, Figure 8 reveals that the plot of deflection verses load does not a curve that reaches a settlement rate of 0.01 inch per ton. Therefore, the resultant load at failure maybe significantly understated.

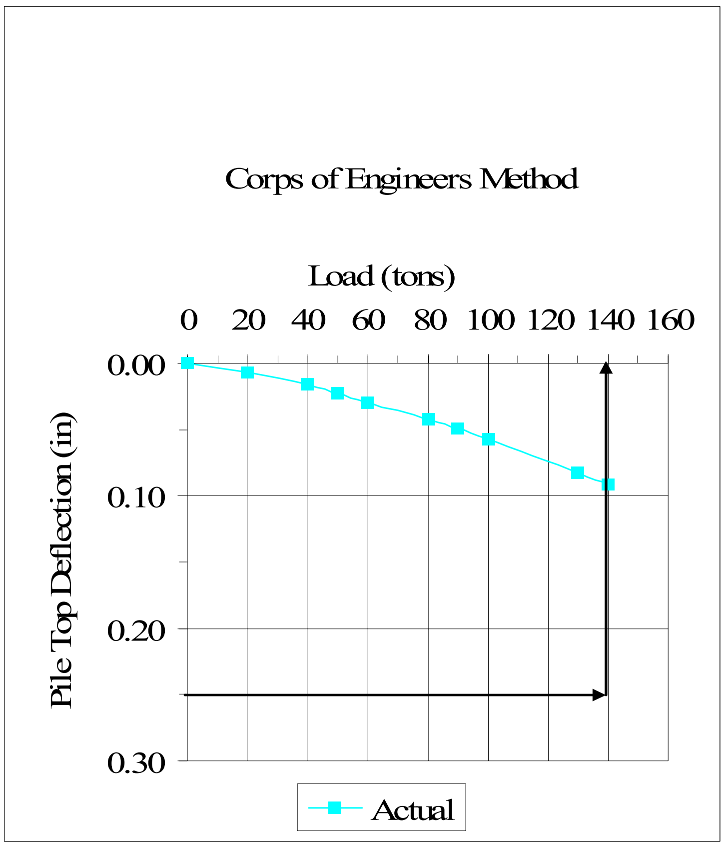


Figure 8: Load Test –vs.- Deflection Curve with Corps of Engineers Plot. (Ref. TP-4)

As indicated in the graph shown in Figure 8 above, the load test was not continued to the point where the pile deflected one-quarter of an inch. Likewise the test was terminated prior to the point of mobilization of the soil. In this case, attempting to utilize the tangent method or the 0.01 inch per ton criteria is not applicable. Consequently, in this example the predicted capacity provided by the Corps of Engineers method is limited to the applied load of 140 tons. Had the test continued with increasing load applications the resulting deflection curve would likely have resulted in a higher predicted capacity for the pile.

Finally, the Chin-Kondner Extrapolation method and the Chin-Kondner Extrapolation of the five percent method also presented challenges in a few cases as a result of the termination of the load test prior to the point of significant deflection of the pile. Since this method utilizes both deflection and load data to form a plot that is then extrapolated to determine a capacity, one can easily see that a limited data set will result in a capacity that has less significance.

## 3.2 Statistical Analysis Method

There are numerous ways to analyze and compare the data that has been generated during this study. The software utilized for the analysis is produced by a company called SPSS, Statistical Package for the Social Sciences, and an example of the SPSS output is provided in Appendix D. The statistical analysis program chosen to complete the analysis of the data is the chi-square method. With all types of analysis there are some limitations to the approach, and according to Snedecor and Cochran (1989) the most significant limitation to the correlational approach is that specific inferences can not be made. However, according to Garson (2006) the chi-square test may be used to determine if a sample of data came from a population with a specific distribution. Employing the chi-square test gives the analyst the ability to determine if a predictive relationship exists between each of the various load test analysis methods and the capacity calculation methods. In order to meet the requirements of this particular statistical test method the following conditions must exist:

- the sample must be random
- the sample must be large enough; typically at least 20 samples
- there must be a minimum of five cells
- the data must be independent
- the data must have like distributions
- the distributions must be given and may not be circular calculations
- the hypotheses must be non-directional
- the data must be finite, and grouped together
- their must be a normal distribution of deviations

The data used in the chi-square statistical analysis did meet the aforementioned requirements, and according to Miller and Freund (2000) the basic equation for the relevant chi-square statistic test is given as:

$$\chi^2 = \sum_{i=1}^{2}\sum_{j=1}^{k}\frac{(o_{ij} - e_{ij})^2}{e_{ij}} \qquad [3.1]$$

where

$o_{ij}$ = the observed frequency

$e_{ij}$ = the expected frequency or eta

$S^2$ is the variance of a random sample of size *n* taken from a normal population having a variance $\sigma^2$, and

$$\chi^2 = \frac{(n-1)S^2}{\sigma^2} = \sum_{i=1}^{n} (X_i - \overline{X})^2 \qquad [3.2]$$

is a random variable having the chi-square distribution with parameter $v = n-1$. [3.3]

In addition, the expected cell frequencies are given by the equation:

$$e_{ij} = \frac{(i^{th} row\_total) \times (j^{th} row\_total)}{grand\_total} \qquad [3.4]$$

Then the "observed frequencies, $o_{ij}$ and the expected frequencies $e_{ij}$ total the same for each row and column, such that only (*r*-1) (*c*-1) of the $e_{ij}$ have to be calculated directly, while the others can be obtained by subtraction from the appropriate row or column totals…the null hypothesis is rejected if the value of the statistic exceeds $\chi^2_\alpha$ for (*r*-1) (*c*-1) degrees of freedom." (Miller 2000). The basic premise of the chi-squared method is to establish a null hypothesis, $H_o$ and an alternate hypothesis, $H_i$ for each comparison to be performed. The null hypothesis, $H_o$ asserts that no difference exists between the data comparison, and the alternate hypothesis, $H_i$ asserts that a statistical difference does exists between the data comparison. Therefore, in Appendix E the null hypothesis and the alternate hypothesis are presented in statement form for each comparison of the analysis. The actual results of the chi-squared analysis are provided more clearly in Table 4.

The analysis also produces values for the significance and the eta of the correlation. When the analysis returns a significance value greater than 0.05 the indication is that no statistical difference exists between the comparisons based upon the parameters chosen for this study. In those cases the null hypothesis shall not be rejected. On the contrary, should the analysis return a significance value less than 0.05 the alternate hypothesis will be accepted. There is also a relationship between the numerical value returned for eta and the correlation of the variables compared as shown in the Table 2.

Table 2 Eta Significance

| **Significance of Eta Returned to Variables** | |
|---|---|
| *Captions*<br>*e*= 1 | Ideal Relationship |
| $0.9 \le e \le 0.99$ | Highly Significant Relationship |
| $0.7 \le e \le 0.89$ | Significant Relationship |
| $0.5 \le e \le 0.69$ | Moderate Relationship |
| $0.3 \le e \le 0.49$ | Moderately Small Relationship |
| $0.1 \le e \le 0.29$ | Small Relationship |
| $0 \le e \le 0.099$ | No Relationship |

In order to better visualize the results between the predicted capacity calculations and the load test interpretations methods, a graphical representation is provided in Appendix C. The four bar graphs in Appendix C illustrate the results of a comparison between the overall average calculated capacities of an individual pile capacity prediction method verses each of the four (4) test analysis methods. For example, the graphical presentations provided in Figure 61 represent the average capacity of the 25 test piles based upon Meyerhof's method compared to the load test averages for the Davisson Offset method, Chin-Kondner Extrapolation, Chin-Kondner Extrapolation of the five percent method, and the Army Corps of Engineers method. The bar graph shows that the Chin-Kondner Method returns results that average approximately 1.7 times greater than those calculated using Meyerhof's method. Both the Davisson Offset and Army Corps of Engineers method resulted in average capacities that were less than the value predicted Meyerhof's by approximately 16% and 37% respectively. Finally, the results of the average Chin-Kondner Extrapolation of the five percent method were approximately 8% greater than the average Meyerhof capacity.

## 3.3 Theoretical Pile Example

To enhance the understanding of the effect of cohesion surrounding a given pile, a theoretical example of an ACIP pile in a purely cohesive soil profile is provided. For this example capacities for a test pile in clay with various levels of cohesion are shown in Table 3.

Table 3: Theoretical Capacity in Clay

| **ACIP Pile in Clay** | | | | |
|---|---|---|---|---|
| **Cohesion** | | **Qp (t)** | **Qs (t)** | **Qu (t)** |
| 250 | (very soft) | 1.6 | 26 | 28 |
| 500 | (soft) | 3.0 | 50 | 53 |
| 1000 | (med. soft) | 4.7 | 79 | 84 |
| 2000 | (stiff) | 6.0 | 99 | 105 |
| 4000 | (very stiff) | 8.2 | 136 | 144 |

The theoretical ACIP pile in this example has a diameter of 16”, a length of 50 feet, and a water table located two feet below grade. The theoretical pile may be compared to test pile number three, (TP-3), which is an actual test pile of similar construction in layered soils. Test pile number three has two distinct stiff clay layers that make up approximately 31% of the total soil stratum, and the pile provides an average ultimate capacity of 191 tons. As shown in the example, for piles installed completely in clay, the majority of the capacity comes from the shaft as a result of the adhesion of the clay. The end bearing is almost negligible since the pile terminates in a comparatively weak medium. The addition of the sand layers noticeably increases the capacity of the pile. One can easily see that as the cohesion increases the capacity of the pile increases due to the increased adhesion along the pile shaft.

# CHAPTER 4: FINDINGS

This chapter provides the findings of the chi-squared statistical analysis preformed on the data obtained through the course of this study. As originally stated in Chapter 1, the primary objective is to compare the capacity of ACIP piles based upon compressive load equations and load test interpretation methods in order to determine if a significant difference exists, and to determine which method provides a best-fit correlation. Ideally, an empirical relationship will be drawn from the comparison between the anticipated capacity verses the results of the load test data.

The actual output of the SPSS program shown in Appendix D may not be clear to the reader. Therefore, Table 2 has been created with the output returned from the execution of the SPSS program in order to provide a more reader friendly presentation of the results. Table 4 shows the results of the individual predictive capacity method used compared to the four load test interpretation methods. The chi-square, level of significance, degrees of freedom, and eta-square value returned for each comparison is provided.

For example, the comparison between the Janbu predictive capacity calculation and the results of he Army Corps of Engineers method produced a chi-squared value of 50, a level of significance of 0.281, and an eta squared value of 0.637. The eta value indicates that there is a moderate relationship between the comparisons.

Table 4: Chi-square Results (N = 25)

| Method | Chi-square | Significance (2 tail) | % Free | Eta-square |
|---|---|---|---|---|
| NAVFAC vs. Davisson | 50 | 0.092 | 24 | 0.141 |
| NAVFAC vs. Chin-Kondner | 48 | 0.392 | 24 | 0.468 |
| NAVFAC vs. Chin-Kondner 5% | 46 | 0.431 | 24 | 0.138 |
| NAVFAC vs. Army Corps | 48 | 0.277 | 24 | 0.497 |
| Meyerhof vs.Davisson | 45.33 | 0.113 | 24 | 0.190 |
| Meyerhof vs.Chin-Kondner | 50 | 0.318 | 24 | 0.468 |
| Meyerhof vs.Chin-Kondner 5% | 48 | 0.352 | 24 | 0.121 |
| Meyerhof vs.Army Corps | 48 | 0.243 | 24 | 0.594 |
| Janbu vs.Davisson | 46.8 | 0.155 | 24 | 0.361 |
| Janbu vs.Chin-Kondner | 44 | 0.514 | 24 | 0.347 |
| Janbu vs.Chin-Kondner 5% | 44 | 0.472 | 24 | 0.120 |
| Janbu vs.Army Corps | 50 | 0.281 | 24 | 0.637 |
| Vesic' vs.Davisson | 48 | 0.087 | 24 | 0.048 |
| Vesic' vs.Chin-Kondner | 46 | 0.389 | 24 | 0.509 |
| Vesic' vs.Chin-Kondner 5% | 46 | 0.349 | 24 | 0.221 |
| Vesic' vs.Army Corps | 47.33 | 0.378 | 24 | 0.238 |

The chi-square statistical test produced the following results for each of the hypothesis statements that were analyzed:

a) NAVFAC vs. Davisson Offset method, $\chi^2$(24, n=25) = 50, $p$ = 0.092

b) NAVFAC vs. Chin-Kondner Extrapolation, $\chi^2$(24, n=25) = 46, $p$ = 0.392

c) NAVFAC vs. Chin-Kondner Extrapolation at 5%, $\chi^2$(24, n=25) = 44, $p$ = 0.431

d) NAVFAC vs. Army Corps method, $\chi^2$(24, n=25) = 48, $p$ = 0.277

e) Meyerhof vs. Davisson Offset method, $\chi^2$(24, n=25) = 45.33, $p$ = 0.113

f) Meyerhof vs. Chin-Kondner Extrapolation, $\chi^2$(24, n=25) = 50, $p$ = 0.318

g) Meyerhof vs. Chin-Kondner Extrapolation at 5%, $\chi^2$(24, n=25) = 48, $p$ = 0.352

h) Meyerhof vs. Army Corps method, $\chi^2$(24, n=25) = 45.33, $p$ = 0.243

i) Janbu vs. Davisson Offset method, $\chi^2$(24, n=25) = 48, $p$ = 0.155

j) Janbu vs. Chin-Kondner Extrpolation, $\chi^2$(24, n=25) = 50, $p$ = 0.514

k) Janbu vs. Chin-Kondner Extrapolation at 5%, $\chi^2$(24, n=25) = 46, $p$ = 0.472

l) Janbu vs. Army Corps method, $\chi^2$(24, n=25) = 50, $p$ = 0.281

m) Vesic' vs. Davisson Offset method, $\chi^2$(24, n=25) = 44.67, $p$ = 0.087

n) Vesic' vs. Chin-Kondner Extrapolation, $\chi^2$(24, n=25) = 48, $p$ = 0.389

o) Vesic' vs. Chin-Kondner Extrapolation at 5%, $\chi^2$(24, n=25) = 45.33, $p$ = 0.349

p) Vesic' vs. Army Corps method, $\chi^2$(24, n=25) = 46, $p$ = 0.378

where:

$\chi^2$ = chi-squared

(24, n=25) = (degrees of freedom, n=number of samples)

$p$ = significance of the value returned

An average of eta values indicating the best fit for each comparison is presented in Table 5 to provide a clearer representation of the statistical results returned by the SPSS software.

Table 5 Eta Average Values Returned

| **Comparison** | **Eta Average** |
|---|---|
| NAVFAC vs. Interpretation Methods | 0.311 |
| Meyerhof vs. Interpretation Methods | 0.343 |
| Janbu vs. Interpretation Methods | 0.366 |
| Vesic' vs. Interpretation Methods | 0.254 |

# CHAPTER 5: CONCLUSIONS

ACIP piles are commonly used as foundation elements in the construction of both buildings and transportation projects. With pile foundations of any type there are inherent uncertainties that force the prudent design engineer to seek information that can only come from actual testing. As originally stated, the primary goal of this study is to determine an empirical relationship between the predicted behavior of a given pile and the results of an actual load test. Therefore, the method of analysis chosen must determine if a correlation exist between the data, and if so which method provides the best correlation between predicted and actual behavior.

## 5.1 Analysis of Results

The statistical analysis first provides an answer to each of the 16 separate hypothesis statements. In each case the result of the comparison provided a significance term with a value greater than 0.05. Therefore, in each case the null hypothesis is accepted and the alternative hypothesis is rejected. Under the parameters established for the chi-square analysis for this study, the analysis indicates that there is no statistically significant difference between the four pile test analyses methods used to predict the pile behavior. Clearly the four methods utilized to predict pile capacity, utilized various methods to determine the anticipated pile behavior and return numbers that look diverse. However, the results are not statistically significant under the parameters of the analysis performed.

## 5.2 NAVFAC Method Results

The NAVFAC method is widely used for predicting the behavior of piles in granular, cohesive, and layered soils. The statistical analysis comparing the NAVFAC predicted behavior to the load test interpretation methods did not provide the best results overall. In fact, the resulting average eta value indicates that there is a moderately small relationship between the load test interpretation methods and the results of the NAVFAC method. As previously stated, the NAVFAC method sets the coefficient of lateral earth pressure at a set value of 0.7 for drilled piles, and the effective angle of friction at $0.75\phi'$. Furthermore, the value of $N_q^*$ is taken from a published table and the values are on the lower end of the scale when compared to the other methods.

## 5.3 Meyerhof Method Results

The chi-squared statistical analysis for this method also provided results with a moderately small relationship between the load test interpretation methods and the results of the Meyerhof method. As previously noted, the ultimate bearing capacity is made up of end bearing and skin friction. Meyerhof method provided the end bearing portion and the Castello and Coyle method provided the skin friction. Meyerhof's method provides a consistently larger value of $N_q^*$ than the other methods. In addition, the coefficient of lateral earth pressure is a calculated value rather ran a fixed of 0.7 for drilled piles and the effective angle of friction at $0.8\phi'$.

## 5.4 Janbu Method Results

This method returned the best overall correlation with an average eta value indicates that there is a moderately small relationship between the Janbu methods and the load test interpretation methods. The Castello and Coyle method provided the skin friction with calculated coefficients of lateral earth pressure and an effective angle of friction of $0.8\phi'$. Janbu's method produced the end bearing capacity, and his process of using an equation to calculate $N_q^*$ using the factor $\psi$ to adjust for the capacity of the bearing layer. Therefore, the analysis indicates that utilizing this combination to predict an ultimate capacity provides the best correlation to the load test interpretation methods analyzed.

## 5.5 Vessic' Method Results

The results of the analysis indicates that the worst fit correlation occurs when the four primary methods used to interpret load test data of a given pile are compared to the Vesic' method of predicting capacity. The eta value indicates that there is a small relationship between the predicted capacity and the load test analysis method. Since the skin friction portion of the ultimate capacity remains constant for Meyerhof, Janbu, and Vesic', the obvious difference is in the surcharge factor. Vesic' utilizes a different approach as shown in equations [2.22] through [2.24]. The value of $N_\sigma^*$ is determined through the use of a rigidity index and the end result is a factor that is normally less than Meyerhof's and greater than both the NAVFAC and Janbu coefficient.

## 5.6 Summation

One issue that led to discrepancies between the predicted behavior calculations and the results of the interpretation methods is that many of the load tests were terminated prior to full mobilization of the pile. Therefore, sufficient data points are not available to allow for the actual settlement curve to intercept the Davisson Offset line. The early load termination may be seen in Figure 13, where the load test was terminated at 200 tons with only 0.178 inches of measured tip deflection, and the initial Davisson Offset begins at 0.283 inches of deflection. In cases where the load test was terminated prior to reaching the Davisson Offset line, the ultimate compressive value is limited to the actual maximum load applied during the test. According to Crowther (1988) this method is “overly conservative.” Indeed, based upon the way the piles were loaded in this study, the allowable capacity would be significantly reduced if the resulting Davisson Offset deflection were to be utilized as the governing limitation. That would result in a factor of safety greater than twice the anticipated working load. While this is conservative and perhaps saves some time and effort in the field, it may not be the most economical solution and certainly does not provide for the best fit correlation.

One reason for discrepancies in the Chin-Kondner Extrapolation is that in some cases the pile head movement was minimal near the end of the load test as shown in Figure 58. In cases of this nature the slope of the resulting plot is relatively flat and the ultimate capacity may be significantly overstated. For example, figure 58 shows the Chin-Kondner Extrapolation plot of test pile 25. The plot produces an ultimate pile capacity of 637 tons which is approximately two (2) times greater than the average of all of the other pile capacity prediction and load test interpretation methods. As noted with the Davisson Offset method, the Chin-Kondner Extrapolation is one of the primary methods approved by the IBC 2003. However, a document data February 2005 describing proposed changes to the International Building Code states that a recent study by Duzceer & Saglamer (2002) indicates that Chin-Kondner Method "gives a substantially higher result than Davisson Method". In addition, that same document states:

> "…the correlation coefficient ("Correl") for Chin-Kondner is also very low, and that the coefficient of variation (COV) is very high, making reliability of the evaluation uncertain and increasing risk. Such a high prediction result for Chin-Kondner lowers the true effective safety factor and may result in serious serviceability problems."

Likewise according to a study published by the FHWA (2006):

> "…application of the Chin-Kondner method yields a failure load that is defined as the asymptotic ultimate load of the load-settlement curve. It therefore yields an upper limit for the failure load leading in practice to overestimating the ultimate load. However, if a distinct plunging ultimate load is not obtained in the test, the pile capacity or ultimate load is determined by considering a specific pile head movement, usually 2 to 10 percent of the diameter of the pile, or a given displacement, often 3.81 cm (1.5 inches)."

The writers of the ASTM D1143 are wise in making the statement that "a qualified geotechnical engineer should interpret the test results for predicting pile performance and capacity." In fact, perhaps Das (2007) explains the current problem best in popular text *Foundation Design,* where in the introduction to his chapter on pile foundations he states:

> “Although numerous investigations, both theoretical and experimental, have been conducted in the past to predict the behavior and load bearing capacity of piles in granular and cohesive soils, the mechanisms are not yet entirely understood and may never be.”

When his observation is extended to layered soil strata, the degree of misunderstanding can only be compounded. Likewise, for the tests reviewed as a part of this study the statistical analysis has led to the conclusion that an improved empirical relationship can not be determined with any degree of certainty; therefore, additional research is needed. Further study with an expanded sample size is recommended; ideally the expanded samples would include predominately cohesive material and the load tests would be continued until the pile is fully mobilized.

# APPENDIX A PILE TEST DATA

Table 6: Pile Test Data TP-1

| Test No.: | TP-1 |
|---|---|
| Design Load (Tons): | 50 |
| Pile Diameter (inches): | 14 |
| Pile Area (sq. inches): | 153.94 |
| Pile Length (ft): | 65.0 |
| Pile Compress. Strength (psi): | 4420.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 3634269 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| 0 | 0.0000 | 0.0000 | 0.267 | -- |
| 15 | 0.0004 | 0.0418 | 0.308 | 0.00003 |
| 30 | 0.0157 | 0.0837 | 0.350 | 0.00052 |
| 45 | 0.0333 | 0.1255 | 0.392 | 0.00074 |
| 55 | 0.0569 | 0.1534 | 0.420 | 0.00103 |
| 70 | 0.0901 | 0.1952 | 0.462 | 0.00129 |
| 85 | 0.1306 | 0.2370 | 0.504 | 0.00154 |
| 100 | 0.2000 | 0.2788 | 0.546 | 0.00200 |
| 75 | 0.1934 | 0.2091 | 0.476 | 0.00258 |
| 50 | 0.1645 | 0.1394 | 0.406 | 0.00329 |
| 25 | 0.1285 | 0.0697 | 0.336 | 0.00514 |
| 0 | 0.0668 | 0.0000 | 0.267 | #DIV/0! |
| 25 | 0.0989 | 0.0697 | 0.336 | 0.00396 |
| 50 | 0.1296 | 0.1394 | 0.406 | 0.00259 |
| 75 | 0.1689 | 0.2091 | 0.476 | 0.00225 |
| 100 | 0.2216 | 0.2788 | 0.546 | 0.00222 |
| 110 | 0.2500 | 0.3067 | 0.573 | 0.00227 |
| 130 | Rxn pile broke | 0.3625 | 0.629 | -- |

Figure 9: Davisson Offset Plot TP-1

Table 7: Soil Profile Data TP-1

| **Layer No.** | **Depth feet** | **Layer depth** | **Water Table** | **Strata Classification** | **N number** | **Friction $\phi$** | **Cohesion c** |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>3 | 3 | 3 | SC | 3 | 26 | 0 |
| 2 | 3<br>5 | 2 | | SP | 10 | 32 | 0 |
| 3 | 5<br>12 | 7 | | CH | 2 | 0 | 500 |
| 4 | 12<br>15 | 3 | | SC | 12 | 32 | 0 |
| 5 | 15<br>27 | 12 | | CH | 7 | 0 | 1500 |
| 6 | 27<br>30 | 3 | | SM | 10 | 33 | 0 |
| 7 | 30<br>35 | 5 | | CH | 7 | 0 | 1500 |
| 8 | 35<br>65 | 30 | | SP | 40 | 38 | |

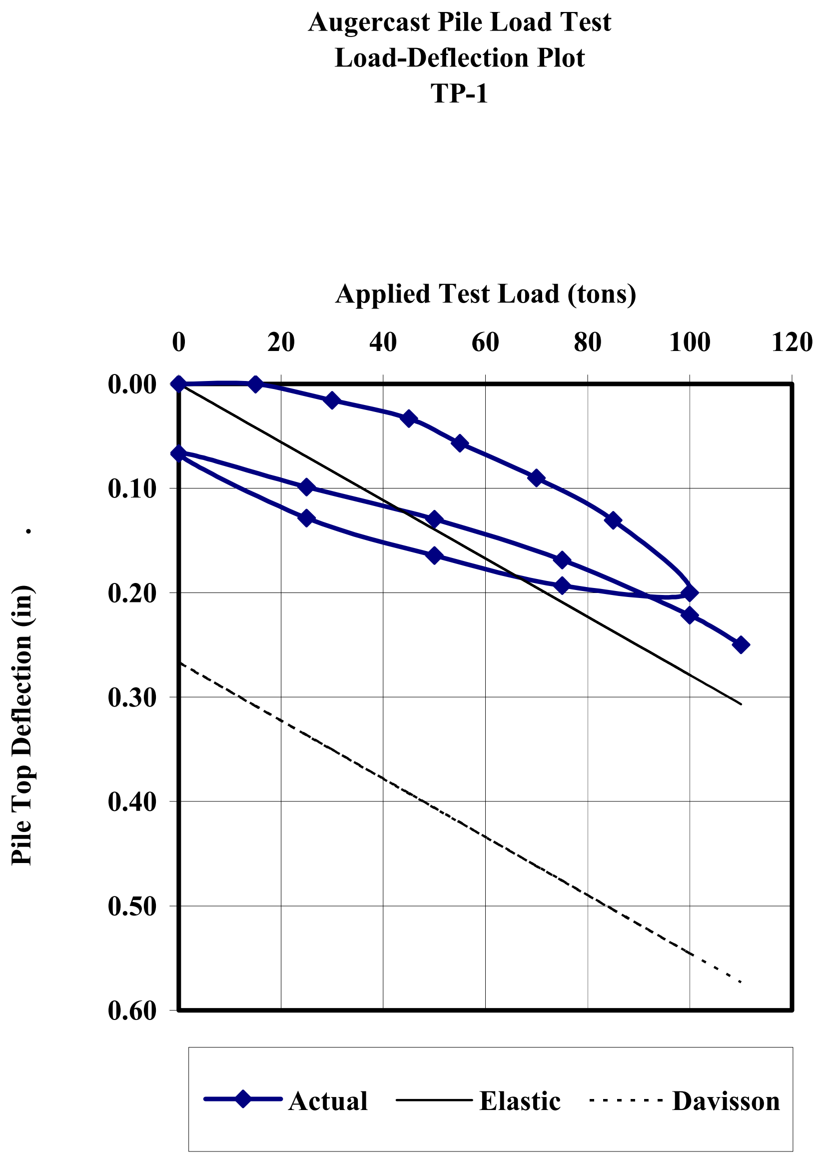


Figure 10: Davisson Offset Plot TP-1

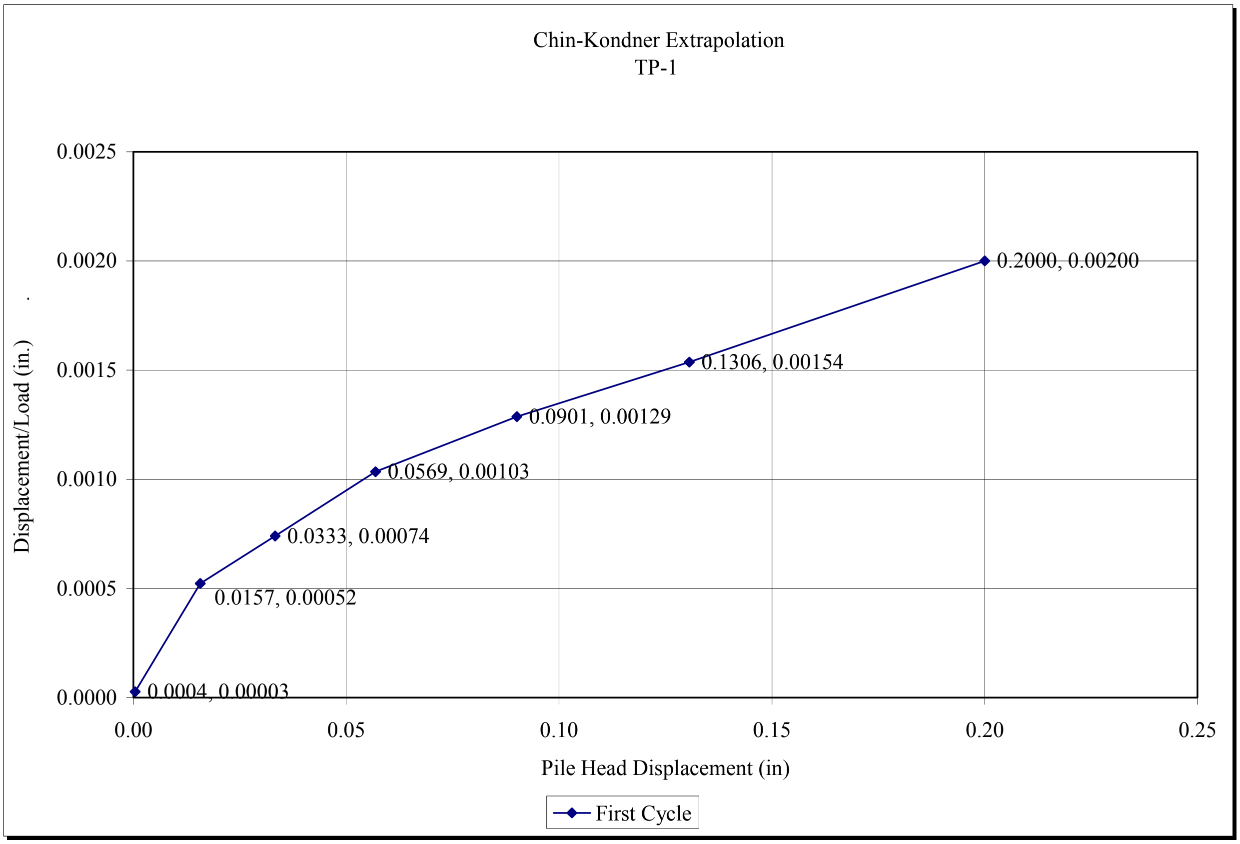


Figure 11: Chin-Kondner Plot TP-1

Table 8: Pile Test Data TP-2

| Test No.: | TP-2 |
|---|---|
| Design Load (Tons): | 35 |
| Pile Diameter (inches): | 16 |
| Pile Area (sq. inches): | 201.06 |
| Pile Length (ft): | 38.0 |
| Pile Compress. Strength (psi): | 4380.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 3617787 |

| Applied Test Load in Q (tons) | Average Measured Pile Top Deflection, D (inches) | Theoretical Pile Elastic Deflection in (inches) | Theoretical Davisson Anticipated Deflection in (inches) | Chin-Kondner Anticipated Deflection D/Q (in/ton) |
|---|---|---|---|---|
| 0 | 0.0000 | 0.0000 | 0.283 | -- |
| 15 | 0.0153 | 0.0188 | 0.302 | 0.00102 |
| 25 | 0.0324 | 0.0313 | 0.315 | 0.00130 |
| 35 | 0.0600 | 0.0439 | 0.327 | 0.00171 |
| 50 | 0.1414 | 0.0627 | 0.346 | 0.00283 |
| 60 | 0.1987 | 0.0752 | 0.359 | 0.00331 |
| 70 | 0.2500 | 0.0878 | 0.371 | 0.00357 |
| 89 | 0.3240 | 0.1116 | 0.395 | 0.00364 |
| 70 | 0.2824 | 0.0878 | 0.371 | 0.00403 |
| 50 | 0.2706 | 0.0627 | 0.346 | 0.00541 |
| 25 | 0.2549 | 0.0313 | 0.315 | 0.01020 |
| 0 | 0.2200 | 0.0000 | 0.283 | -- |

Table 9: Soil Profile Data TP-2

| Layer No. | Depth feet | Layer depth | Water Table | Strata Classification | Blow Count N | Friction $\phi$ | Cohesion c |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>2.5 | 2.5 | 2.5 | SP | 3 | 28 | 0 |
| 2 | 2.5<br>12 | 9.5 | | SP | 14 | 32 | 0 |
| 3 | 12<br>22 | 10 | | CH | 2 | 0 | 500 |
| 4 | 22<br>23 | 1 | | SC | 12 | 35 | 0 |
| 5 | 23<br>25 | 2 | | CH | 8 | 0 | 1700 |
| 6 | 25<br>32 | 7 | | SC | 11 | 32 | 0 |
| 7 | 32<br>36 | 4 | | CH | 8 | 0 | 1700 |
| 8 | 36<br>38 | 2 | | SP | 29 | 35 | 0 |

Augercast Pile Load Test
Load-Deflection Plot
TP-2

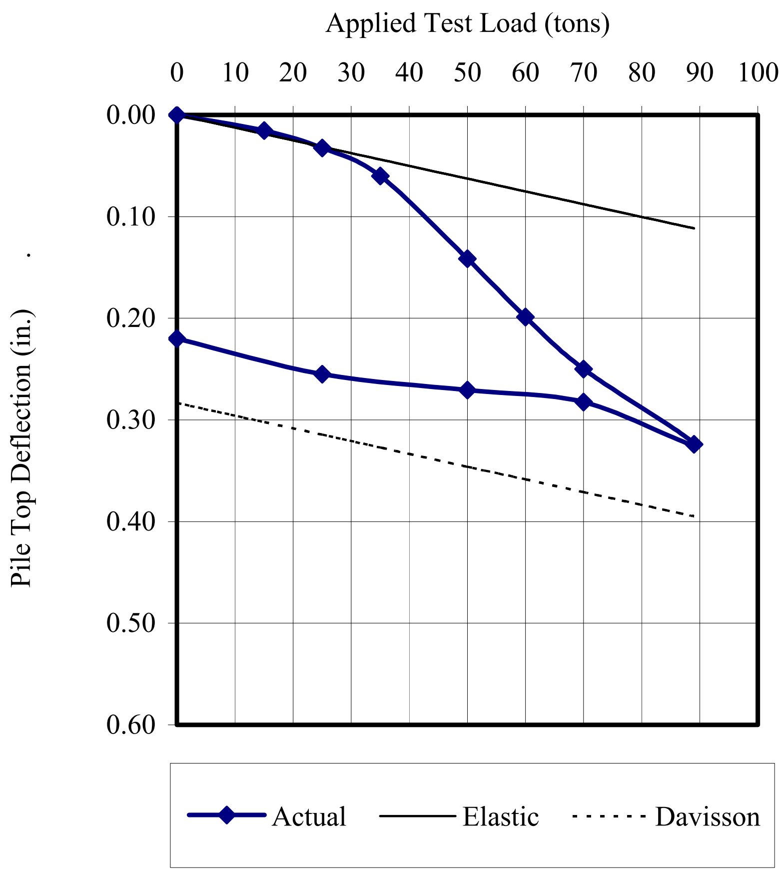


Figure 12: Davisson Offset Plot TP-2

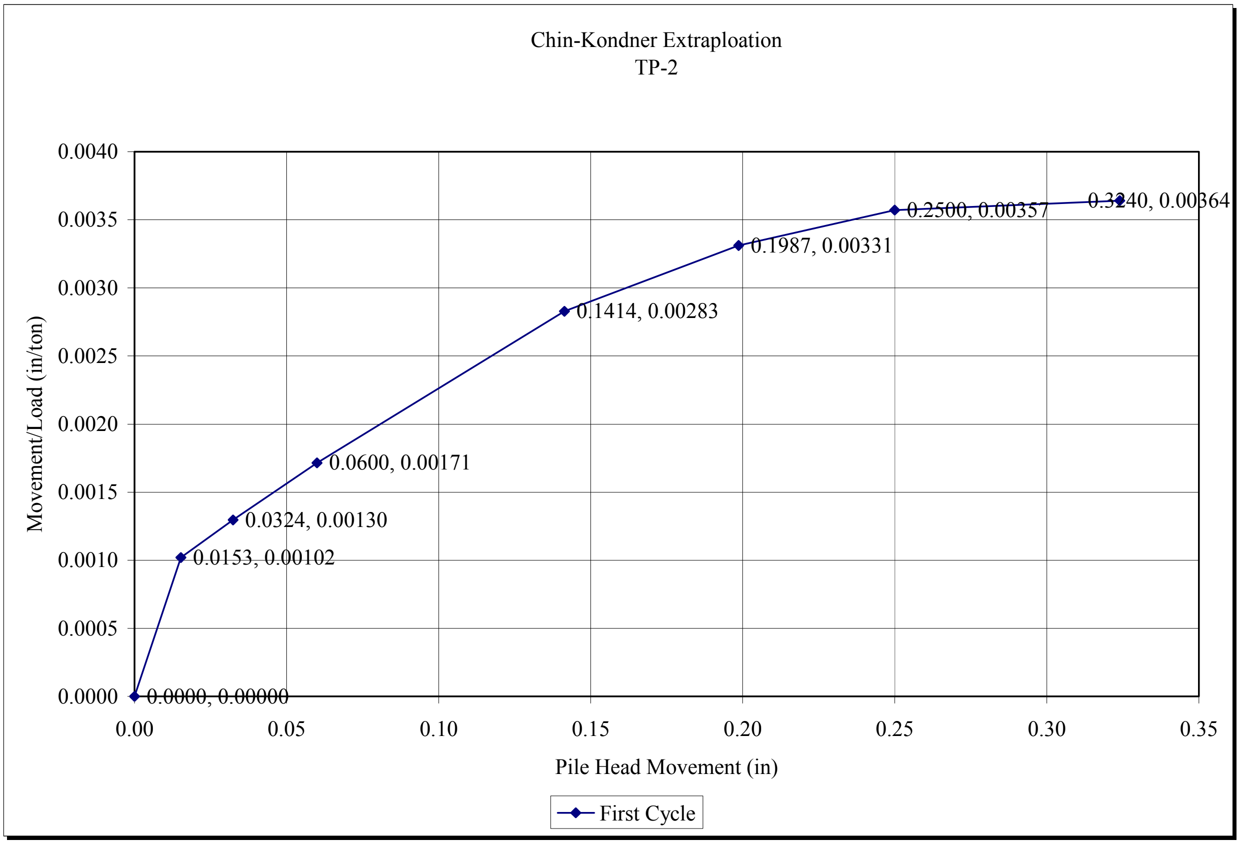


Figure 13: Chin-Kondner Plot TP-2

Table 10: Pile Test Data TP-3

| | |
|---|---|
| Test No.: | TP-3 |
| Design Load (Tons): | 50 |
| Pile Diameter (inches): | 16 |
| Pile Area (sq. inches): | 201.06 |
| Pile Length (ft): | 50.0 |
| Pile Compress. Strength (psi): | 5790.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 4159540 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| 0 | 0.0000 | 0.0000 | 0.283 | -- |
| 10 | 0.0015 | 0.0143 | 0.298 | 0.00015 |
| 30 | 0.0100 | 0.0430 | 0.326 | 0.00033 |
| 50 | 0.0225 | 0.0717 | 0.355 | 0.00045 |
| 70 | 0.0355 | 0.1004 | 0.384 | 0.00051 |
| 100 | 0.0565 | 0.1435 | 0.427 | 0.00057 |
| 120 | 0.0730 | 0.1722 | 0.456 | 0.00061 |
| 150 | 0.1005 | 0.2152 | 0.499 | 0.00067 |
| 170 | 0.1240 | 0.2439 | 0.527 | 0.00073 |
| 200 | 0.1760 | 0.2870 | 0.570 | 0.00088 |
| 150 | 0.1685 | 0.2152 | 0.499 | 0.00112 |
| 100 | 0.1475 | 0.1435 | 0.427 | 0.00148 |
| 50 | 0.1180 | 0.0717 | 0.355 | 0.00236 |
| 0 | 0.0660 | 0.0000 | 0.283 | -- |

Table 11: Soil Profile Data TP-3

| Layer No. | Depth feet | Layer depth | Water Table | Strata Classification | Blow Count N | Friction $\phi$ | Cohesion c |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>2 | 2 | 2 | SC | 17 | 33 | 0 |
| 2 | 2<br>9.5 | 7.5 | | CH | 6 | 0 | 1250 |
| 3 | 9.5<br>40 | 30.5 | | SP-SM | 15 | 30 | 0 |
| 4 | 40<br>48 | 8 | | CH | 2 | 0 | 2000 |
| 5 | 48<br>50 | 2 | | SP-SC | 49 | 38 | |

Augercast Pile Load Test
Load-Deflection Plot
TP-3

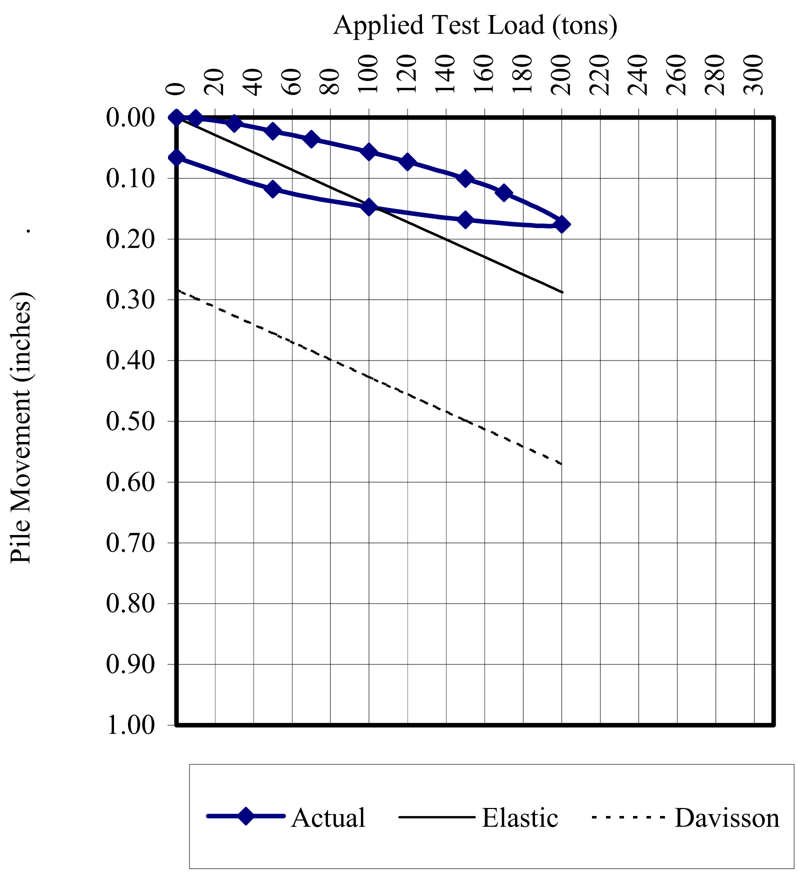


Figure 14: Davisson Offest Plot TP-3

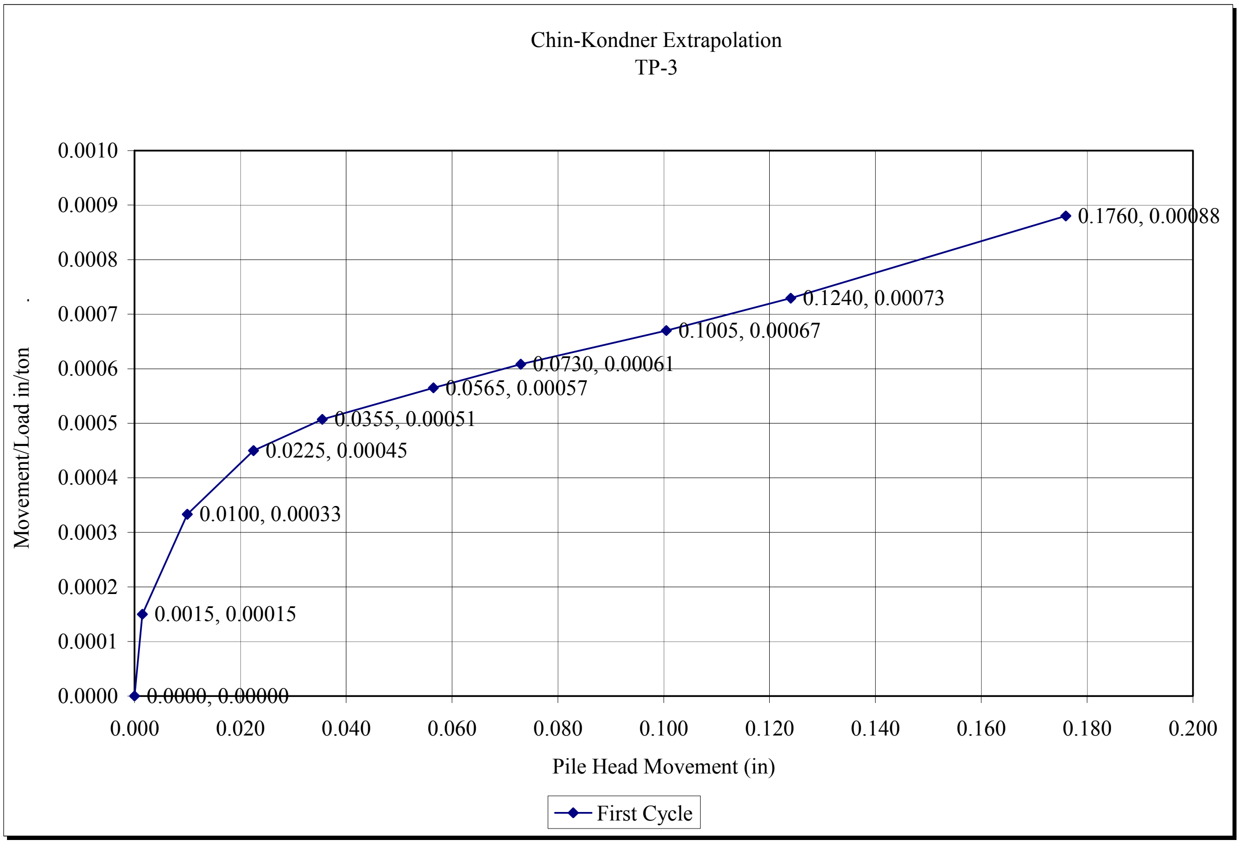


Figure 15: Chin-Kondner Plot TP-3

Table 12: Pile Test Data TP-4

| **Test No.:** | **TP-4** |
|---|---|
| Design Load (Tons): | 50 |
| Pile Diameter (inches): | 16 |
| Pile Area (sq. inches): | 201.06 |
| Pile Length (ft): | 30.0 |
| Pile Compress. Strength (psi): | 5790.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 4159540 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| 0 | 0.0000 | 0.0000 | 0.283 | -- |
| 20 | 0.0065 | 0.0172 | 0.301 | 0.00033 |
| 40 | 0.0160 | 0.0344 | 0.318 | 0.00040 |
| 50 | 0.0230 | 0.0430 | 0.326 | 0.00046 |
| 60 | 0.0294 | 0.0517 | 0.335 | 0.00049 |
| 80 | 0.0425 | 0.0689 | 0.352 | 0.00053 |
| 90 | 0.0495 | 0.0775 | 0.361 | 0.00055 |
| 100 | 0.0572 | 0.0861 | 0.369 | 0.00057 |
| 130 | 0.0820 | 0.1119 | 0.395 | 0.00063 |
| 140 | 0.0915 | 0.1205 | 0.404 | 0.00065 |
| 100 | 0.0800 | 0.0861 | 0.369 | 0.00080 |
| 50 | 0.0497 | 0.0430 | 0.326 | 0.00099 |
| 0 | 0.0201 | 0.0000 | 0.283 | -- |

Table 13: Soil Profile Data TP-4

| **Layer No.** | **Depth feet** | **Layer depth** | **Water Table** | **Strata Classification** | **Blow Count N** | **Friction $\phi$** | **Cohesion c** |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>1.5 | 1.5 | 1.5 | SP | 40 | 35 | 0 |
| 2 | 1.5<br>5 | 3.5 | | SP | 15 | 30 | 0 |
| 3 | 5<br>19 | 14 | | CH | 7 | 0 | 1500 |
| 4 | 19<br>47 | 28 | | SP | 25 | 35 | 0 |
| 5 | 47<br>60 | 13 | | CH | 4 | 0 | 840 |
| 6 | 60<br>75 | 15 | | SP-SC | 39 | 36 | 2000 |

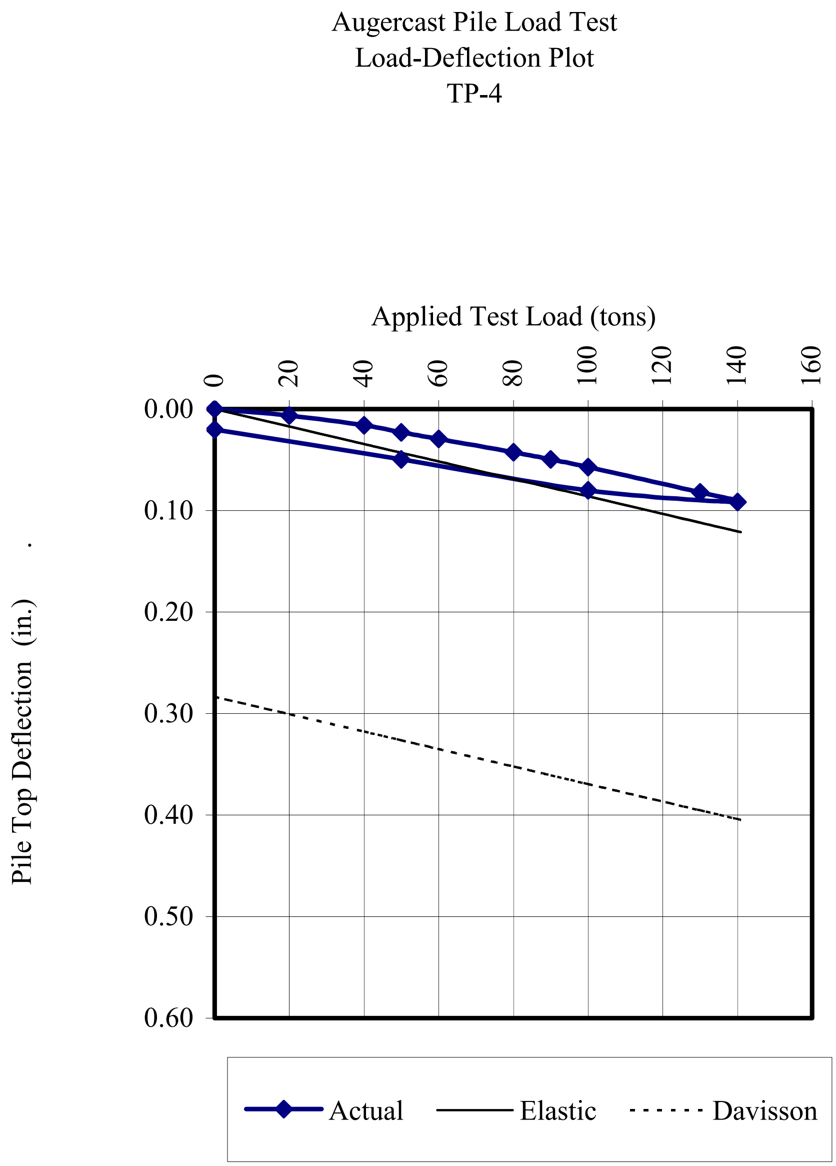


Figure 16: Davisson Offset Plot TP-4

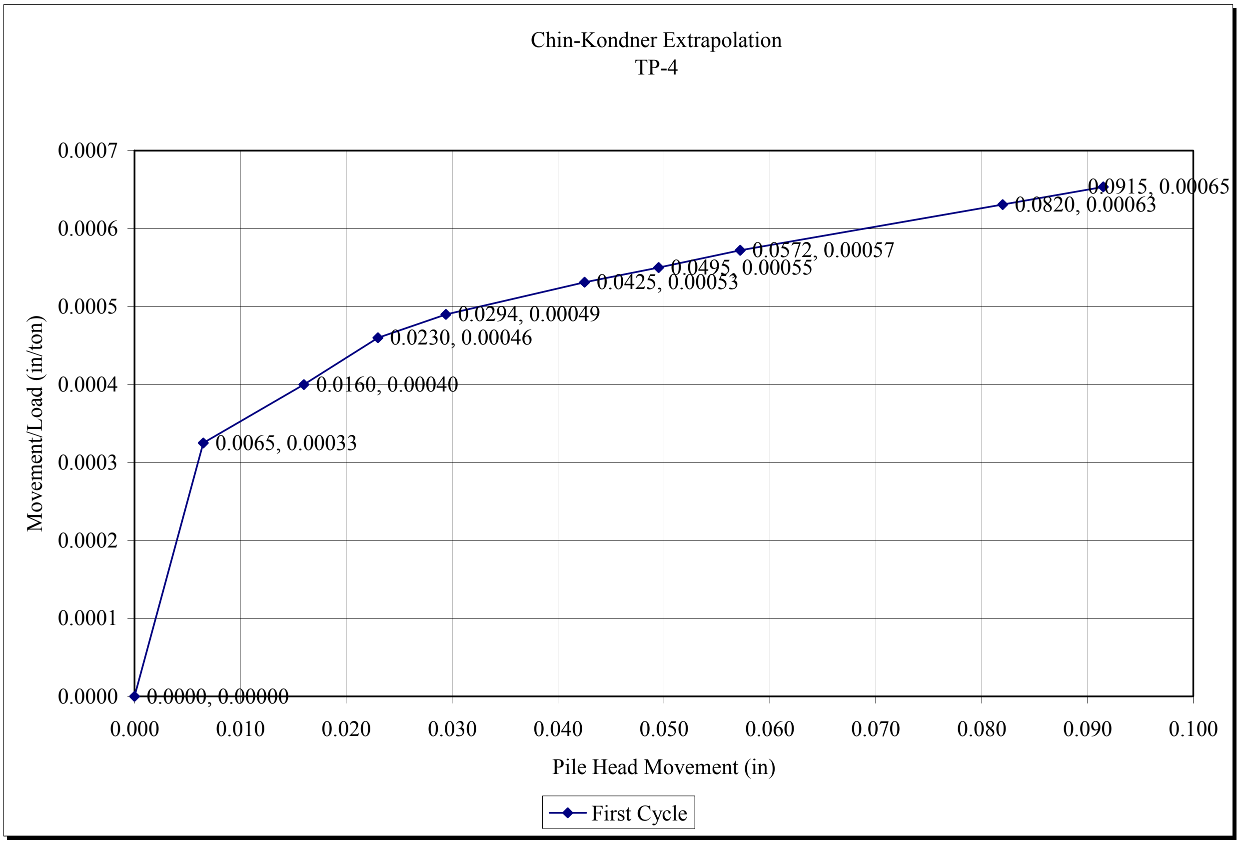


Figure 17: Chin-Kondner Plot TP-4

Table 14 Pile Test Data TP-5

| Test No.: | TP-5 |
|---|---|
| Design Load (Tons): | 60 |
| Pile Diameter (inches): | 14 |
| Pile Area (sq. inches): | 153.94 |
| Pile Length (ft): | 58.0 |
| Pile Compress. Strength (psi): | 5750.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 4145147 |

| Applied Test Load in Q (tons) | Average Measured Pile Top Deflection, D (inches) | Theoretical Pile Elastic Deflection in (inches) | Theoretical Davisson Anticipated Deflection in (inches) | Chin-Kondner Anticipated Deflection D/Q (in/ton) |
|---|---|---|---|---|
| 0 | 0.0000 | 0.0000 | 0.267 | -- |
| 30 | 0.0230 | 0.0654 | 0.332 | 0.00077 |
| 60 | 0.0630 | 0.1309 | 0.398 | 0.00105 |
| 90 | 0.1340 | 0.1963 | 0.463 | 0.00149 |
| 120 | 0.2600 | 0.2618 | 0.528 | 0.00217 |
| 150 | 0.3840 | 0.3272 | 0.594 | 0.00256 |
| 200 | 0.6250 | 0.4363 | 0.703 | 0.00313 |
| 230 | 0.8000 | 0.5017 | 0.768 | 0.00348 |

Table 15: Soil Profile Data TP-5

| Layer No. | Depth feet | Layer depth | Water Table | Strata Classification | Blow Count N | Friction $\phi$ | Cohesion c |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>5 | 5 | 5 | SP | 39 | 36 | 0 |
| 2 | 5<br>20 | 15 | | SP | 30 | 33 | 0 |
| 3 | 20<br>24 | 4 | | CH | 7 | 0 | 1500 |
| 4 | 24<br>47 | 23 | | SP | 29 | 36 | 0 |
| 5 | 47<br>58 | 11 | | CH | 12 | 0 | 2500 |

Augercast Pile Load Test
Load-Deflection Plot
TP-5

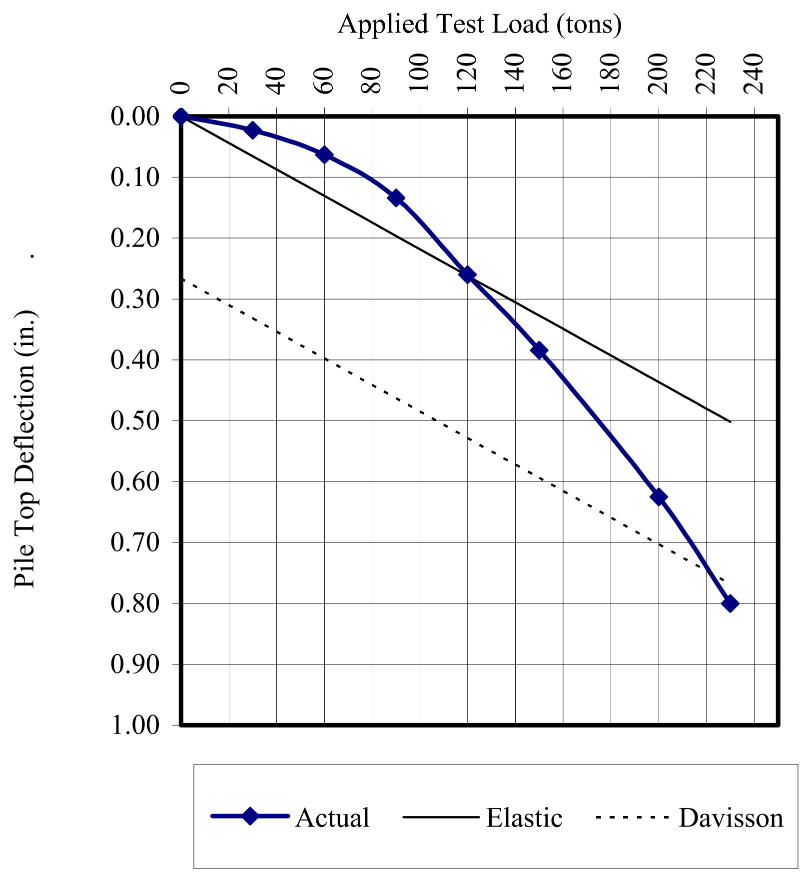


Figure 18: Davisson Offset Plot TP-5

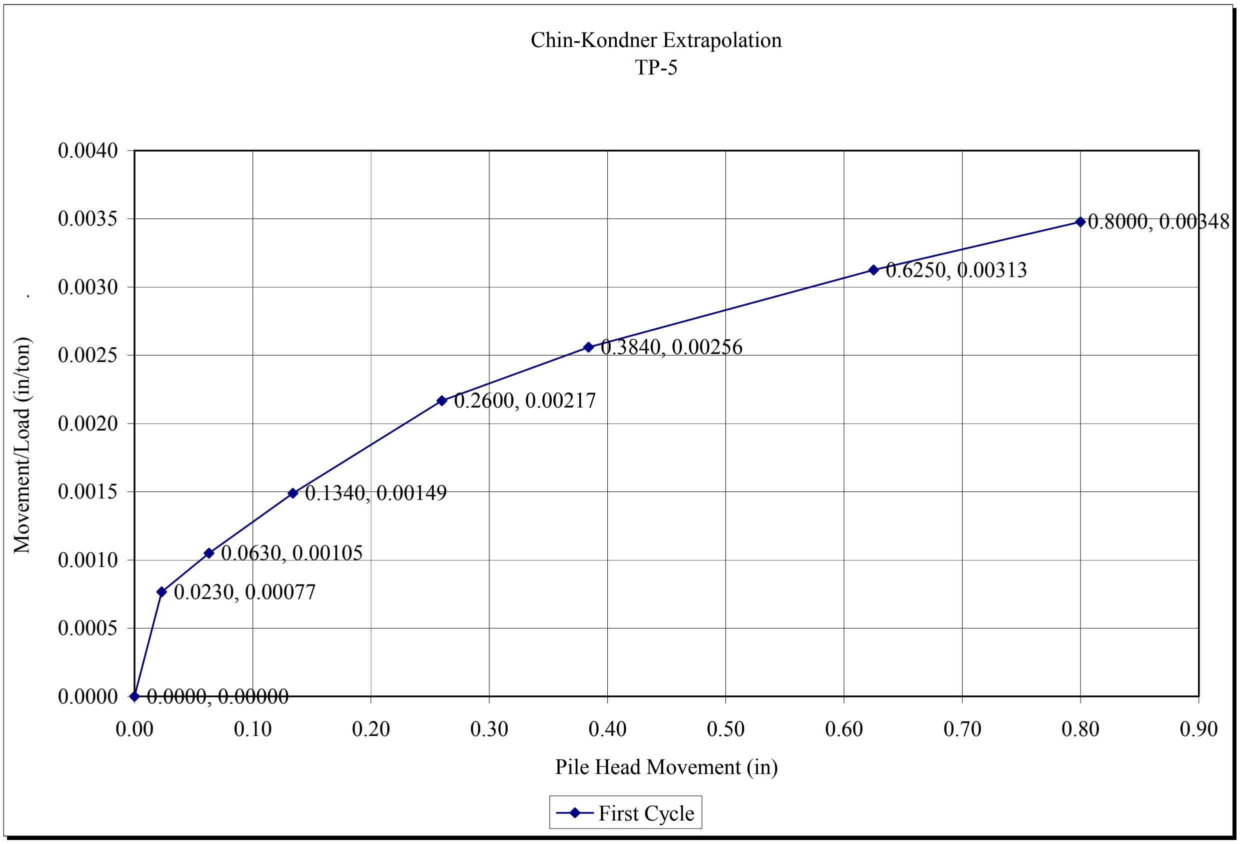


Figure 19: Chin-Kondner Extrapolation TP-5

Table 16: Pile Test Data TP-6

| Test No.: | TP-6 |
|---|---|
| Design Load (Tons): | 60 |
| Pile Diameter (inches): | 14 |
| Pile Area (sq. inches): | 153.94 |
| Pile Length (ft): | 51.0 |
| Pile Compress. Strength (psi): | 5750.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 4145147 |

| Applied Test Load in Q (tons) | Average Measured Pile Top Deflection, D (inches) | Theoretical Pile Elastic Deflection in (inches) | Theoretical Davisson Anticipated Deflection in (inches) | Chin-Kondner Anticipated Deflection D/Q (in/ton) |
|---|---|---|---|---|
| 0 | 0.0000 | 0.0000 | 0.267 | -- |
| 30 | 0.0187 | 0.0575 | 0.324 | 0.00062 |
| 60 | 0.0540 | 0.1151 | 0.382 | 0.00090 |
| 90 | 0.1100 | 0.1726 | 0.439 | 0.00122 |
| 120 | 0.2270 | 0.2302 | 0.497 | 0.00189 |
| 150 | 0.2940 | 0.2877 | 0.554 | 0.00196 |
| 100 | 0.2120 | 0.1918 | 0.458 | 0.00212 |
| 50 | 0.1240 | 0.0959 | 0.363 | 0.00248 |
| 0 | 0.0154 | 0.0000 | 0.267 | -- |

Table 17: Soil Profile Data TP-6

| Layer No. | Depth feet | Layer depth | Water Table | Strata Classification | Blow Count N | Friction $\phi$ | Cohesion c |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>3 | 3 | 3 | SP | 39 | 36 | 0 |
| 2 | 3<br>15 | 12 | | SP | 30 | 34 | 0 |
| 3 | 15<br>21 | 6 | | CH | 7 | 0 | 1500 |
| 4 | 21<br>36 | 15 | | SP | 29 | 31 | 0 |
| 5 | 36<br>51 | 15 | | CH | 12 | 0 | 2500 |

Augercast Pile Load Test
Load-Deflection Plot
TP-6

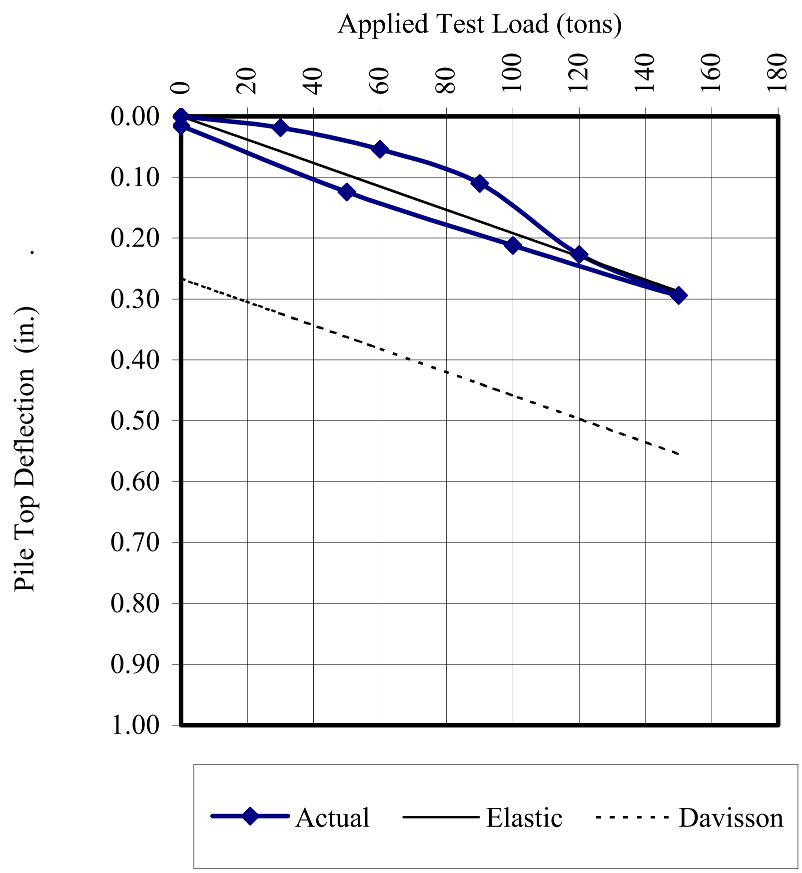


Figure 20: Davisson Offset Plot TP-6

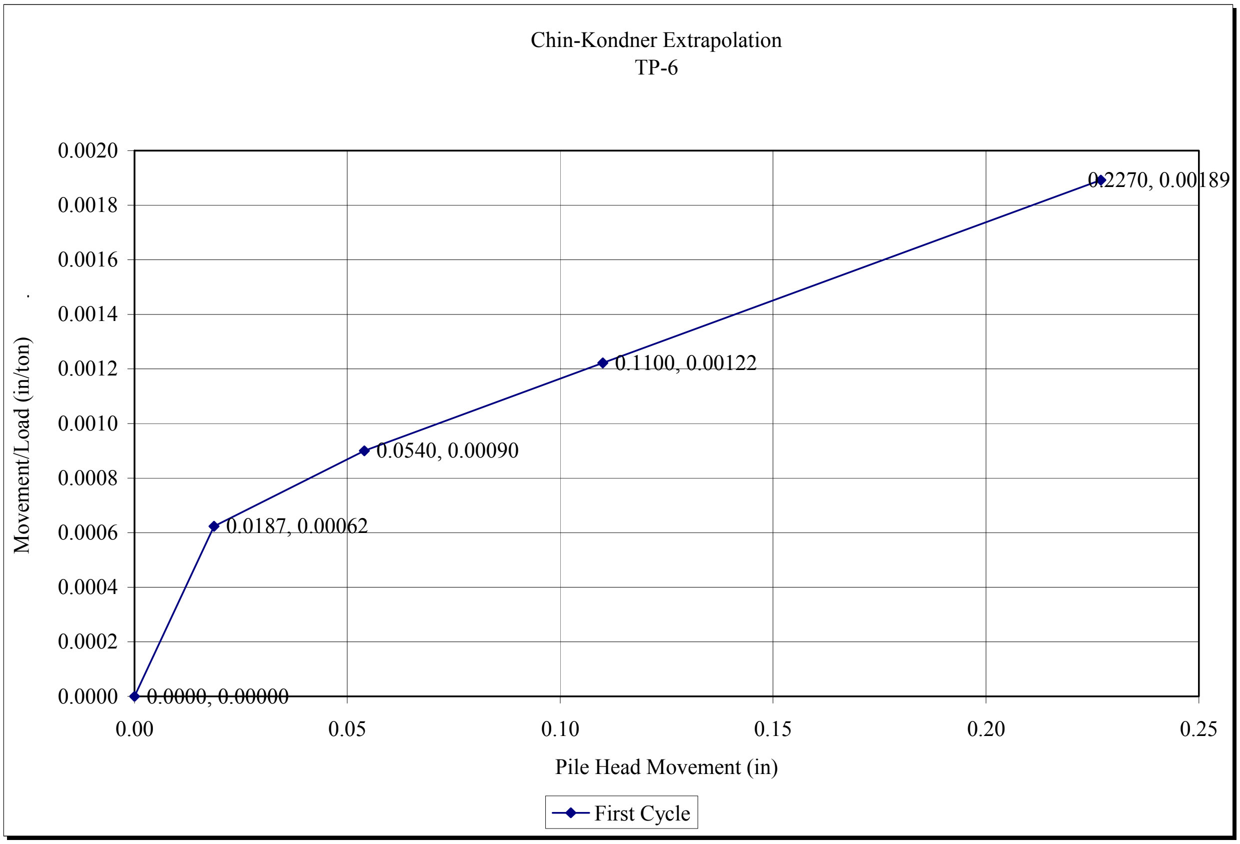


Figure 21: Chin-Kondner Extrapolation TP-6

Table 18: Pile Test Data TP-7

| Test No.: | TP-7 |
|---|---|
| Design Load (Tons): | 200 |
| Pile Diameter (inches): | 18 |
| Pile Area (sq. inches): | 254.47 |
| Pile Length (ft): | 95.0 |
| Pile Compress. Strength (psi): | 6000.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 4234300 |

| Applied Test Load in Q (tons) | Average Measured Pile Top Deflection, D (inches) | Theoretical Pile Elastic Deflection in (inches) | Theoretical Davisson Anticipated Deflection in (inches) | Chin-Kondner Anticipated Deflection D/Q (in/ton) |
|---|---|---|---|---|
| 0 | 0.0000 | 0.0000 | 0.300 | -- |
| 29.5 | 0.0060 | 0.0624 | 0.362 | 0.00020 |
| 55.5 | 0.0320 | 0.1174 | 0.417 | 0.00058 |
| 84.5 | 0.0700 | 0.1788 | 0.479 | 0.00083 |
| 116.5 | 0.1120 | 0.2465 | 0.547 | 0.00096 |
| 147 | 0.1600 | 0.3111 | 0.611 | 0.00109 |
| 170.5 | 0.1960 | 0.3608 | 0.661 | 0.00115 |
| 220 | 0.2760 | 0.4655 | 0.766 | 0.00125 |
| 269.5 | 0.3510 | 0.5703 | 0.870 | 0.00130 |
| 313.5 | 0.4260 | 0.6634 | 0.963 | 0.00136 |
| 353.5 | 0.4900 | 0.7480 | 1.048 | 0.00139 |
| 400 | 0.5690 | 0.8464 | 1.146 | 0.00142 |
| 358.5 | 0.5570 | 0.7586 | 1.059 | 0.00155 |
| 242.5 | 0.4540 | 0.5131 | 0.813 | 0.00187 |
| 125.5 | 0.2940 | 0.2656 | 0.566 | 0.00234 |
| 0 | 0.0060 | 0.0000 | 0.300 | -- |

Table 19: Soil Profile Data TP-7

| **Layer No.** | **Depth feet** | **Layer depth** | **Water Table** | **Strata Classification** | **Blow Count N** | **Friction $\phi$** | **Cohesion c** |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>1 | 1 | 1 | SP | 10 | 30 | 0 |
| 2 | 1<br>21 | 20 | | SM | 30 | 36 | 0 |
| 3 | 21<br>53 | 32 | | SC | 27 | 33 | 0 |
| 4 | 53<br>93 | 40 | | CH | 8 | 0 | 1700 |
| 5 | 93<br>95 | 2 | | SP | 40 | 35 | 0 |

Augercast Pile Load Test
Load-Deflection Plot
TP-7

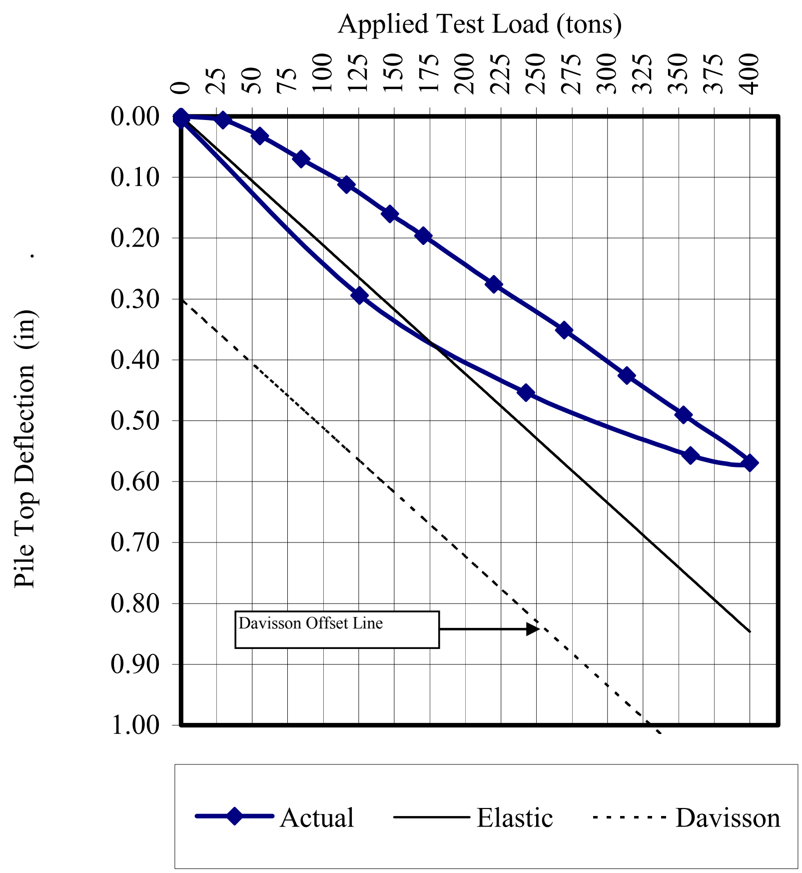


Figure 22: Davisson Offset Plot TP-7

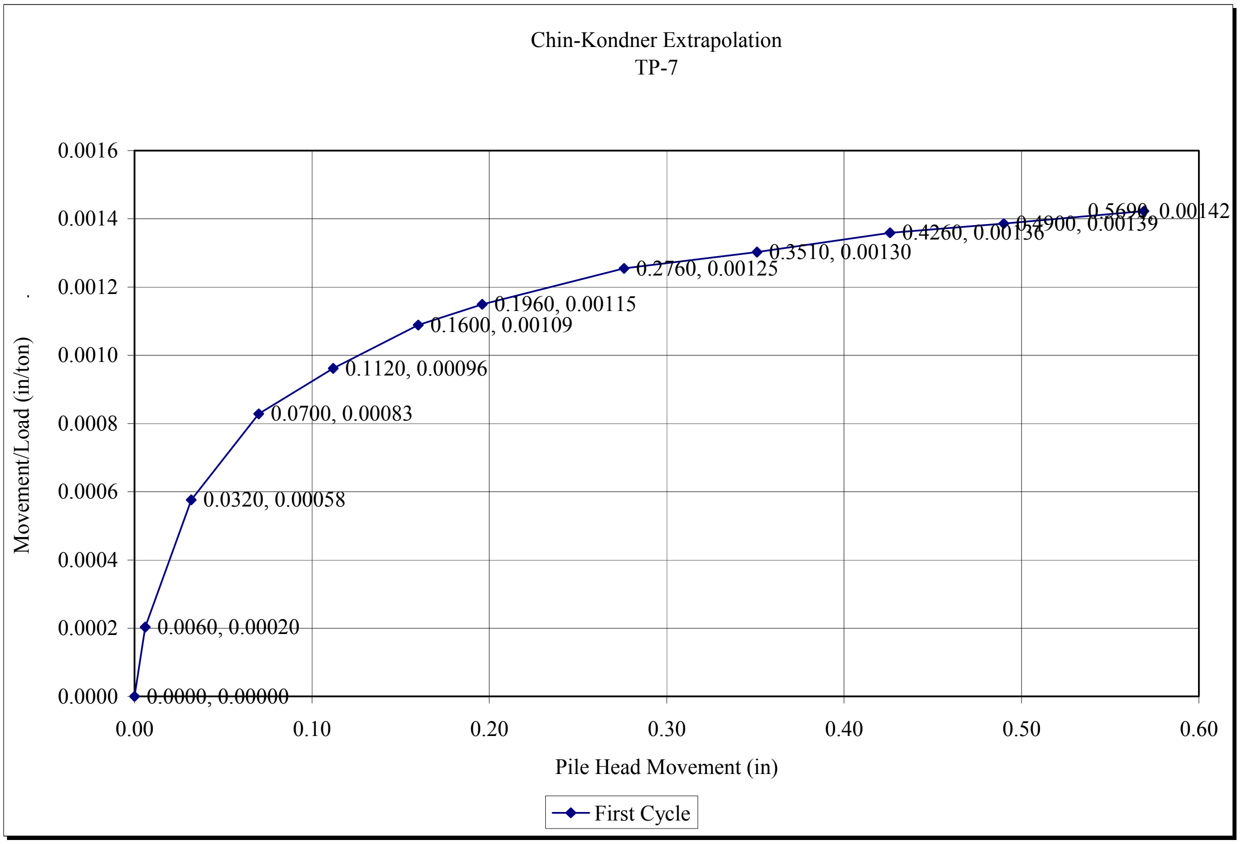


Figure 23: Chin-Kondner Extrapolation TP-7

Table 20: Pile Test Data TP-8

| **Test No.:** | **TP-8** |
|---|---|
| Design Load (Tons): | 130 |
| Pile Diameter (inches): | 14 |
| Pile Area (sq. inches): | 153.94 |
| Pile Length (ft): | 98.0 |
| Pile Compress. Strength (psi): | 6000.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 4234300 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| 0 | 0.0000 | 0.0000 | 0.267 | -- |
| 16.5 | 0.0000 | 0.0595 | 0.326 | 0.00000 |
| 33 | 0.0000 | 0.1191 | 0.386 | 0.00000 |
| 86.5 | 0.1360 | 0.3121 | 0.579 | 0.00157 |
| 137 | 0.2500 | 0.4943 | 0.761 | 0.00182 |
| 210 | 0.4000 | 0.7578 | 1.024 | 0.00190 |
| 260.5 | 0.5000 | 0.9400 | 1.207 | 0.00192 |
| 194 | 0.4060 | 0.7000 | 0.967 | 0.00209 |
| 134 | 0.3440 | 0.4835 | 0.750 | 0.00257 |
| 67 | 0.2500 | 0.2418 | 0.508 | 0.00373 |
| 0 | 0.0940 | 0.0000 | 0.267 | -- |

Table 21: Soil Profile Data TP-8

| **Layer No.** | **Depth feet** | **Layer depth** | **Water Table** | **Strata Classification** | **Blow Count N** | **Friction $\phi$** | **Cohesion c** |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>5 | 5 | 5 | SP | 10 | 30 | 0 |
| 2 | 5<br>22 | 17 | | SM | 30 | 34 | 0 |
| 3 | 22<br>57 | 35 | | SC | 27 | 33 | 0 |
| 4 | 57<br>93 | 36 | | CH | 7 | 0 | 1500 |
| 5 | 93<br>98 | 5 | | SP | 40 | 38 | 0 |

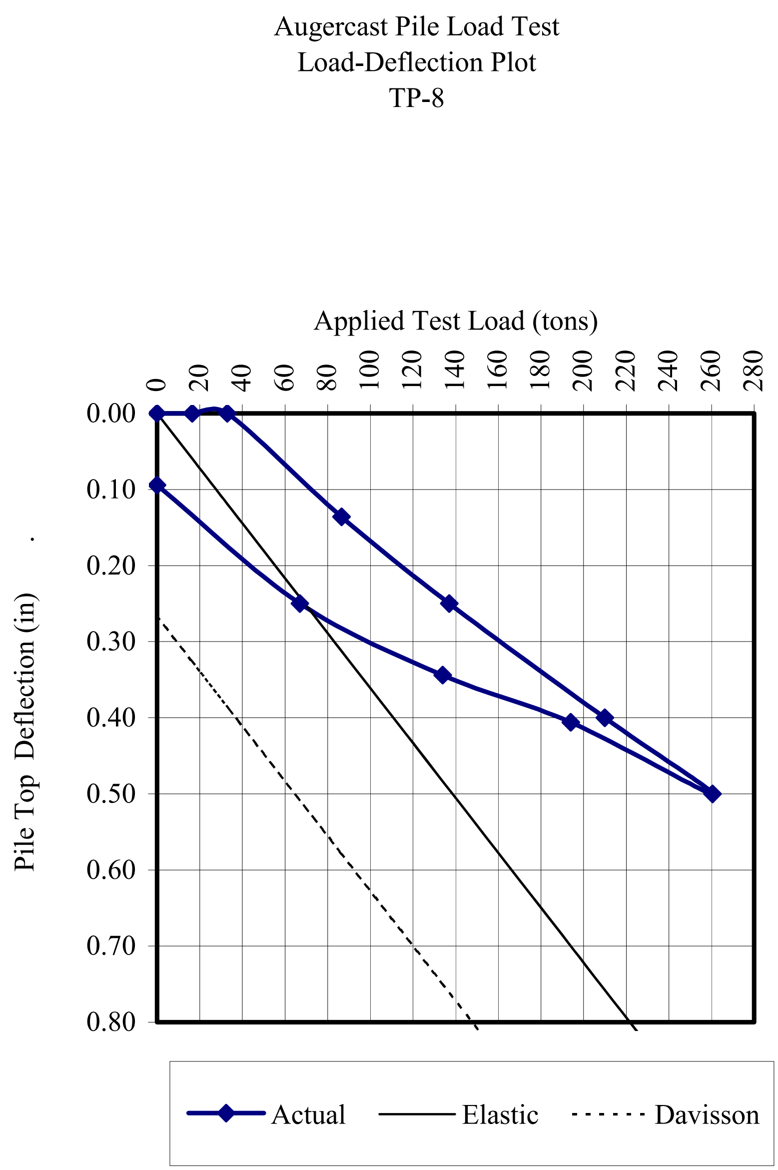


Figure 24: Davisson Offset Plot TP-8

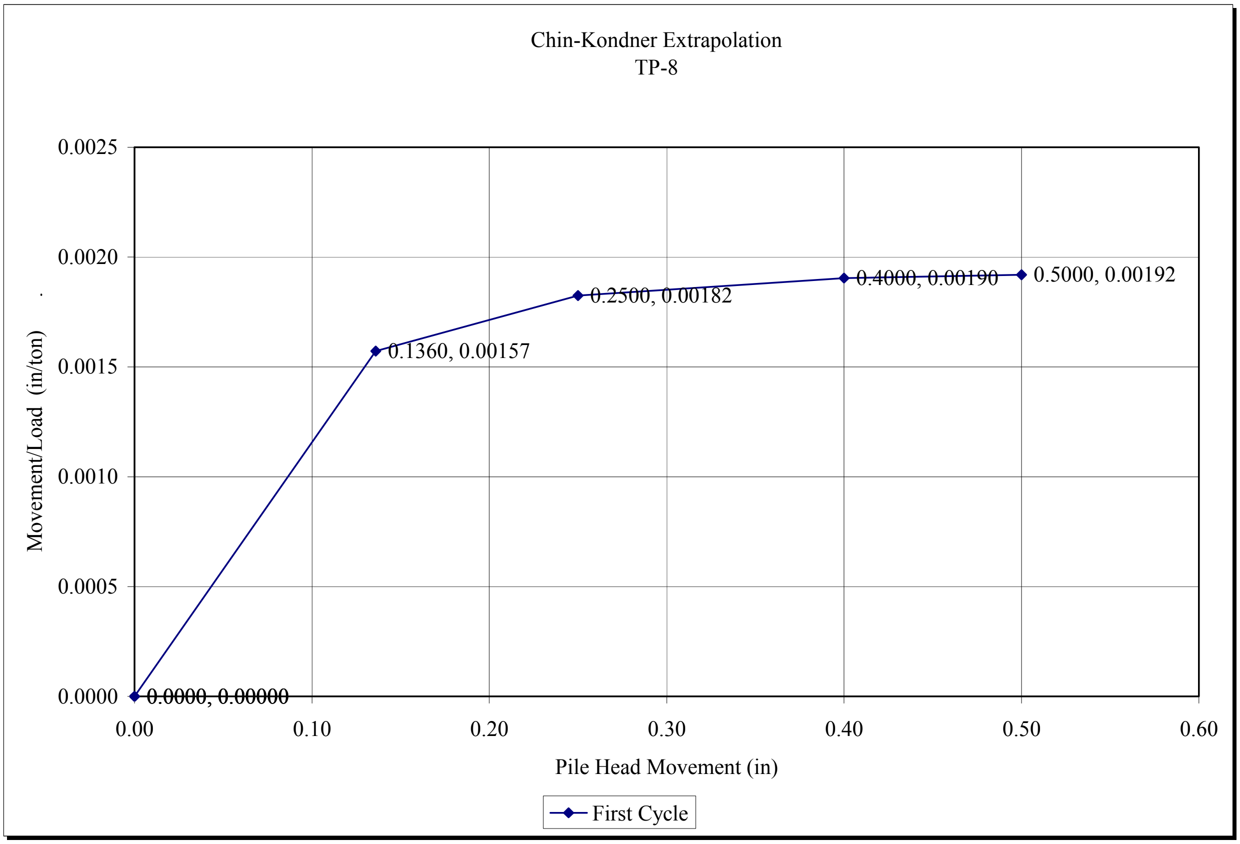


Figure 25: Chin-Kondner Extrapolation TP-8

Table 22: Pile Test Data TP-9

| Test No.: | TP-9 |
|---|---|
| Design Load (Tons): | 160 |
| Pile Diameter (inches): | 20 |
| Pile Area (sq. inches): | 314.16 |
| Pile Length (ft): | 85.0 |
| Pile Compress. Strength (psi): | 4500.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 3667011 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| 0 | 0.0000 | 0.0000 | 0.317 | -- |
| 20 | 0.0095 | 0.0354 | 0.352 | 0.00048 |
| 100 | 0.0750 | 0.1771 | 0.494 | 0.00075 |
| 120 | 0.0955 | 0.2125 | 0.529 | 0.00080 |
| 140 | 0.1180 | 0.2479 | 0.565 | 0.00084 |
| 160 | 0.1405 | 0.2833 | 0.600 | 0.00088 |
| 180 | 0.1650 | 0.3187 | 0.635 | 0.00092 |
| 200 | 0.2005 | 0.3542 | 0.671 | 0.00100 |
| 220 | 0.2345 | 0.3896 | 0.706 | 0.00107 |
| 240 | 0.2870 | 0.4250 | 0.742 | 0.00120 |
| 260 | 0.3220 | 0.4604 | 0.777 | 0.00124 |
| 280 | 0.3660 | 0.4958 | 0.812 | 0.00131 |
| 300 | 0.4435 | 0.5312 | 0.848 | 0.00148 |
| 320 | 0.5010 | 0.5667 | 0.883 | 0.00157 |
| 240 | 0.4980 | 0.4250 | 0.742 | 0.00208 |
| 160 | 0.4270 | 0.2833 | 0.600 | 0.00267 |
| 80 | 0.3420 | 0.1417 | 0.458 | 0.00428 |
| 0 | 0.1930 | 0.0000 | 0.317 | -- |

Table 23: Soil Profile Data TP-9

| **Layer No.** | **Depth feet** | **Layer depth** | **Water Table** | **Strata Classification** | **Blow Count N** | **Friction $\phi$** | **Cohesion c** |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>9 | 9 | 9 | SP | 5 | 27 | 0 |
| 2 | 9<br>16 | 7 | | SP | 9 | 32 | 0 |
| 3 | 16<br>31 | 15 | | CH | 4 | 0 | 840 |
| 4 | 31<br>56 | 25 | | SP | 40 | 36 | 0 |
| 5 | 56<br>85 | 29 | | CH | 8 | 0 | 1700 |

Augercast Pile Load Test
Load-Deflection Plot
TP-9

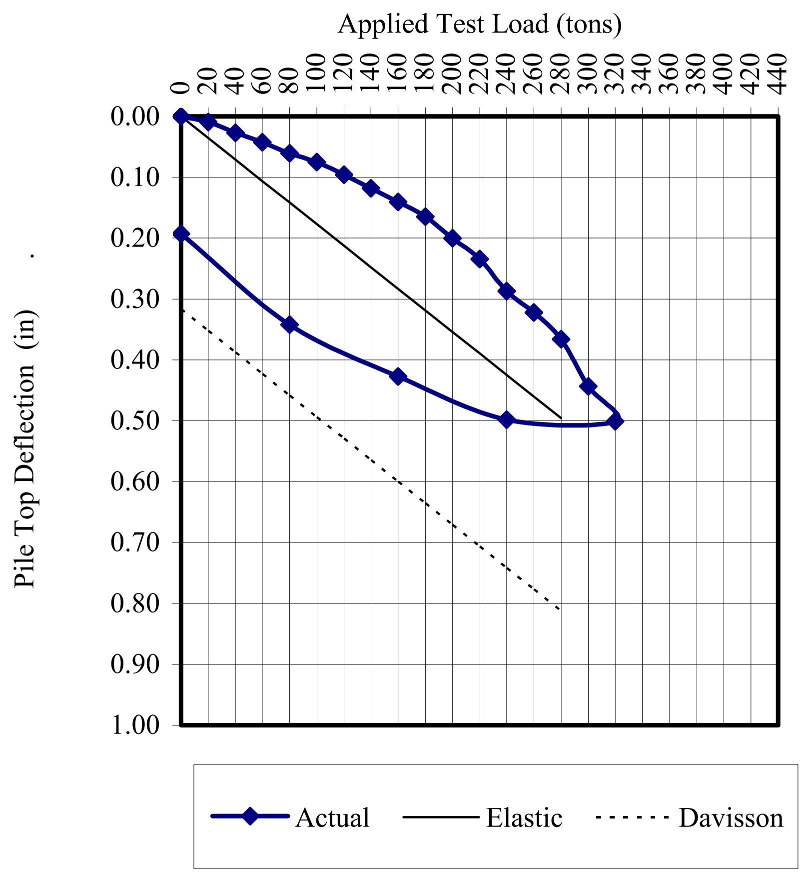


Figure 26: Davisson Offset Plot TP-9

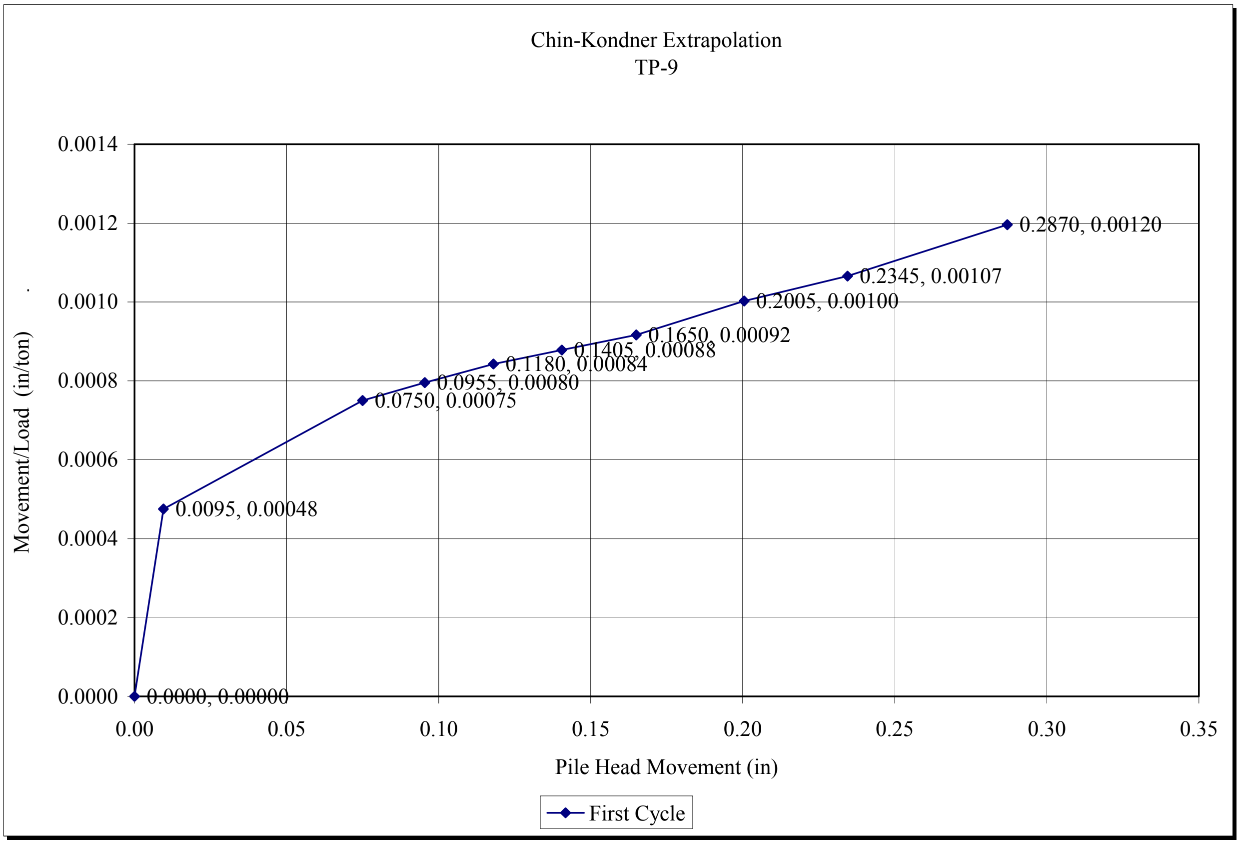


Figure 27: Chin-Kondner Extrapolation TP-9

Table 24: Pile Test Data TP-10

| **Test No.:** | **TP-10** |
|---|---|
| Design Load (Tons): | 50 |
| Pile Diameter (inches): | 16 |
| Pile Area (sq. inches): | 201.06 |
| Pile Length (ft): | 49.0 |
| Pile Compress. Strength (psi): | 6300.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 4338866 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| 0 | 0.0000 | 0.0000 | 0.283 | -- |
| 50 | 0.0630 | 0.0674 | 0.351 | 0.00126 |
| 100 | 0.4260 | 0.1348 | 0.418 | 0.00426 |
| 150 | 1.1150 | 0.2022 | 0.486 | 0.00743 |
| 200 | 1.9430 | 0.2696 | 0.553 | 0.00972 |
| 0 | 1.5930 | 0.0000 | 0.283 | -- |

Table 25: Soil Profile Data TP-10

| **Layer No.** | **Depth feet** | **Layer depth** | **Water Table** | **Strata Classification** | **Blow Count N** | **Friction $\phi$** | **Cohesion c** |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>3.5 | 3.5 | 3.5 | SP | 5 | 27 | 0 |
| 2 | 3.5<br>5 | 1.5 | | SP | 9 | 32 | 0 |
| 3 | 5<br>22 | 17 | | CH | 8 | 0 | 1700 |
| 4 | 22<br>31 | 9 | | SP | 45 | 45 | 0 |
| 5 | 31<br>49 | 18 | | SP-SM | 40 | 39 | 0 |

Augercast Pile Load Test
Load-Deflection Plot
TP-10

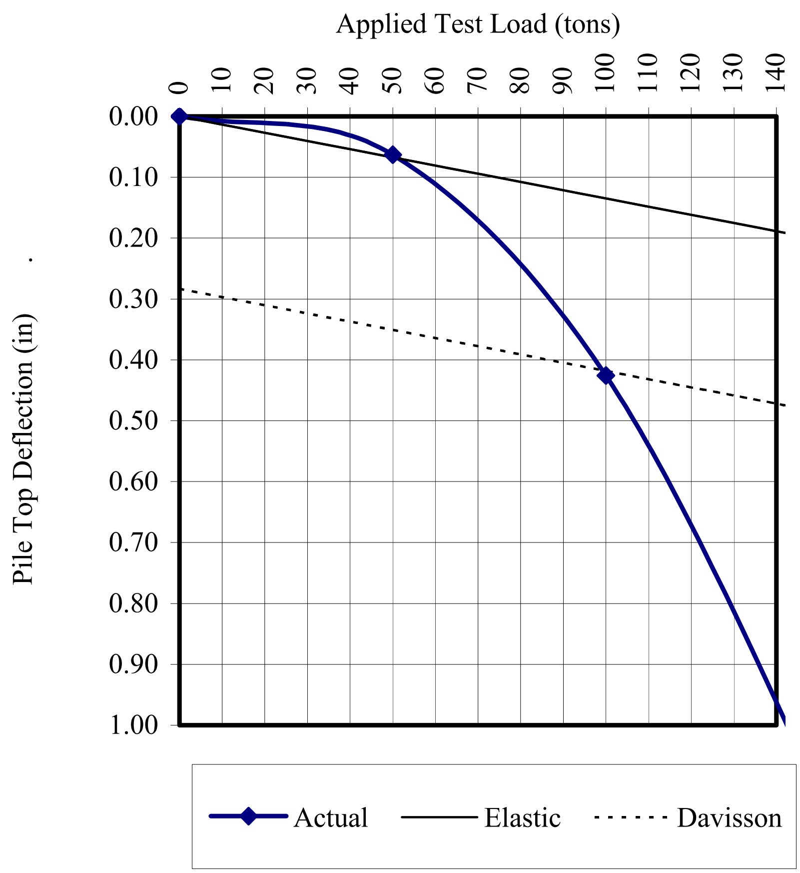


Figure 28: Davisson Offset Plot TP-10

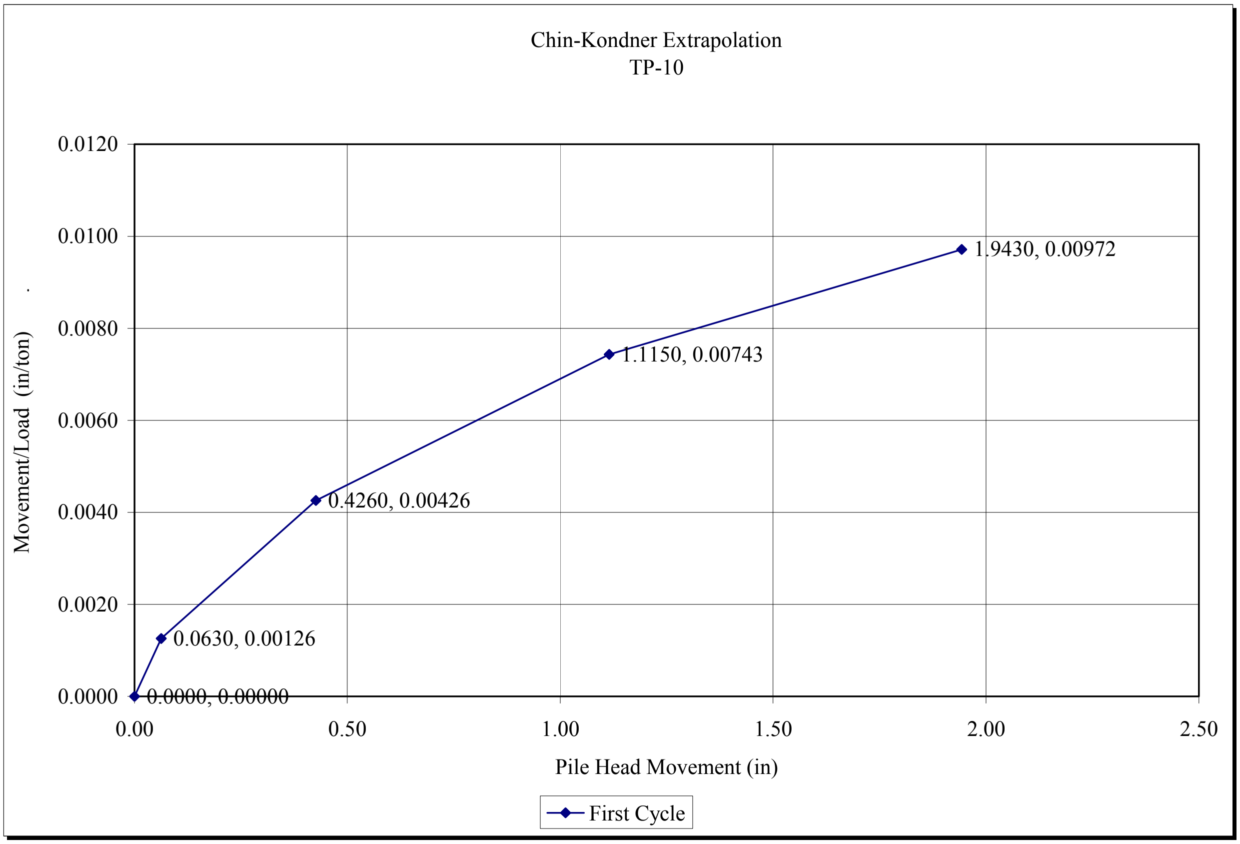


Figure 29: Chin-Kondner Extrapolation TP-10

Table 26: Pile Test Data TP-11

| Test No.: | | | | TP-11 |
|---|---|---|---|---|
| Design Load (Tons): | | | | 100 |
| Pile Diameter (inches): | | | | 16 |
| Pile Area (sq. inches): | | | | 201.06 |
| Pile Length (ft): | | | | 49.0 |
| Pile Compress. Strength (psi): | | | | 4500.00 |
| Pile Unit Weight (pcf): | | | | 140 |
| Pile Modulus of Elasticity (psi): | | | | 3667011 |
| | | | | |
| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
| 0 | 0.0000 | 0.0000 | 0.283 | -- |
| 50 | 0.0974 | 0.0798 | 0.363 | 0.00195 |
| 100 | 0.2760 | 0.1595 | 0.443 | 0.00276 |
| 150 | 0.5290 | 0.2393 | 0.523 | 0.00353 |
| 200 | 0.9410 | 0.3190 | 0.602 | 0.00471 |
| 0 | 0.6710 | 0.0000 | 0.283 | -- |

Table 27: Soil Profile Data TP-11

| **Layer No.** | **Depth feet** | **Layer depth** | **Water Table** | **Strata Classification** | **Blow Count N** | **Friction $\phi$** | **Cohesion c** |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>3 | 3 | 3 | SP | 5 | 27 | 0 |
| 2 | 3<br>5 | 2 | | SP | 9 | 32 | 0 |
| 3 | 5<br>24 | 19 | | CH | 10 | 0 | 2100 |
| 4 | 24<br>31 | 7 | | SP | 45 | 45 | 0 |
| 5 | 31<br>49 | 18 | | SP-SM | 40 | 39 | 0 |

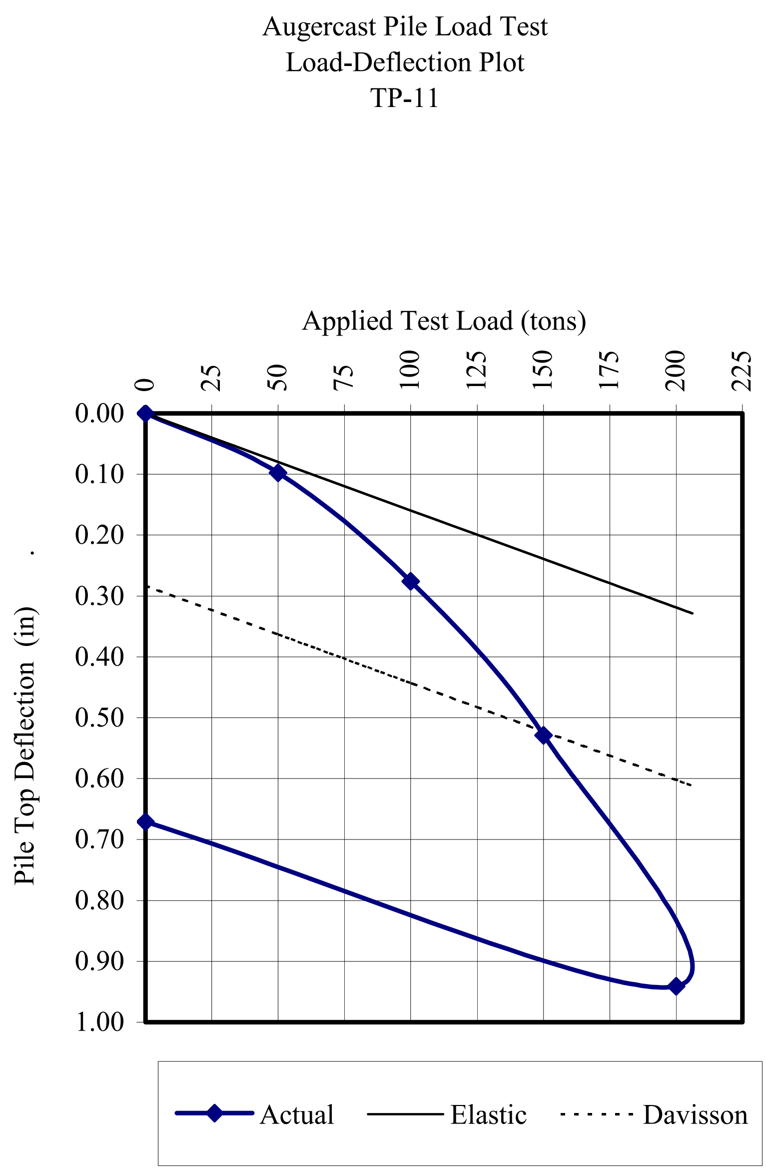


Figure 30: Davisson Offset Plot TP-11

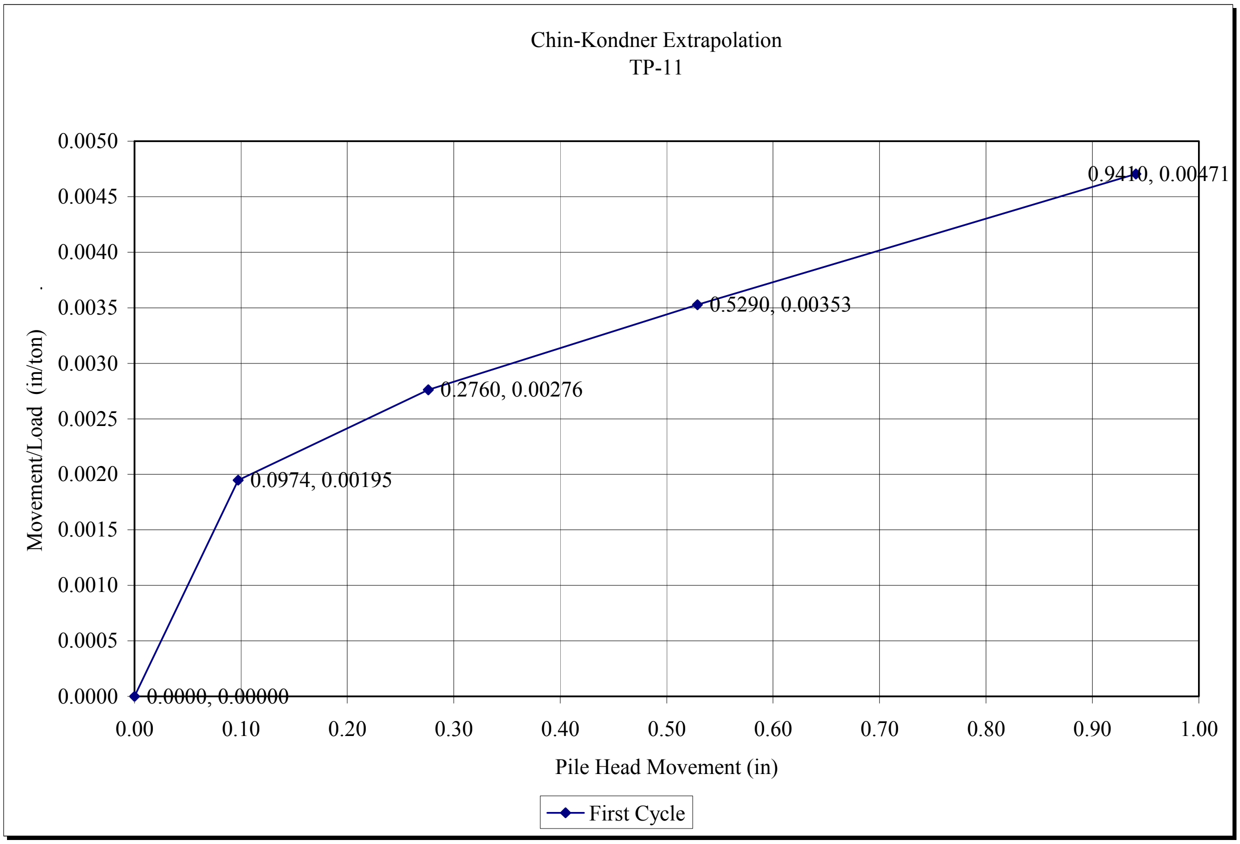


Figure 31: Chin-Kondner Extrapolation TP-11

Table 28: Pile Test Data TP-12

| **Test No.:** | **TP-12** |
|---|---|
| Design Load (Tons): | 50 |
| Pile Diameter (inches): | 16 |
| Pile Area (sq. inches): | 201.06 |
| Pile Length (ft): | 59.0 |
| Pile Compress. Strength (psi): | 4500.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 3667011 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| 0 | 0.0000 | 0.0000 | 0.283 | -- |
| 50 | 0.0740 | 0.0960 | 0.379 | 0.00148 |
| 100 | 0.1880 | 0.1921 | 0.475 | 0.00188 |
| 125 | 0.2680 | 0.2401 | 0.523 | 0.00214 |
| 150 | 0.3420 | 0.2881 | 0.571 | 0.00228 |
| 0 | 0.0940 | 0.0000 | 0.283 | -- |

Table 29: Soil Profile Data TP-12

| **Layer No.** | **Depth feet** | **Layer depth** | **Water Table** | **Strata Classification** | **Blow Count N** | **Friction $\phi$** | **Cohesion c** |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>2.5 | 2.5 | 2.5 | SP | 5 | 27 | 0 |
| 2 | 2.5<br>4 | 1.5 | | SP | 9 | 32 | 0 |
| 3 | 4<br>20 | 16 | | CH | 10 | 0 | 2100 |
| 4 | 20<br>33 | 13 | | SP | 45 | 45 | 0 |
| 5 | 33<br>59 | 26 | | SP-SM | 40 | 39 | 0 |

Augercast Pile Load Test
Load-Deflection Plot
TP-12

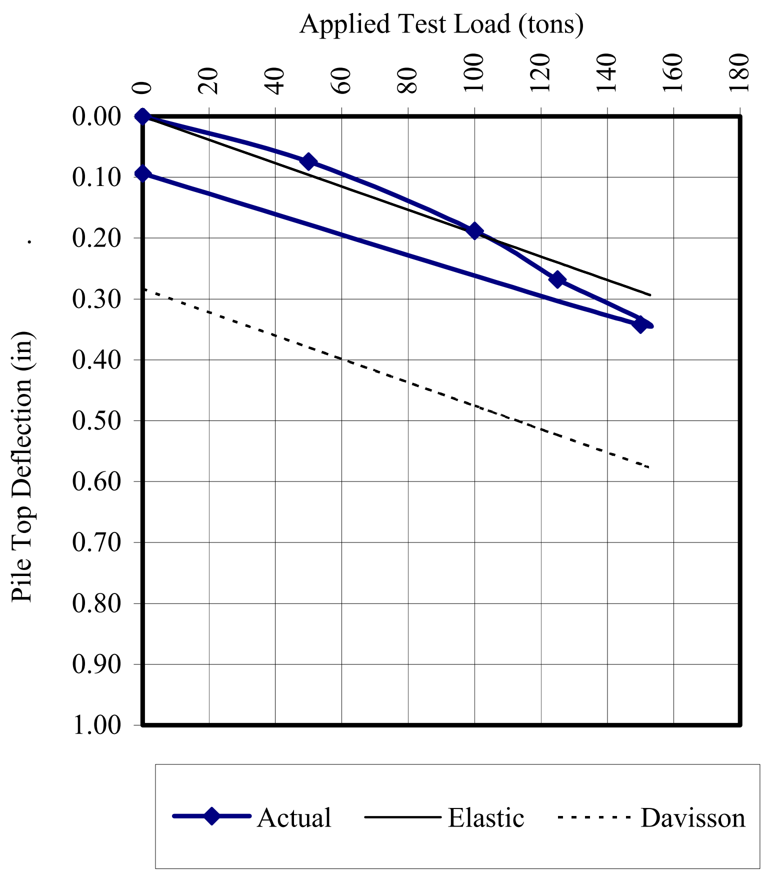


Figure 32: Davisson Offset TP-12

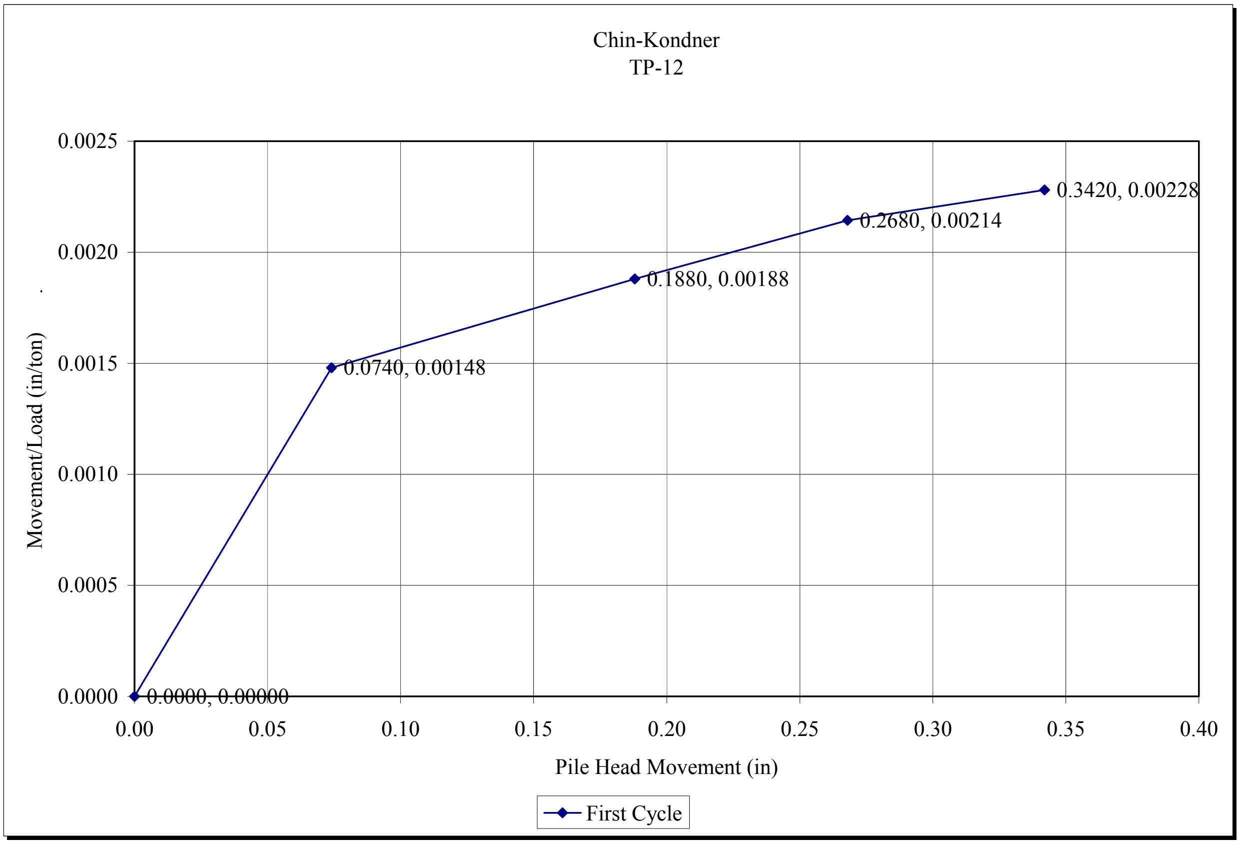


Figure 33: Chin-Kondner Extrapolation TP-12

Table 30: Pile Test Data TP-13

| **Test No.:** | **TP-13** |
|---|---|
| Design Load (Tons): | 100 |
| Pile Diameter (inches): | 16 |
| Pile Area (sq. inches): | 201.06 |
| Pile Length (ft): | 59.0 |
| Pile Compress. Strength (psi): | 8165.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 4939512 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| 0 | 0.0000 | 0.0000 | 0.283 | -- |
| 50 | 0.0500 | 0.0713 | 0.355 | 0.00100 |
| 100 | 0.4000 | 0.1426 | 0.426 | 0.00400 |
| 150 | 1.0000 | 0.2139 | 0.497 | 0.00667 |
| 190 | 1.6800 | 0.2709 | 0.554 | 0.00884 |
| 200 | 1.9000 | 0.2852 | 0.568 | 0.00950 |

Table 31: Soil Profile Data TP-13

| **Layer No.** | **Depth feet** | **Layer depth** | **Water Table** | **Strata Classification** | **Blow Count N** | **Friction $\phi$** | **Cohesion c** |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>3 | 3 | 3 | SP | 5 | 27 | 0 |
| 2 | 3<br>6 | 3 | | SP | 9 | 32 | 0 |
| 3 | 6<br>23 | 17 | | CH | 10 | 0 | 2100 |
| 4 | 23<br>35 | 12 | | SP | 45 | 45 | 0 |
| 5 | 35<br>59 | 24 | | SP-SM | 40 | 39 | 0 |

Augercast Pile Load Test
Load-Deflection Plot
TP-13

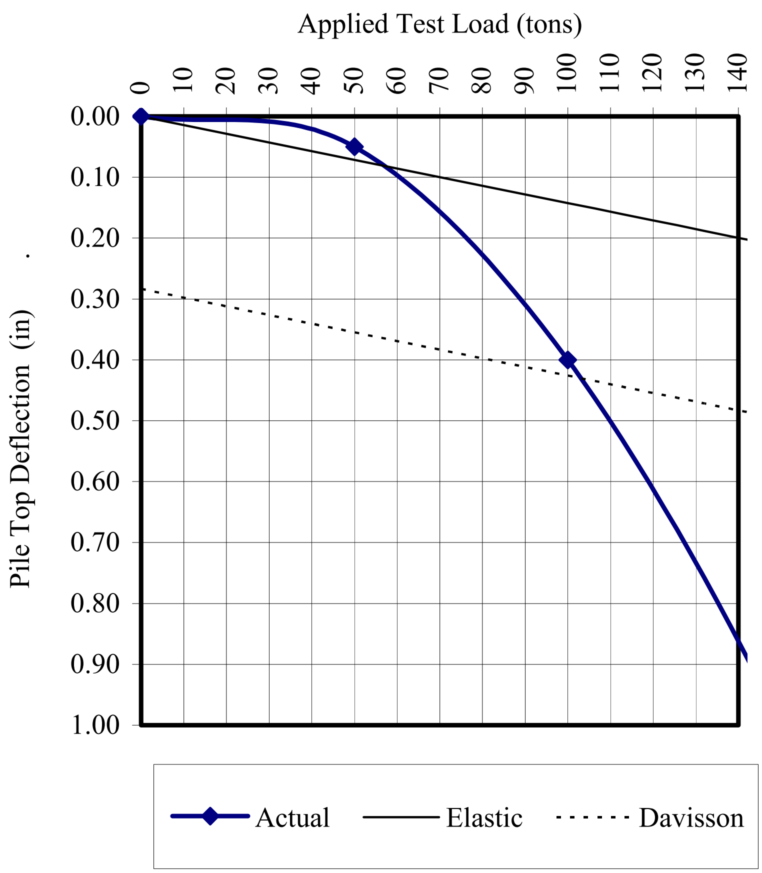


Figure 34: Davisson Offset Plot TP-13

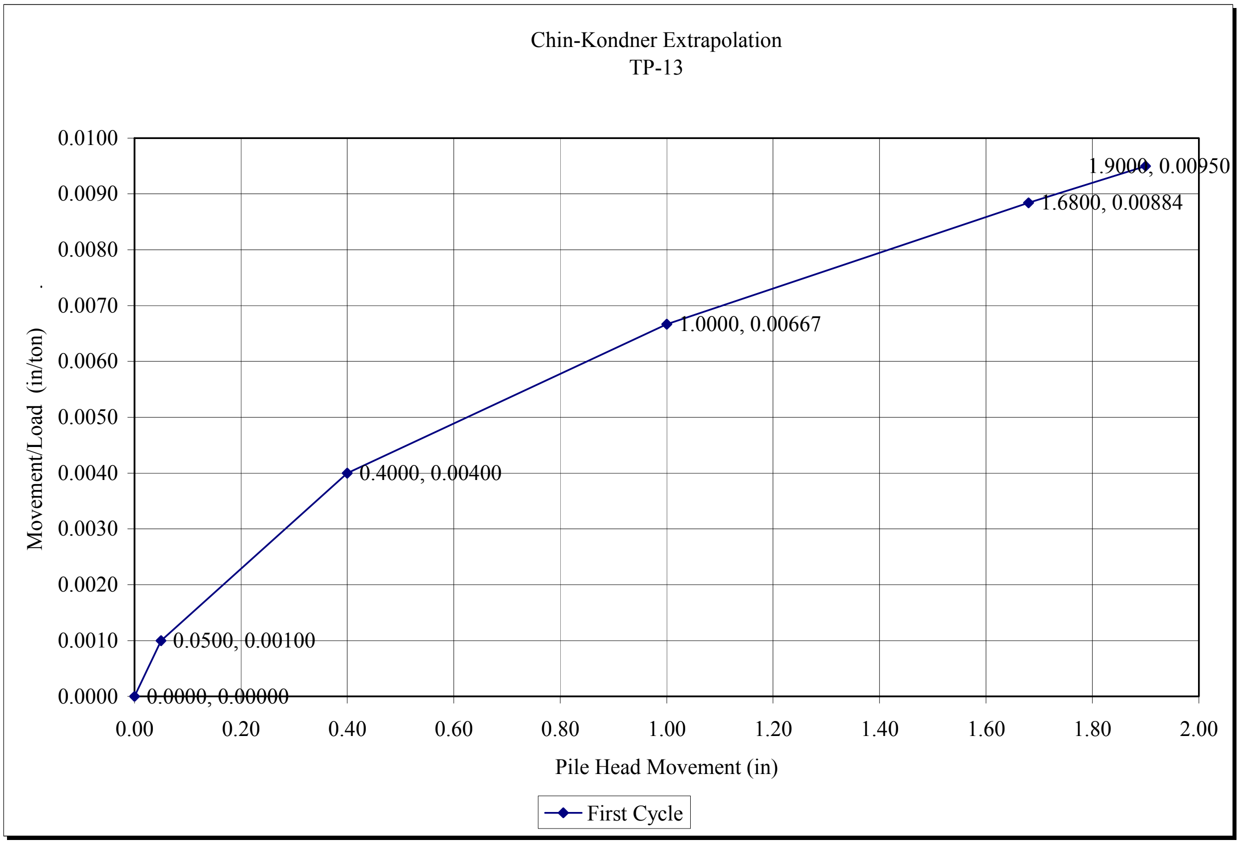


Figure 35: Chin-Kondner Extrapolation TP-13

Table 32: Pile Test Data TP-14

| **Test No.:** | **TP-14** |
|---|---|
| Design Load (Tons): | 35 |
| Pile Diameter (inches): | 14 |
| Pile Area (sq. inches): | 153.94 |
| Pile Length (ft): | 30.0 |
| Pile Compress. Strength (psi): | 4250.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 3563694 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| **0** | **0** | 0.0000 | 0.267 | -- |
| 5 | 0.0100 | 0.0066 | 0.273 | 0.00200 |
| 10 | 0.0250 | 0.0131 | 0.280 | 0.00250 |
| 20 | 0.0550 | 0.0262 | 0.293 | 0.00275 |
| 30 | 0.0920 | 0.0394 | 0.306 | 0.00307 |
| 35 | 0.1150 | 0.0459 | 0.313 | 0.00329 |
| 50 | 0.2260 | 0.0656 | 0.332 | 0.00452 |
| 70 | 0.3440 | 0.0919 | 0.359 | 0.00491 |
| 35 | 0.3730 | 0.0459 | 0.313 | 0.01066 |
| 17.5 | 0.3520 | 0.0230 | 0.290 | 0.02011 |
| 0 | 0.2940 | 0.0000 | 0.267 | -- |

Table 33: Soil Profile Data TP-14

| **Layer No.** | **Depth feet** | **Layer depth** | **Water Table** | **Strata Classification** | **Blow Count N** | **Friction $\phi$** | **Cohesion c** |
|---|---|---|---|---|---|---|---|
| 1 | 0 | | | | | | |
| | 2 | 2 | 2 | SP | 10 | 32 | 0 |
| 2 | 2 | | | | | | |
| | 8 | 6 | | SP | 8 | 30 | 0 |
| 3 | 8 | | | | | | |
| | 20 | 12 | | CH | 5 | 0 | 1050 |
| 4 | 20 | | | | | | |
| | 30 | 10 | | SP | 40 | 38 | 0 |

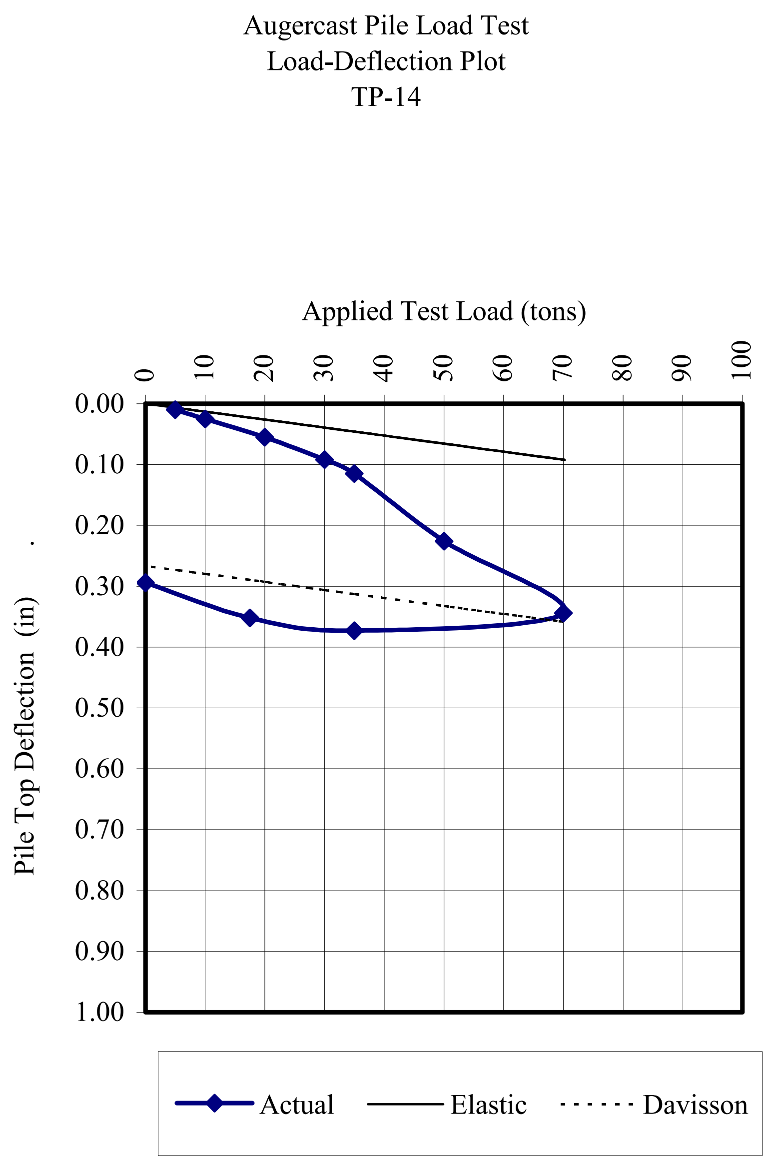


Figure 36: Davisson Offset Plot TP-14

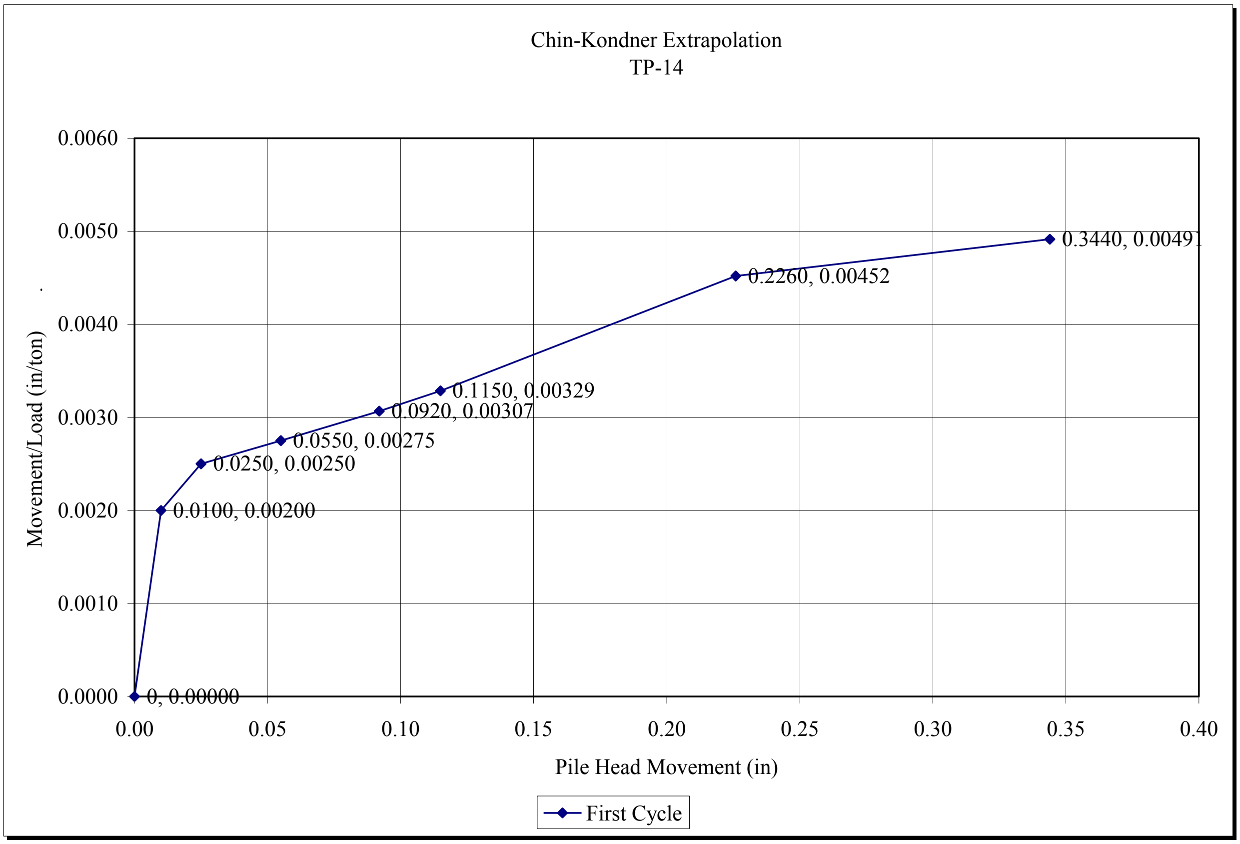


Figure 37: Chin-Kondner Extrapolation TP-14

Table 34: Pile Test Data TP-15

| **Test No.:** | **TP-15** |
|---|---|
| Design Load (Tons): | 75 |
| Pile Diameter (inches): | 18 |
| Pile Area (sq. inches): | 254.47 |
| Pile Length (ft): | 75.0 |
| Pile Compress. Strength (psi): | 4250.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 3563694 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| **0** | **0** | 0.0000 | 0.300 | -- |
| 15 | 0.0050 | 0.0298 | 0.330 | 0.00033 |
| 30 | 0.0150 | 0.0595 | 0.360 | 0.00050 |
| 60 | 0.0560 | 0.1191 | 0.419 | 0.00093 |
| 75 | 0.0990 | 0.1489 | 0.449 | 0.00132 |
| 120 | 0.2210 | 0.2382 | 0.538 | 0.00184 |
| 150 | 0.3210 | 0.2977 | 0.598 | 0.00214 |
| 70 | 0.2840 | 0.1389 | 0.439 | 0.00406 |
| 35 | 0.1970 | 0.0695 | 0.369 | 0.00563 |
| 0 | 0.1030 | 0.0000 | 0.300 | -- |

Table 35: Soil Profile Data TP-15

| **Layer No.** | **Depth feet** | **Layer depth** | **Water Table** | **Strata Classification** | **Blow Count N** | **Friction $\phi$** | **Cohesion c** |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>7 | 7 | 7 | SP | 5 | 27 | 0 |
| 2 | 7<br>28 | 21 | | SP | 9 | 30 | 0 |
| 3 | 28<br>31 | 3 | | SM | 10 | 32 | 0 |
| 4 | 31<br>53 | 22 | | CH | 5 | 0 | 1050 |
| 5 | 53<br>60 | 7 | | SP-SM | 20 | 32 | 0 |
| 6 | 53<br>75 | 22 | | SM | 30 | 36 | 0 |

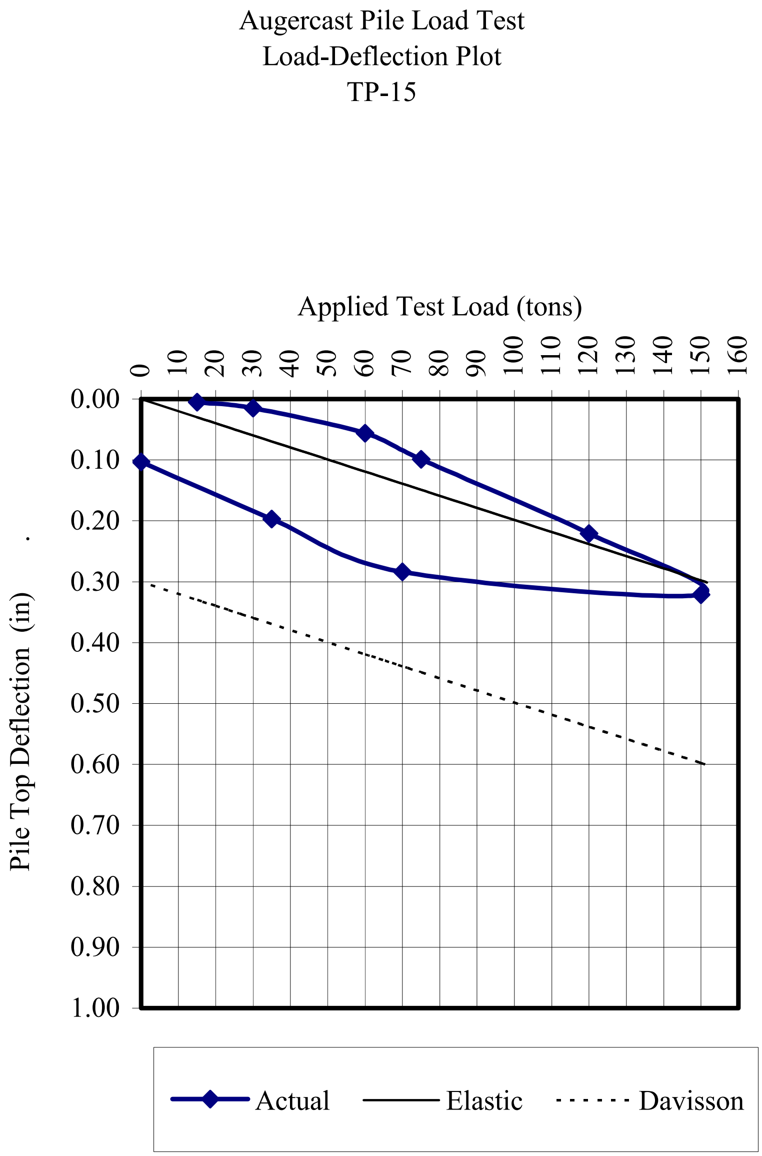


Figure 38: Davisson Offset Plot TP-15

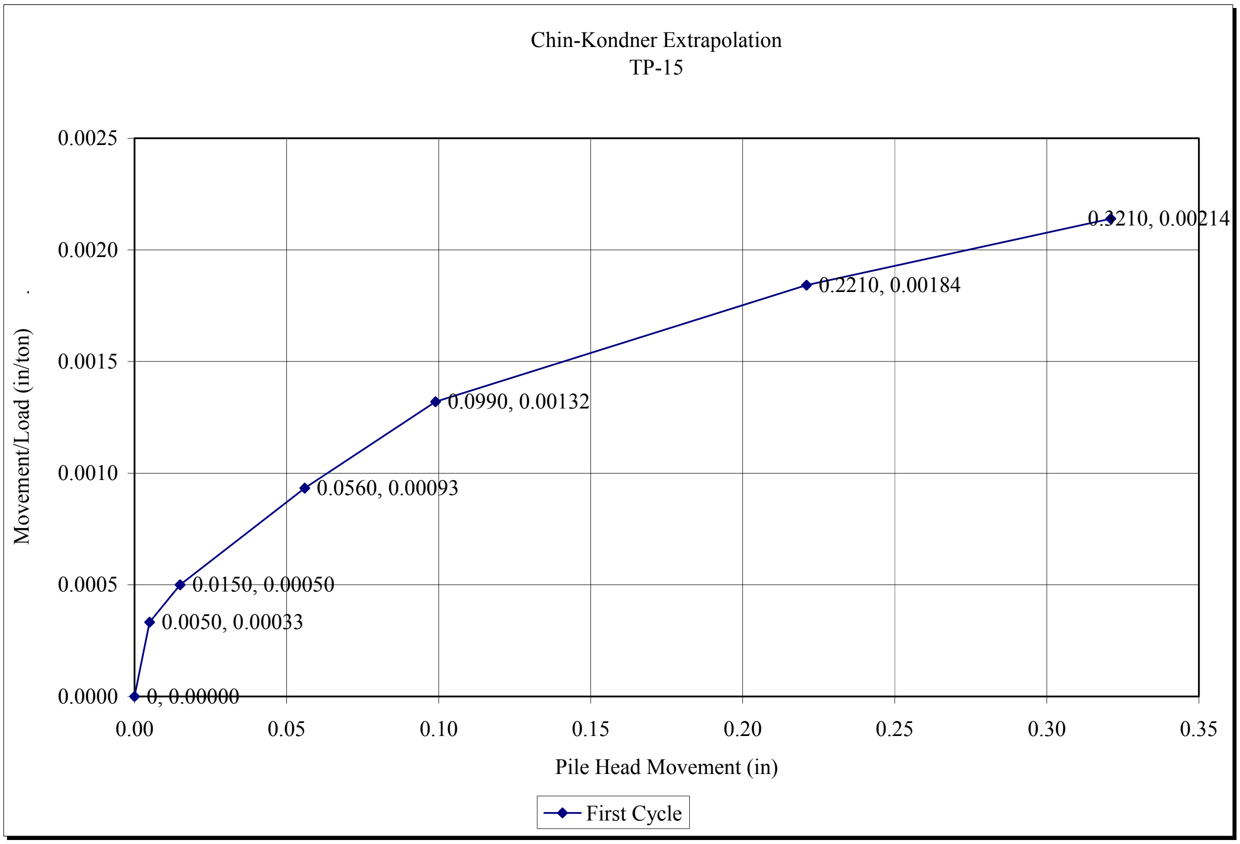


Figure 39: Chin-Kondner Extrapolation TP-15

Table 36: Pile Test Data TP-16

| Test No.: | TP-16 |
|---|---|
| Design Load (Tons): | 150 |
| Pile Diameter (inches): | 18 |
| Pile Area (sq. inches): | 254.47 |
| Pile Length (ft): | 75.0 |
| Pile Compress. Strength (psi): | 4250.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 3563694 |

| Applied Test Load in Q (tons) | Average Measured Pile Top Deflection, D (inches) | Theoretical Pile Elastic Deflection in (inches) | Theoretical Davisson Anticipated Deflection in (inches) | Chin-Kondner Anticipated Deflection D/Q (in/ton) |
|---|---|---|---|---|
| **0** | **0** | 0.0000 | 0.300 | -- |
| 30 | 0.0840 | 0.0595 | 0.360 | 0.00280 |
| 60 | 0.2210 | 0.1191 | 0.419 | 0.00368 |
| 90 | 0.3670 | 0.1786 | 0.479 | 0.00408 |
| 120 | 0.4970 | 0.2382 | 0.538 | 0.00414 |
| 180 | 0.7210 | 0.3573 | 0.657 | 0.00401 |
| 210 | 0.8520 | 0.4168 | 0.717 | 0.00406 |
| 255 | 0.9400 | 0.5061 | 0.806 | 0.00369 |
| 300 | 1.2500 | 0.5955 | 0.895 | 0.00417 |
| 0 | 0.4360 | 0.0000 | 0.300 | -- |

Table 37: Soil Profile Data TP-16

| Layer No. | Depth feet | Layer depth | Water Table | Strata Classification | Blow Count N | Friction $\phi$ | Cohesion c |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>13 | 13 | 13 | SP | 5 | 27 | 0 |
| 2 | 13<br>26 | 13 | | SP | 9 | 30 | 0 |
| 3 | 26<br>30 | 4 | | SM | 13 | 33 | 0 |
| 4 | 30<br>54 | 24 | | CH | 12 | 0 | 2505 |
| 5 | 54<br>60 | 6 | | SP-SM | 24 | 36 | 0 |
| 6 | 60<br>75 | 15 | | SM | 38 | 40 | 0 |

Augercast Pile Load Test
Load-Deflection Plot
TP-16

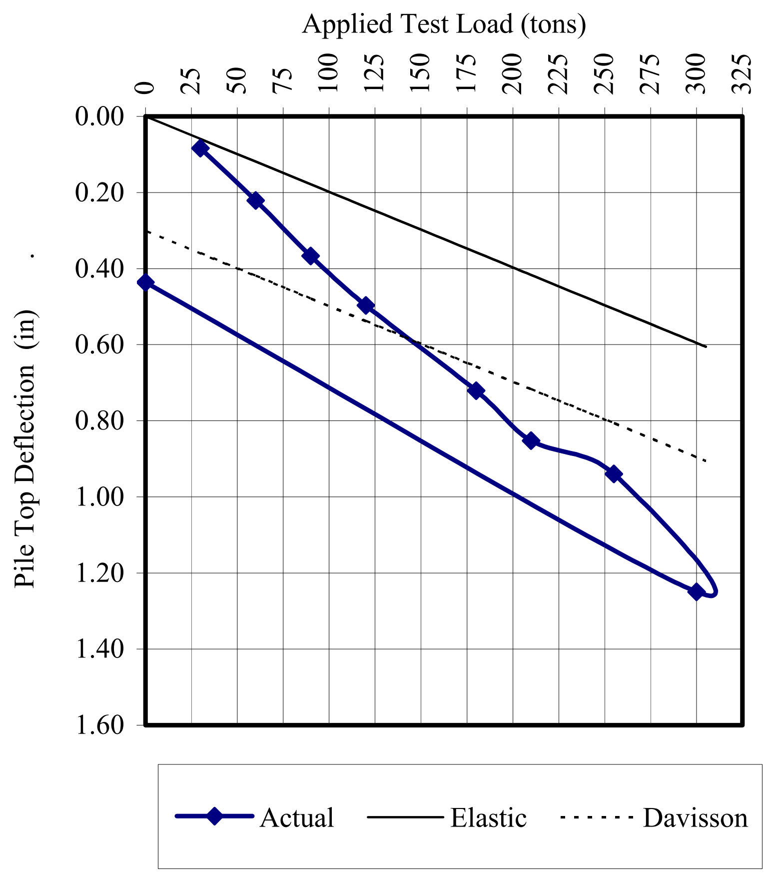


Figure 40: Davisson Offset Plot TP-16

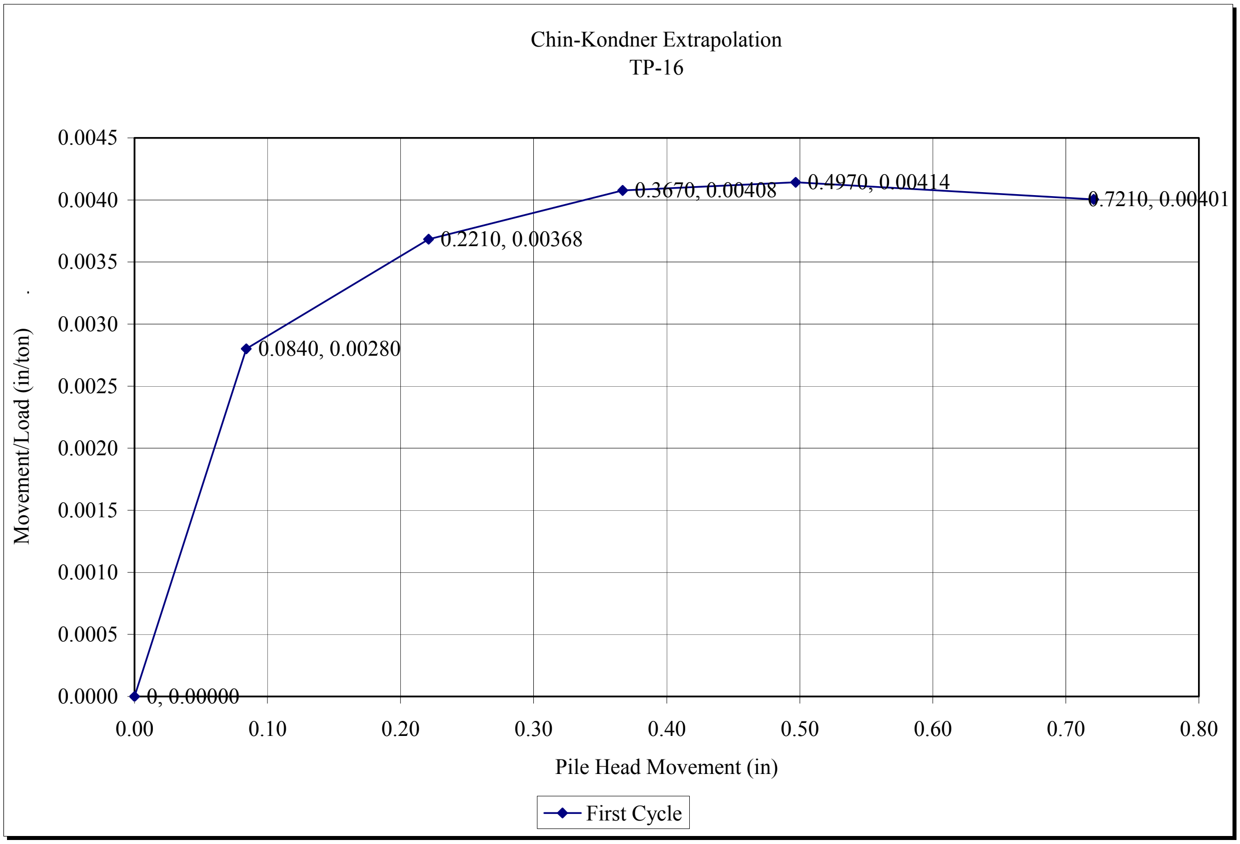


Figure 41: Chin-Kondner Extrapolation TP-16

Table 38: Pile Test Data TP-17

| Test No.: | TP-17 |
|---|---|
| Design Load (Tons): | 80 |
| Pile Diameter (inches): | 16 |
| Pile Area (sq. inches): | 201.06 |
| Pile Length (ft): | 78.0 |
| Pile Compress. Strength (psi): | 4000.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 3457291 |

| Applied Test Load in Q (tons) | Average Measured Pile Top Deflection, D (inches) | Theoretical Pile Elastic Deflection in (inches) | Theoretical Davisson Anticipated Deflection in (inches) | Chin-Kondner Anticipated Deflection D/Q (in/ton) |
|---|---|---|---|---|
| **0** | **0** | 0.0000 | 0.283 | -- |
| 10 | 0.005 | 0.0269 | 0.310 | 0.00050 |
| 20 | 0.0180 | 0.0539 | 0.337 | 0.00090 |
| 40 | 0.0430 | 0.1077 | 0.391 | 0.00108 |
| 60 | 0.0800 | 0.1616 | 0.445 | 0.00133 |
| 80 | 0.1200 | 0.2154 | 0.499 | 0.00150 |
| 100 | 0.1500 | 0.2693 | 0.553 | 0.00150 |
| 120 | 0.2000 | 0.3232 | 0.606 | 0.00167 |
| 140 | 0.2750 | 0.3770 | 0.660 | 0.00196 |
| 160 | 0.9200 | 0.4309 | 0.714 | 0.00575 |
| 0 | 0.8700 | 0.0000 | 0.283 | -- |

Table 39: Soil Profile Data TP-17

| Layer No. | Depth feet | Layer depth | Water Table | Strata Classification | Blow Count N | Friction $\phi$ | Cohesion c |
|---|---|---|---|---|---|---|---|
| 1 | 0 | | | | | | |
| | 3 | 3 | 3 | SP-SM | 4 | 26 | 0 |
| 2 | 3 | | | | | | |
| | 7.5 | 4.5 | | SM | 9 | 30 | 0 |
| 3 | 7.5 | | | | | | |
| | 23 | 15.5 | | SM | 13 | 33 | 0 |
| 4 | 23 | | | | | | |
| | 47 | 24 | | CH | 12 | 0 | 2505 |
| 5 | 47 | | | | | | |
| | 78 | 31 | | SP-SM | 24 | 36 | 0 |

Augercast Pile Load Test
Load-Deflection Plot
TP-17

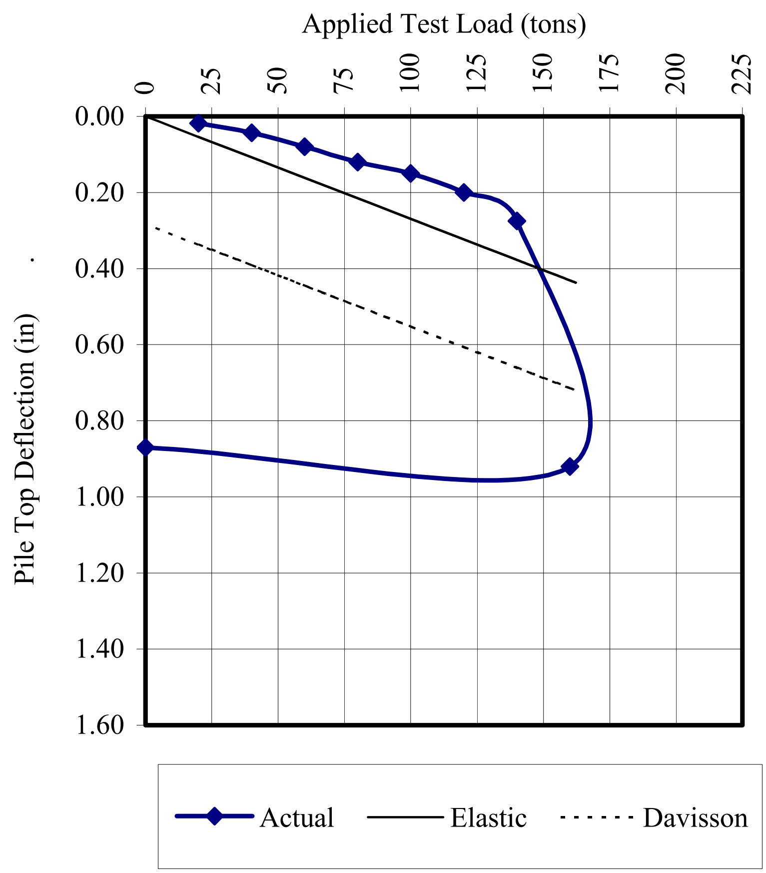


Figure 42: Davisson Offset Plot TP-17

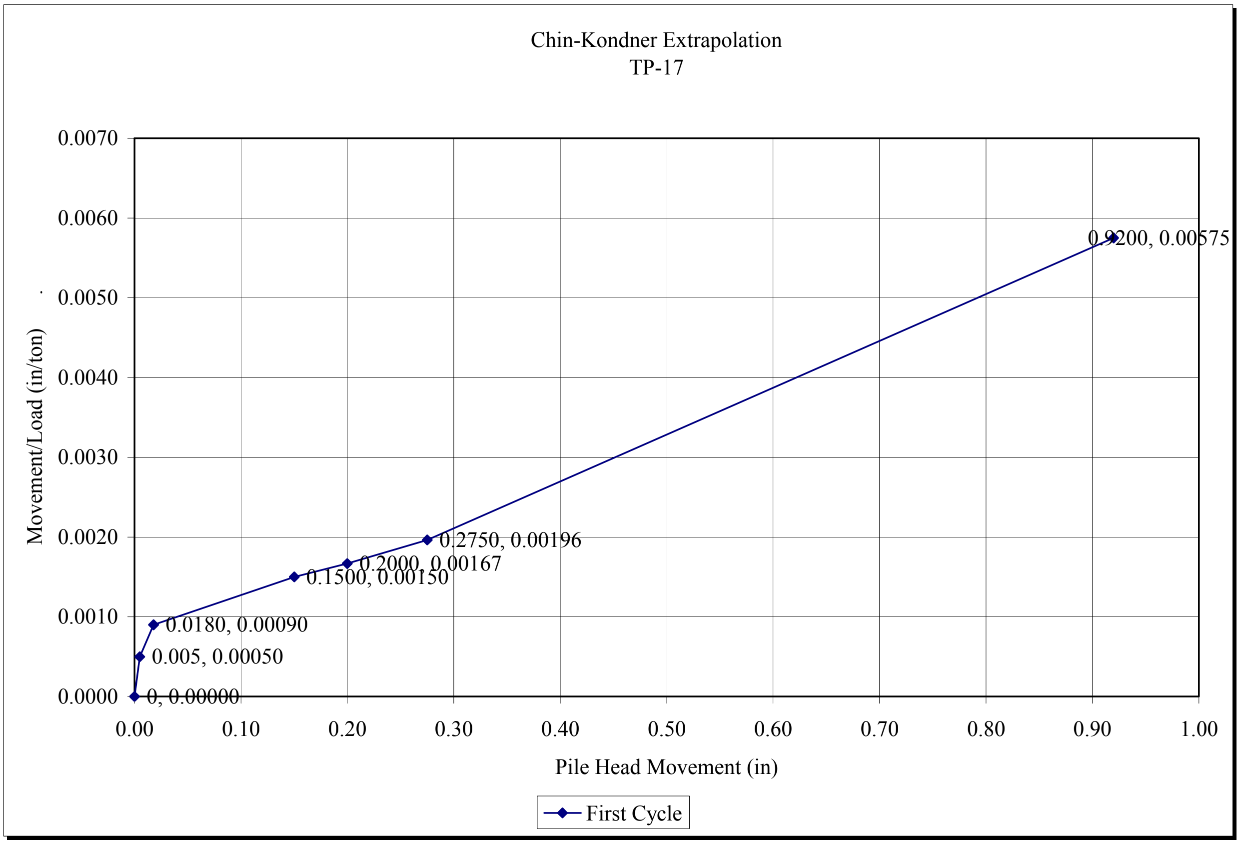


Figure 43: Chin-Kondner Extrapolation TP-17

Table 40: Pile Test Data TP-18

| **Test No.:** | **TP-18** |
|---|---|
| Design Load (Tons): | 100 |
| Pile Diameter (inches): | 16 |
| Pile Area (sq. inches): | 201.06 |
| Pile Length (ft): | 79.5 |
| Pile Compress. Strength (psi): | 4000.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 3457291 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| **0** | **0** | 0.0000 | 0.283 | -- |
| 10 | 0.002 | 0.0274 | 0.311 | 0.00020 |
| 20 | 0.0100 | 0.0549 | 0.338 | 0.00050 |
| 40 | 0.0250 | 0.1098 | 0.393 | 0.00063 |
| 60 | 0.0500 | 0.1647 | 0.448 | 0.00083 |
| 80 | 0.0750 | 0.2196 | 0.503 | 0.00094 |
| 100 | 0.0980 | 0.2745 | 0.558 | 0.00098 |
| 140 | 0.1480 | 0.3843 | 0.668 | 0.00106 |
| 180 | 0.2200 | 0.4941 | 0.777 | 0.00122 |
| 200 | 0.2750 | 0.5490 | 0.832 | 0.00138 |
| 150 | 0.2650 | 0.4117 | 0.695 | 0.00177 |
| 100 | 0.2470 | 0.2745 | 0.558 | 0.00247 |
| 50 | 0.2300 | 0.1372 | 0.421 | 0.00460 |
| 0 | 0.1800 | 0.0000 | 0.283 | -- |

Table 41: Soil Profile Data TP-18

| Layer No. | Depth feet | Layer depth | Water Table | Strata Classification | Blow Count N | Friction $\phi$ | Cohesion c |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>4 | 4 | | SP-SM | 4 | 26 | 0 |
| 2 | 4<br>6 | 2 | 6 | SC | 9 | 30 | 0 |
| 3 | 6<br>21 | 15 | | SC | 13 | 33 | 0 |
| 4 | 21<br>42 | 21 | | CH | 12 | 0 | 2505 |
| 5 | 42<br>79.5 | 37.5 | | SP-SM | 30 | 39 | 0 |

Augercast Pile Load Test
Load-Deflection Plot
TP-18

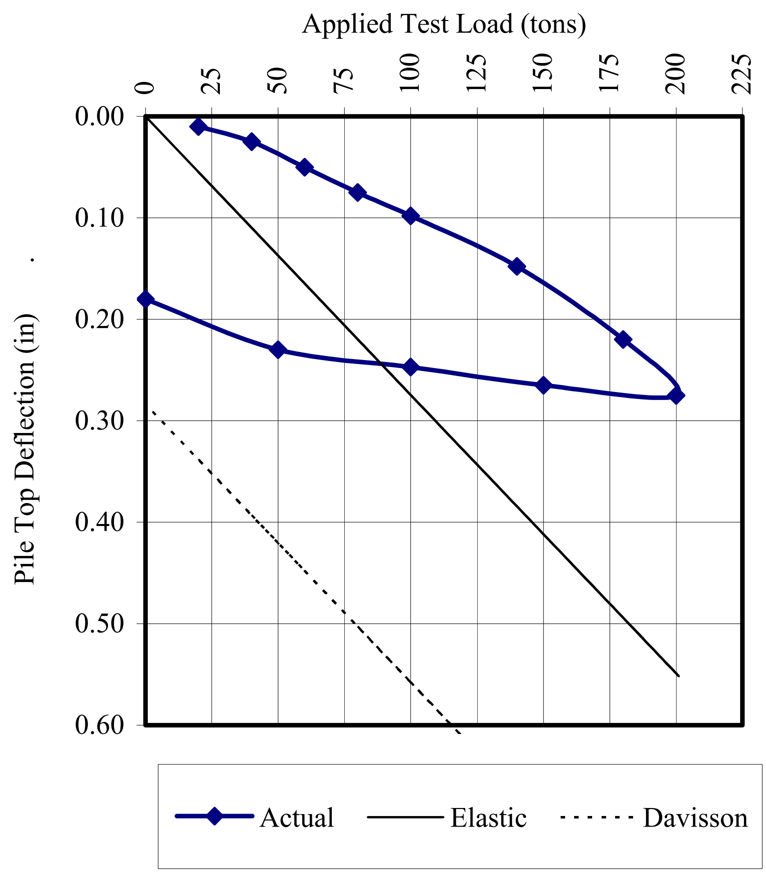


Figure 44: Davisson Offset Plot TP-18

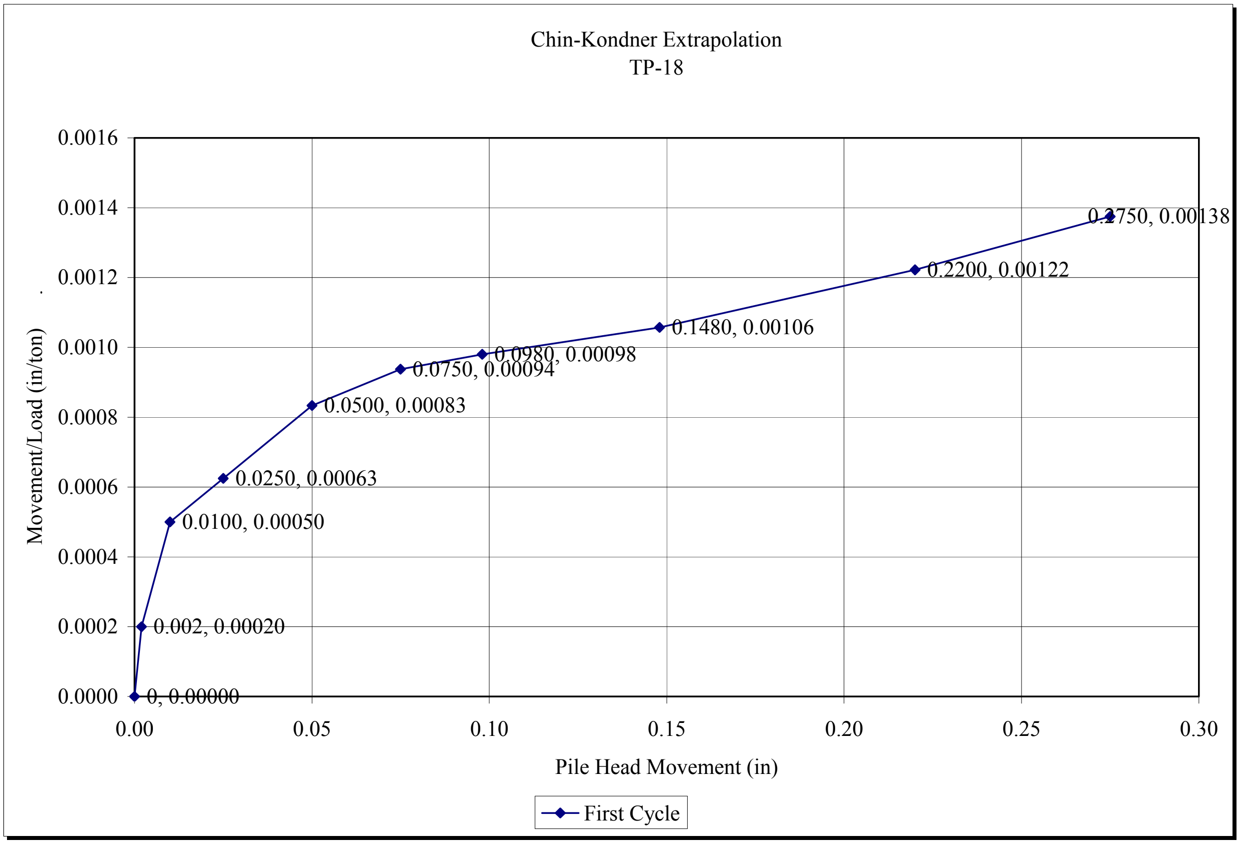


Figure 45: Chin-Kondner Extrapolation TP-18

Table 42: Pile Test Data TP-19

| **Test No.:** | **TP-19** |
|---|---|
| Design Load (Tons): | 100 |
| Pile Diameter (inches): | 16 |
| Pile Area (sq. inches): | 201.06 |
| Pile Length (ft): | 78.5 |
| Pile Compress. Strength (psi): | 4000.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 3457291 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| **0** | **0** | 0.0000 | 0.283 | -- |
| 20 | 0.0110 | 0.0542 | 0.338 | 0.00055 |
| 40 | 0.0350 | 0.1084 | 0.392 | 0.00088 |
| 60 | 0.0620 | 0.1626 | 0.446 | 0.00103 |
| 80 | 0.1010 | 0.2168 | 0.500 | 0.00126 |
| 100 | 0.1400 | 0.2710 | 0.554 | 0.00140 |
| 140 | 0.2410 | 0.3794 | 0.663 | 0.00172 |
| 180 | 0.3450 | 0.4879 | 0.771 | 0.00192 |
| 200 | 0.5200 | 0.5421 | 0.825 | 0.00260 |
| 150 | 0.4850 | 0.4065 | 0.690 | 0.00323 |
| 100 | 0.4420 | 0.2710 | 0.554 | 0.00442 |
| 50 | 0.3730 | 0.1355 | 0.419 | 0.00746 |
| 0 | 0.2880 | 0.0000 | 0.283 | -- |

Table 43: Soil Profile Data TP-19

| Layer No. | Depth feet | Layer depth | Water Table | Strata Classification | Blow Count N | Friction $\phi$ | Cohesion c |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>4 | 4 | | SP-SM | 6 | 27 | 0 |
| 2 | 4<br>7.5 | 3.5 | 7.5 | SC | 10 | 30 | 0 |
| 3 | 7.5<br>40 | 32.5 | | SC | 12 | 32 | 0 |
| 4 | 40<br>63 | 23 | | CH | 10 | 0 | 2100 |
| 5 | 63<br>79.5 | 16.5 | | SP-SM | 28 | 36 | 0 |

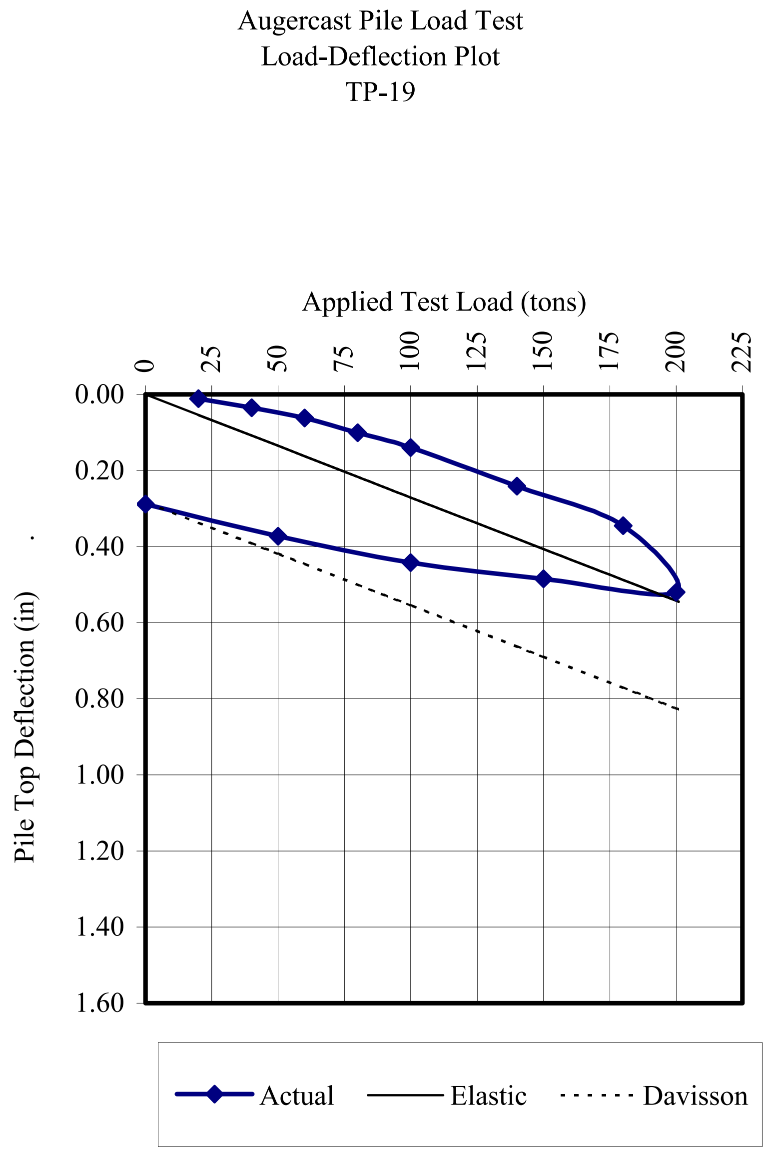


Figure 46: Davisson Offset Plot TP-19

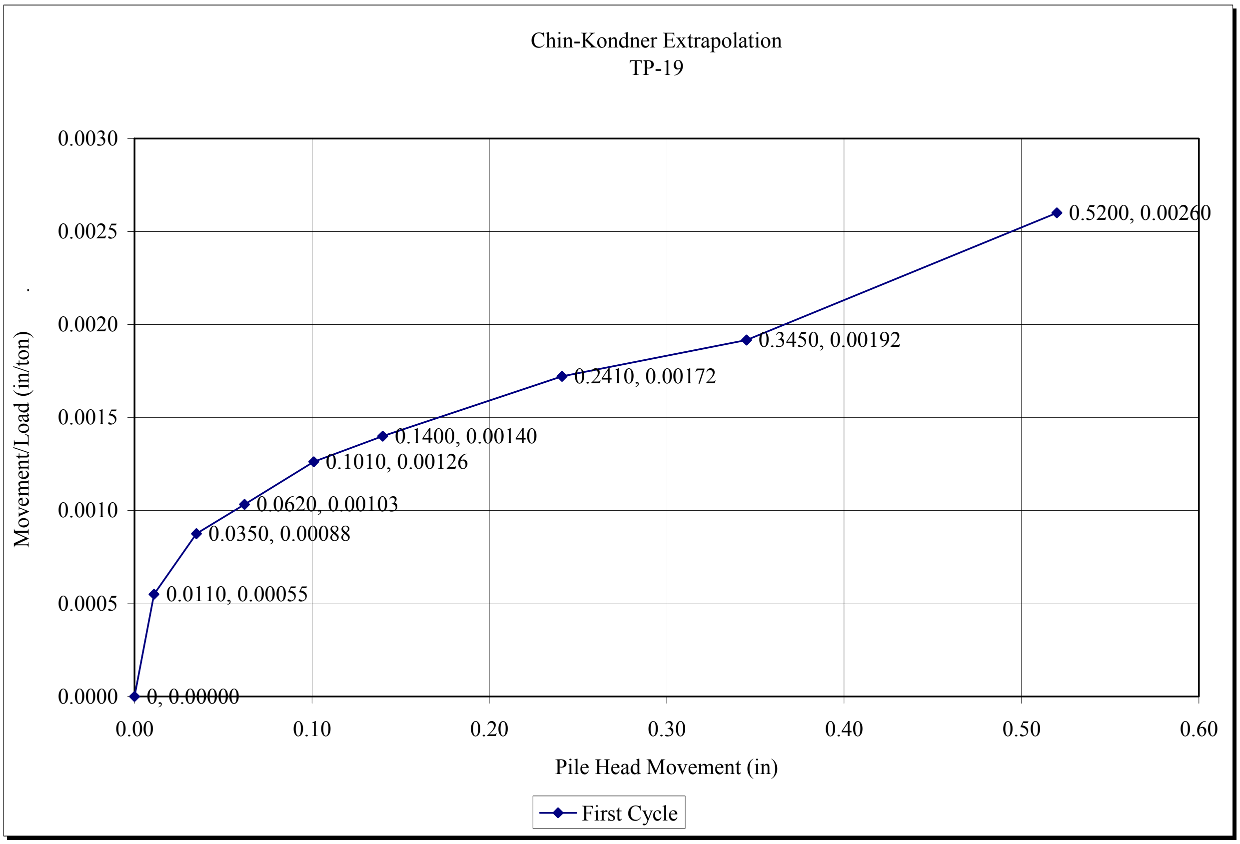


Figure 47: Chin-Kondner Extrapolation TP-19

Table 44: Pile Test Data TP-20

| Test No.: | TP-20 |
|---|---|
| Design Load (Tons): | 75 |
| Pile Diameter (inches): | 16 |
| Pile Area (sq. inches): | 201.06 |
| Pile Length (ft): | 35.5 |
| Pile Compress. Strength (psi): | 4000.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 3457291 |

| Applied Test Load in Q (tons) | Average Measured Pile Top Deflection, D (inches) | Theoretical Pile Elastic Deflection in (inches) | Theoretical Davisson Anticipated Deflection in (inches) | Chin-Kondner Anticipated Deflection D/Q (in/ton) |
|---|---|---|---|---|
| **0** | **0** | 0.0000 | 0.283 | -- |
| 25 | 0.0050 | 0.0306 | 0.314 | 0.00020 |
| 50 | 0.0200 | 0.0613 | 0.345 | 0.00040 |
| 75 | 0.0400 | 0.0919 | 0.375 | 0.00053 |
| 100 | 0.0800 | 0.1226 | 0.406 | 0.00080 |
| 150 | 0.1500 | 0.1839 | 0.467 | 0.00100 |
| 200 | 0.2700 | 0.2451 | 0.528 | 0.00135 |
| 250 | 0.4900 | 0.3064 | 0.590 | 0.00196 |
| 300 | 0.8200 | 0.3677 | 0.651 | 0.00273 |
| 350 | 1.4100 | 0.4290 | 0.712 | 0.00403 |
| 250 | 1.3600 | 0.3064 | 0.590 | 0.00544 |
| 150 | 1.2700 | 0.1839 | 0.467 | 0.00847 |
| 50 | 1.0800 | 0.0613 | 0.345 | 0.02160 |
| 0 | 0.9200 | 0.0000 | 0.283 | -- |

Table 45: Soil Profile Data TP-20

| **Layer No.** | **Depth feet** | **Layer depth** | **Water Table** | **Strata Classification** | **Blow Count N** | **Friction $\phi$** | **Cohesion c** |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>2 | 2 | 2 | SP | 6 | 27 | 0 |
| 2 | 2<br>7.5 | 5.5 | | SP | 10 | 30 | 0 |
| 3 | 7.5<br>10 | 2.5 | | SC | 12 | 32 | 0 |
| 4 | 10<br>27.5 | 17.5 | | CH | 12 | 0 | 2250 |
| 5 | 27.5<br>35.5 | 8 | | SP | 48 | 40 | 0 |

Augercast Pile Load Test
Load-Deflection Plot
TP-20

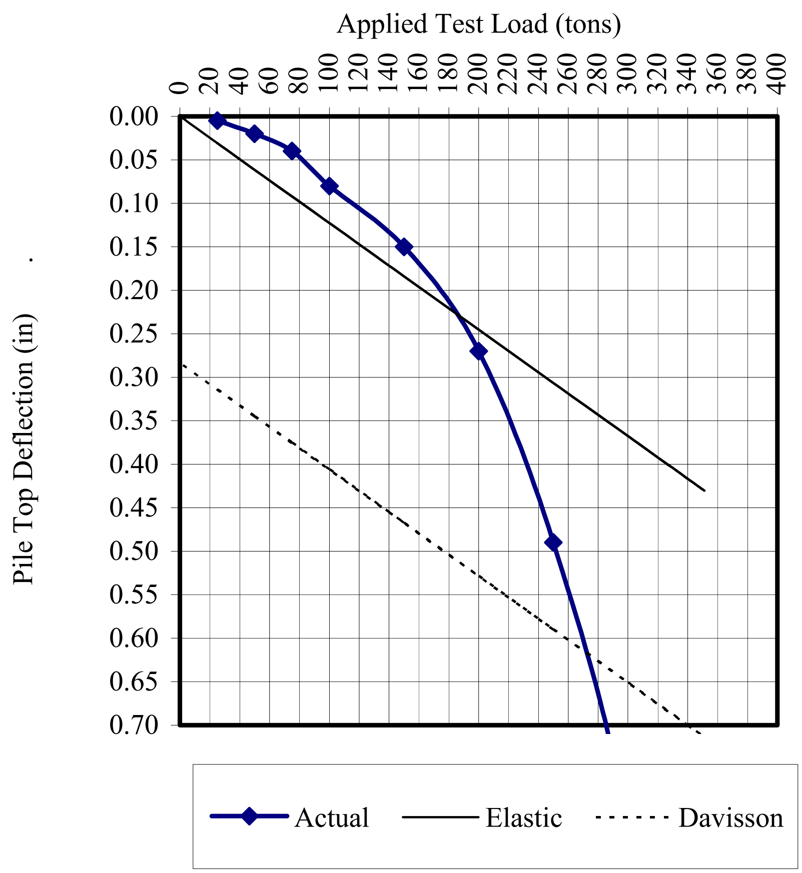


Figure 48: Davisson Offset Plot TP-20

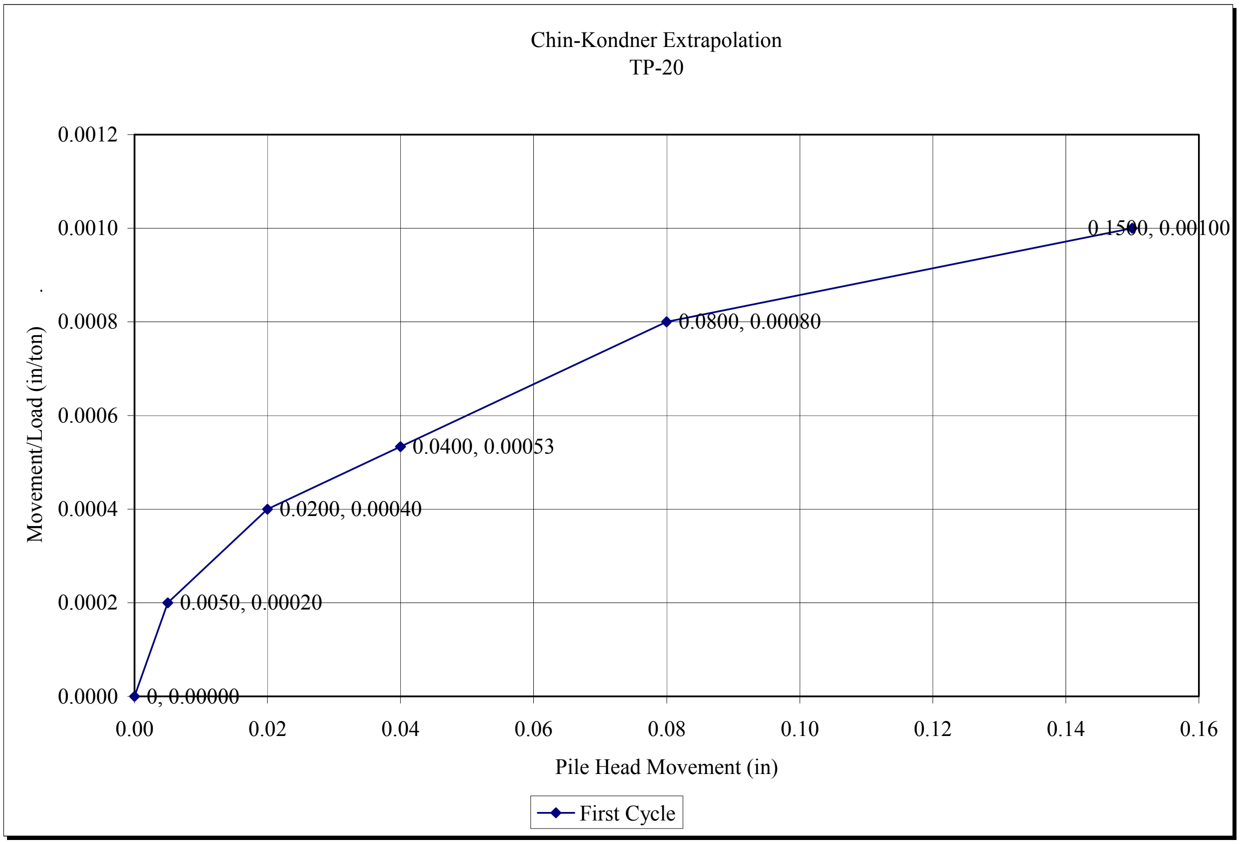


Figure 49: Chin-Kondner Extrapolation TP-20

Table 46: Pile Test Data TP-21

| **Test No.:** | **TP-21** |
|---|---|
| Design Load (Tons): | 100 |
| Pile Diameter (inches): | 16 |
| Pile Area (sq. inches): | 201.06 |
| Pile Length (ft): | 55.0 |
| Pile Compress. Strength (psi): | 4750.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 3767496 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| **0** | **0** | 0.0000 | 0.283 | -- |
| 25 | 0.0100 | 0.0436 | 0.327 | 0.00040 |
| 50 | 0.0250 | 0.0871 | 0.370 | 0.00050 |
| 75 | 0.0500 | 0.1307 | 0.414 | 0.00067 |
| 100 | 0.0850 | 0.1743 | 0.458 | 0.00085 |
| 150 | 0.1450 | 0.2614 | 0.545 | 0.00097 |
| 200 | 0.2100 | 0.3485 | 0.632 | 0.00105 |
| 250 | 0.2800 | 0.4356 | 0.719 | 0.00112 |
| 150 | 0.1900 | 0.2614 | 0.545 | 0.00127 |
| 100 | 0.1500 | 0.1743 | 0.458 | 0.00150 |
| 50 | 0.0850 | 0.0871 | 0.370 | 0.00170 |
| 0 | 0.0620 | 0.0000 | 0.283 | -- |

Table 47: Soil Profile Data TP-21

| **Layer No.** | **Depth feet** | **Layer depth** | **Water Table** | **Strata Classification** | **Blow Count N** | **Friction $\phi$** | **Cohesion c** |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>3 | 3 | 3 | SP-SM | 6 | 27 | 0 |
| 2 | 3<br>12 | 9 | | SP-SM | 12 | 32 | 0 |
| 3 | 12<br>23 | 11 | | CH | 2 | 0 | 500 |
| 4 | 23<br>33 | 10 | | SP | 19 | 34 | 0 |
| 5 | 33<br>38 | 5 | | CH | 3 | 0 | 625 |
| 6 | 38<br>39 | 1 | | SM | 48 | 40 | 0 |

Augercast Pile Load Test
Load-Deflection Plot
TP-21

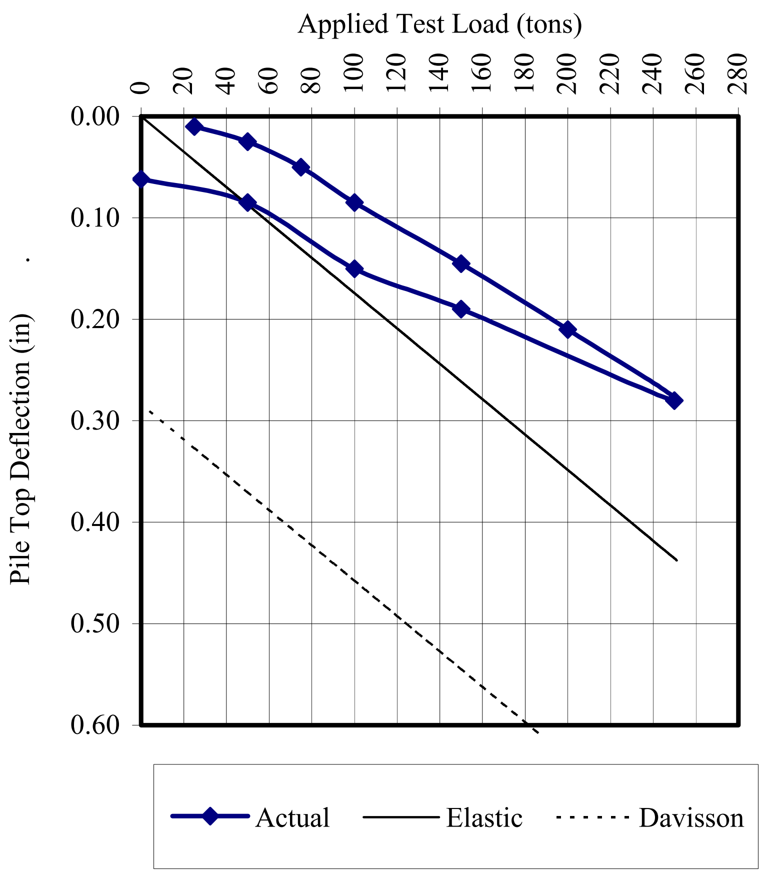


Figure 50: Davisson Offset Plot TP-21

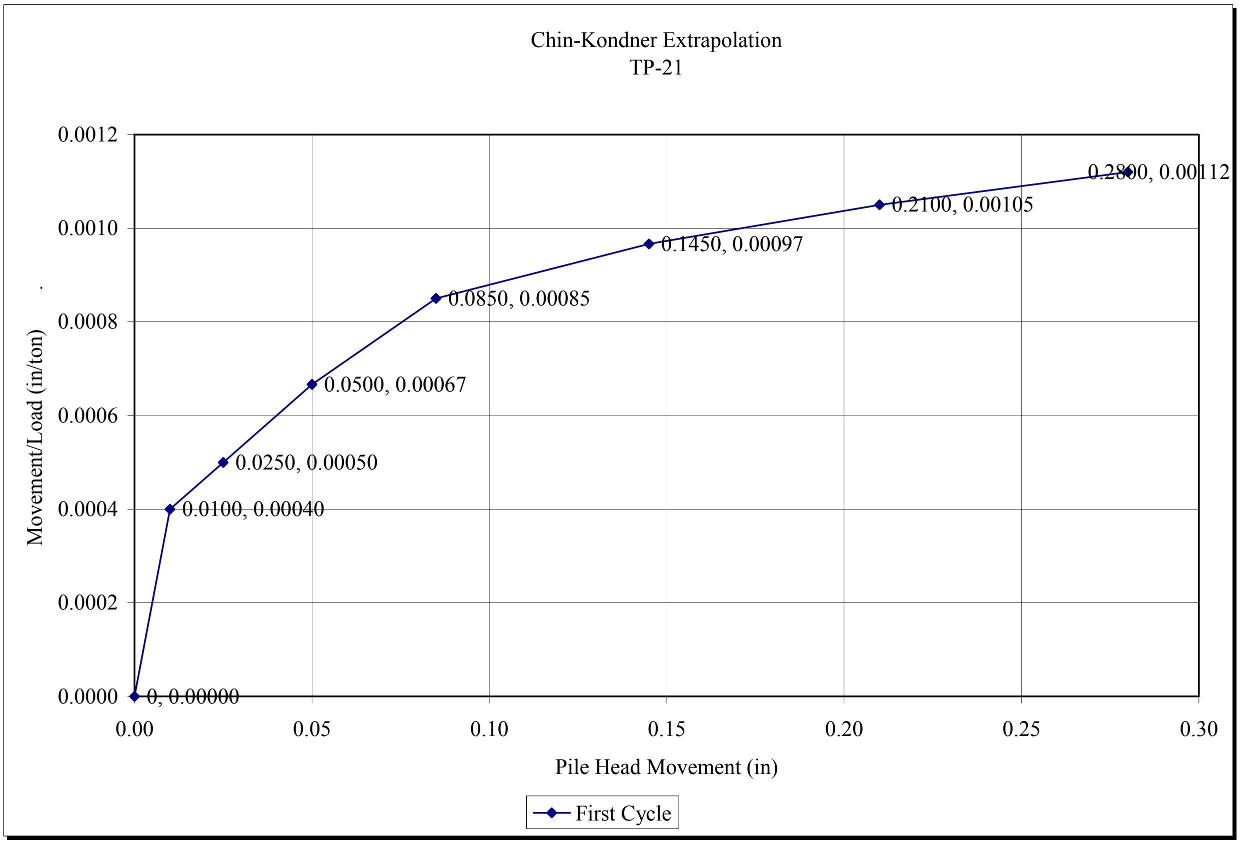


Figure 51: Chin-Kondner Extrapolation TP-21

Table 48: Pile Test Data TP-22

| **Test No.:** | **TP-22** |
|---|---|
| Design Load (Tons): | 100 |
| Pile Diameter (inches): | 16 |
| Pile Area (sq. inches): | 201.06 |
| Pile Length (ft): | 60.0 |
| Pile Compress. Strength (psi): | 4620.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 3715583 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| **0** | **0** | 0.0000 | 0.283 | -- |
| 25 | 0.0140 | 0.0482 | 0.332 | 0.00056 |
| 50 | 0.0370 | 0.0964 | 0.380 | 0.00074 |
| 75 | 0.0650 | 0.1446 | 0.428 | 0.00087 |
| 100 | 0.1100 | 0.1928 | 0.476 | 0.00110 |
| 150 | 0.1920 | 0.2891 | 0.572 | 0.00128 |
| 200 | 0.2790 | 0.3855 | 0.669 | 0.00140 |
| 150 | 0.2750 | 0.2891 | 0.572 | 0.00183 |
| 100 | 0.2370 | 0.1928 | 0.476 | 0.00158 |
| 50 | 0.1740 | 0.0964 | 0.380 | 0.00174 |
| 0 | 0.1510 | 0.0000 | 0.283 | 0.00302 |

Table 49: Soil Profile Data TP-22

| **Layer No.** | **Depth feet** | **Layer depth** | **Water Table** | **Strata Classification** | **Blow Count N** | **Friction $\phi$** | **Cohesion c** |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>2 | 2 | 2 | SP-SM | 6 | 27 | 0 |
| 2 | 2<br>9 | 7 | | SP-SM | 13 | 33 | 0 |
| 3 | 9<br>16 | 7 | | CH | 2 | 0 | 500 |
| 4 | 16<br>31 | 15 | | SP | 40 | 42 | 0 |
| 5 | 31<br>42 | 11 | | CH | 2 | 0 | 500 |
| 6 | 42<br>65 | 23 | | SM | 38 | 40 | 0 |

Augercast Pile Load Test
Load-Deflection Plot
TP-22

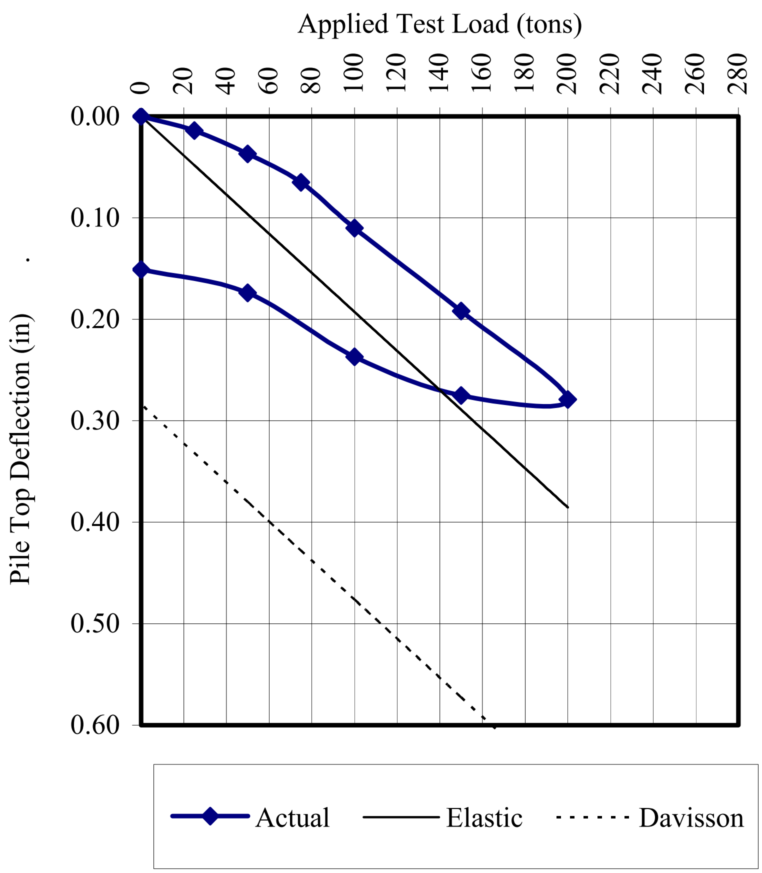


Figure 52: Davisson Offset Plot TP-22

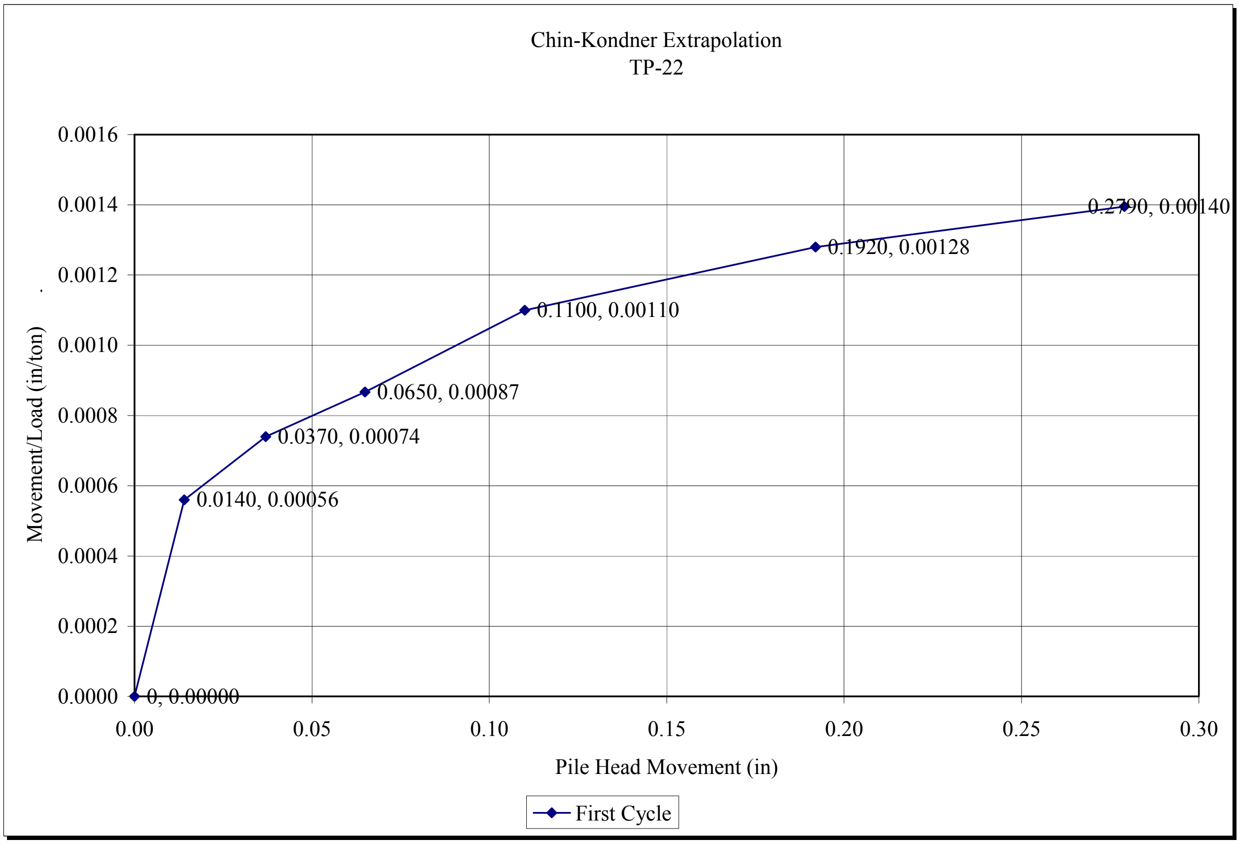


Figure 53: Chin-Kondner Plot TP-22

Table 50: Pile Test Data TP-23

| **Test No.:** | **TP-23** |
|---|---|
| Design Load (Tons): | 100 |
| Pile Diameter (inches): | 16 |
| Pile Area (sq. inches): | 201.06 |
| Pile Length (ft): | 48.0 |
| Pile Compress. Strength (psi): | 7810.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 4830939 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| **0** | **0** | 0.0000 | 0.283 | -- |
| 25 | 0.0130 | 0.0297 | 0.313 | 0.00052 |
| 50 | 0.0320 | 0.0593 | 0.343 | 0.00064 |
| 75 | 0.0610 | 0.0890 | 0.372 | 0.00081 |
| 100 | 0.1210 | 0.1186 | 0.402 | 0.00121 |
| 150 | 0.2010 | 0.1779 | 0.461 | 0.00134 |
| 200 | 0.3130 | 0.2372 | 0.521 | 0.00157 |
| 250 | 0.4190 | 0.2965 | 0.580 | 0.00168 |
| 300 | 0.6820 | 0.3558 | 0.639 | 0.00273 |
| 150 | 0.6410 | 0.1779 | 0.461 | 0.00214 |
| 50 | 0.5620 | 0.0593 | 0.343 | 0.00375 |
| 0 | 0.4400 | 0.0000 | 0.283 | 0.00880 |

Table 51: Soil Profile Data TP-23

| Layer No. | Depth feet | Layer depth | Water Table | Strata Classification | Blow Count N | Friction $\phi$ | Cohesion c |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>1.5 | 1.5 | 1.5 | SP-SM | 6 | 27 | 0 |
| 2 | 1.5<br>4 | 2.5 | | SP-SM | 13 | 33 | 0 |
| 3 | 4<br>12 | 8 | | CH | 2 | 0 | 500 |
| 4 | 12<br>29 | 17 | | SP | 40 | 42 | 0 |
| 5 | 29<br>38 | 9 | | CH | 2 | 0 | 500 |
| 6 | 38<br>48 | 10 | | SM | 38 | 40 | 0 |

Augercast Pile Load Test
Load-Deflection Plot
TP-23

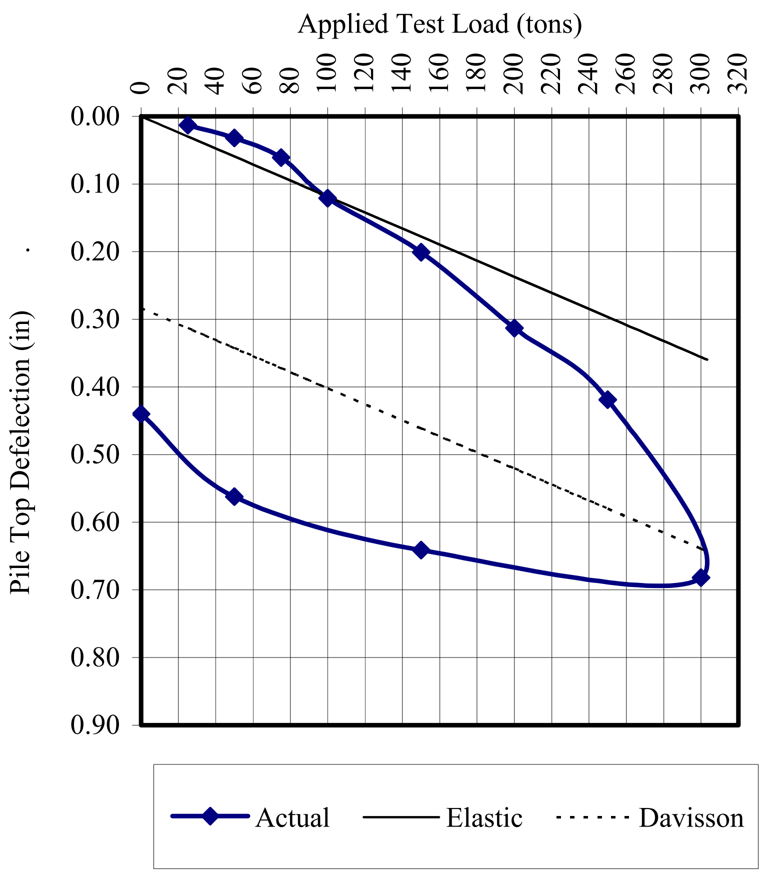


Figure 54: Davisson Offset Plot TP-23

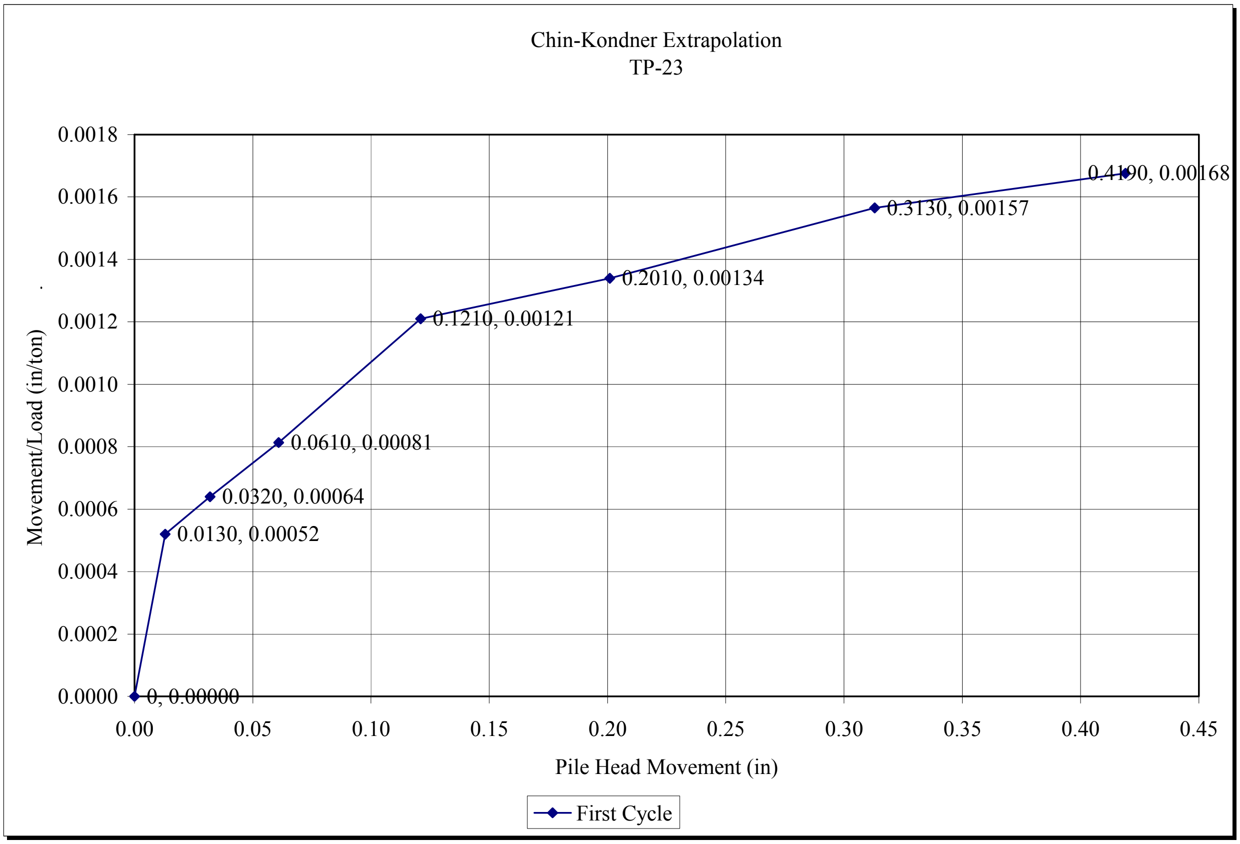


Figure 55: Chin-Kondner Extrapolation TP-23

Table 52: Pile Test Data TP-24

| **Test No.:** | **TP-24** |
|---|---|
| Design Load (Tons): | 80 |
| Pile Diameter (inches): | 16 |
| Pile Area (sq. inches): | 201.06 |
| Pile Length (ft): | 84.0 |
| Pile Compress. Strength (psi): | 3700.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 3325116 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| **0** | **0** | 0.0000 | 0.283 | -- |
| 25 | 0.0300 | 0.0754 | 0.359 | 0.00120 |
| 50 | 0.0620 | 0.1508 | 0.434 | 0.00124 |
| 100 | 0.1300 | 0.3015 | 0.585 | 0.00130 |
| 150 | 0.2010 | 0.4523 | 0.736 | 0.00134 |
| 250 | 0.3400 | 0.7539 | 1.037 | 0.00136 |
| 350 | 0.4950 | 1.0554 | 1.339 | 0.00141 |
| 450 | 0.6500 | 1.3570 | 1.640 | 0.00144 |
| 479 | 0.7300 | 1.4444 | 1.728 | 0.00162 |
| 300 | 0.7100 | 0.9046 | 1.188 | 0.00148 |
| 150 | 0.6800 | 0.4523 | 0.736 | 0.00227 |
| 0 | 0.4600 | 0.0000 | 0.283 | 0.00307 |

Table 53: Soil Profile Data TP-24

| Layer No. | Depth feet | Layer depth | Water Table | Strata Classification | Blow Count N | Friction $\phi$ | Cohesion c |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>5 | 5 | | SP | 7 | 28 | 0 |
| 2 | 5<br>11 | 6 | 11 | SP | 16 | 33 | 0 |
| 3 | 11<br>23 | 12 | | SP | 22 | 35 | 0 |
| 4 | 23<br>36 | 13 | | CH | 2 | 0 | 500 |
| 5 | 36<br>43 | 7 | | SP | 40 | 42 | 0 |
| 6 | 43<br>65 | 22 | | CH | 2 | 0 | 500 |
| 7 | 65<br>84 | 19 | | SM | 38 | 40 | 0 |

Augercast Pile Load Test
Load-Deflection Plot
TP-24

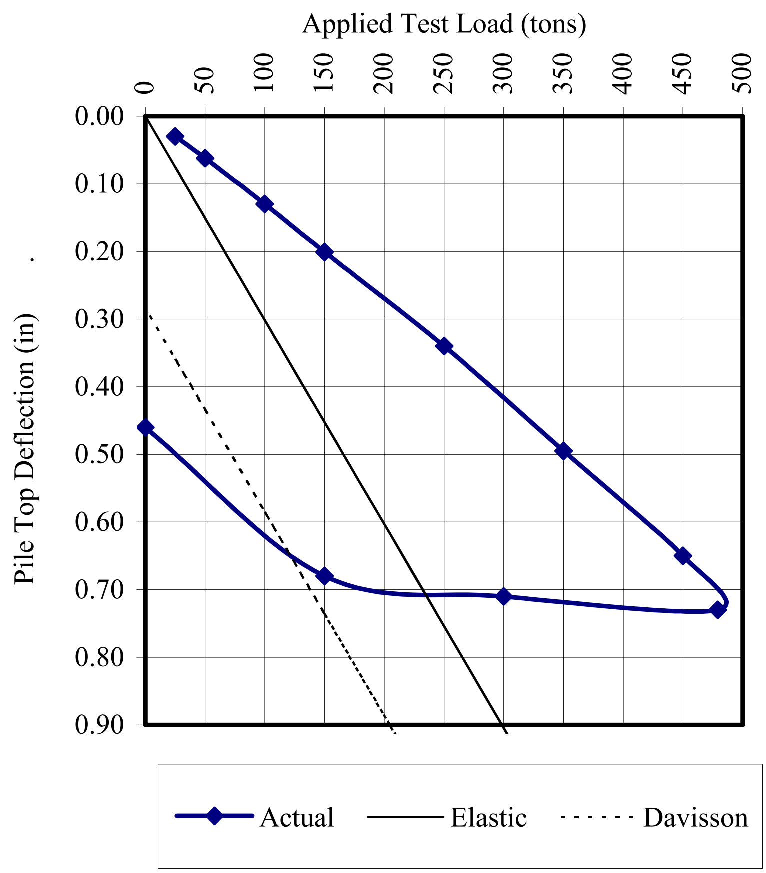


Figure 56: Davisson Offset Plot TP-24

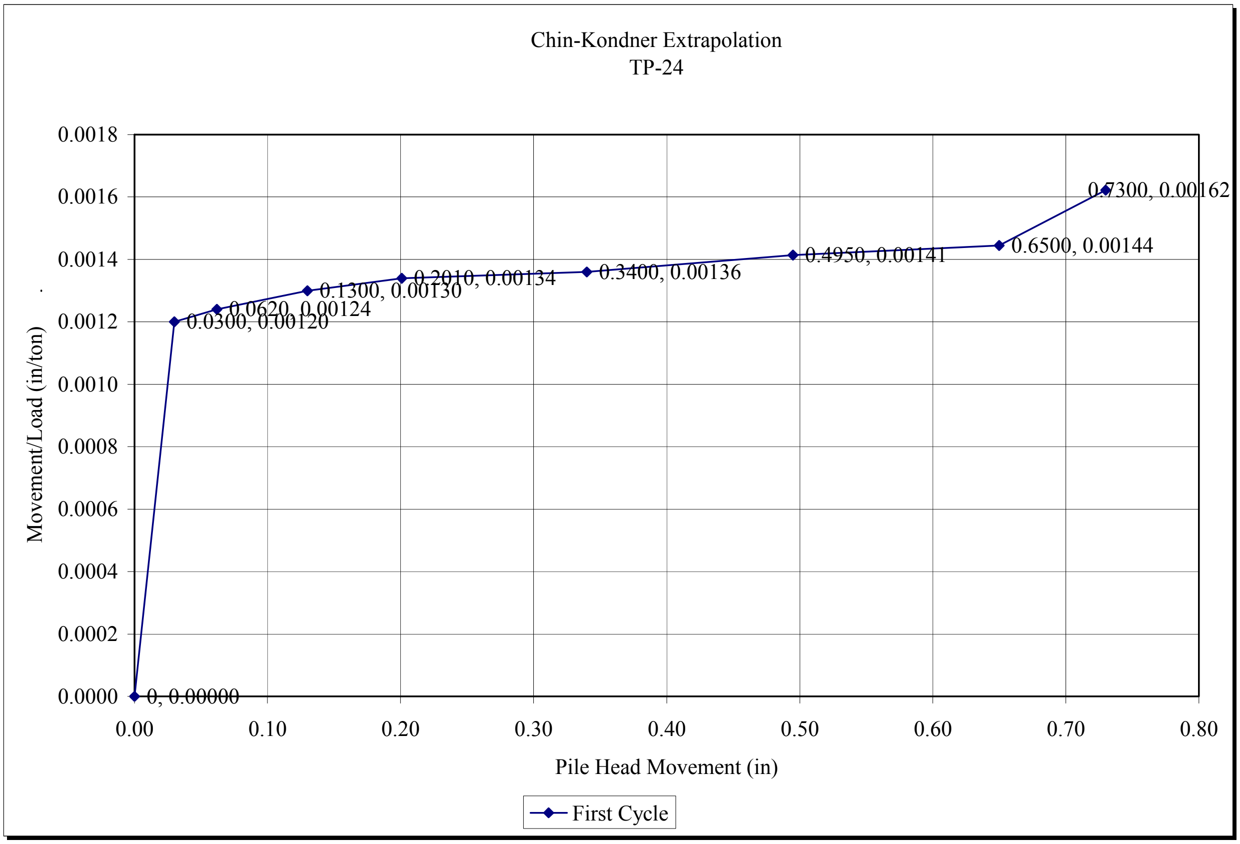


Figure 57: Chin-Kondner Plot TP-24

Table 54: Pile Test Data TP-25

| **Test No.:** | **TP-25** |
|---|---|
| Design Load (Tons): | 80 |
| Pile Diameter (inches): | 16 |
| Pile Area (sq. inches): | 201.06 |
| Pile Length (ft): | 84.0 |
| Pile Compress. Strength (psi): | 3400.00 |
| Pile Unit Weight (pcf): | 140 |
| Pile Modulus of Elasticity (psi): | 3187465 |

| **Applied Test Load in Q (tons)** | **Average Measured Pile Top Deflection, D (inches)** | **Theoretical Pile Elastic Deflection in (inches)** | **Theoretical Davisson Anticipated Deflection in (inches)** | **Chin-Kondner Anticipated Deflection D/Q (in/ton)** |
|---|---|---|---|---|
| **0** | **0** | 0.0000 | 0.283 | 0 |
| 25 | 0.0180 | 0.0786 | 0.362 | 0.00072 |
| 50 | 0.0420 | 0.1573 | 0.441 | 0.00084 |
| 100 | 0.0900 | 0.3146 | 0.598 | 0.00090 |
| 150 | 0.1450 | 0.4719 | 0.755 | 0.00097 |
| 200 | 0.2000 | 0.6291 | 0.912 | 0.00100 |
| 250 | 0.2850 | 0.7864 | 1.070 | 0.00114 |
| 150 | 0.2300 | 0.4719 | 0.755 | 0.00153 |
| 100 | 0.1800 | 0.3146 | 0.598 | 0.00120 |
| 0 | 0.0910 | 0.0000 | 0.283 | 0.00091 |

Table 55: Soil Profile Data TP-25

| Layer No. | Depth feet | Layer depth | Water Table | Strata Classification | Blow Count N | Friction $\phi$ | Cohesion c |
|---|---|---|---|---|---|---|---|
| 1 | 0<br>3 | 3 | | SP | 7 | 28 | 0 |
| 2 | 3<br>7 | 4 | 7 | SP | 16 | 33 | 0 |
| 3 | 7<br>22 | 15 | | SP | 22 | 35 | 0 |
| 4 | 22<br>36 | 14 | | CH | 2 | 0 | 500 |
| 5 | 36<br>46 | 10 | | SP | 40 | 42 | 0 |
| 6 | 46<br>64 | 18 | | CH | 2 | 0 | 500 |
| 7 | 64<br>84 | 20 | | SM | 38 | 40 | 0 |

Augercast Pile Load Test
Load-Deflection Plot
TP-25

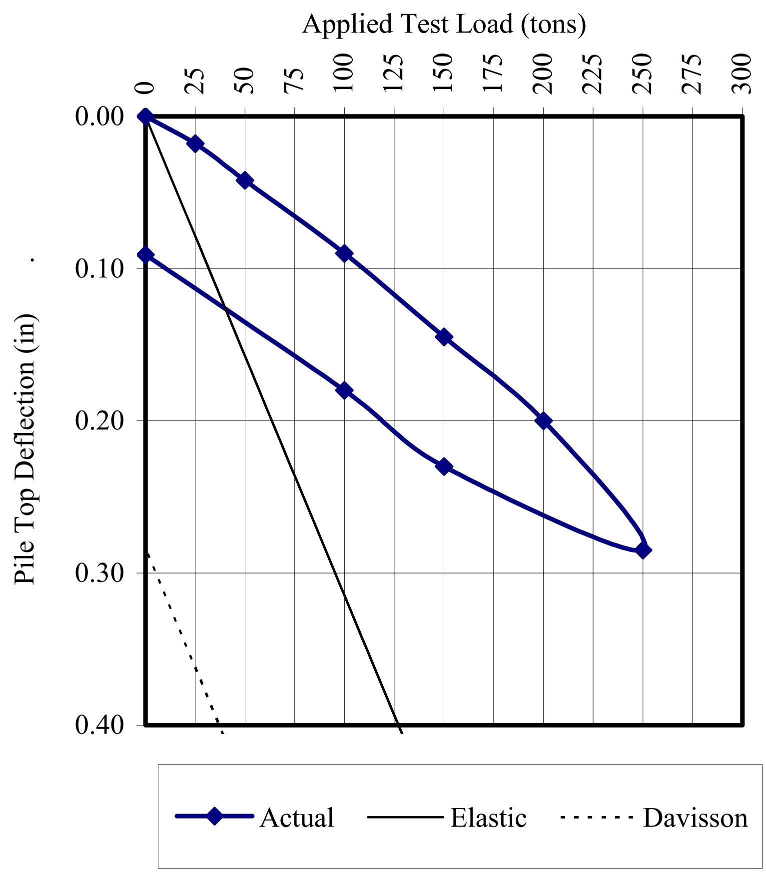


Figure 58: Davisson Offset Plot TP-25

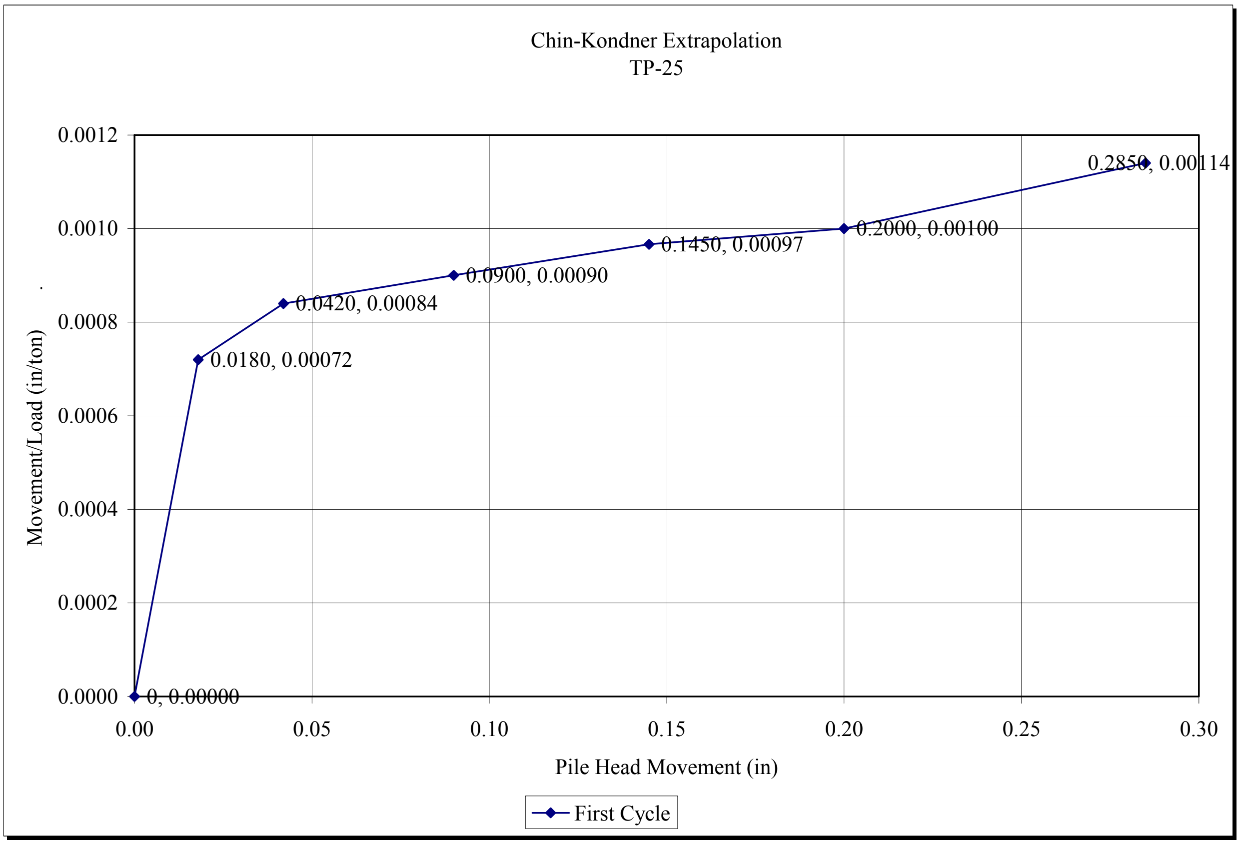


Figure 59: Chin-Kondner Extrapolation TP-25

# APPENDIX B NUMERICAL COMPARISON OF METHODS

Table 56: Predicted Pile Capacity and Settlement

| Test No. | Load tons | Actual Δ in. | NAVDOC *Qu* tons | Meyerhof *Qu* tons | Janbu *Qu* tons | Vesic' *Qu* tons | Theoretical Δ PL/AE" |
|---|---|---|---|---|---|---|---|
| **1** | 100 | 0.2 | 198 | 142 | 216 | 280 | 0.2788 |
| **2** | 8 | 0.25 | 93 | 112 | 150 | 96 | 0.0878 |
| **3** | 200 | 0.176 | 153 | 210 | 253 | 148 | 0.287 |
| **4** | 140 | 0.092 | 224 | 266 | 346 | 235 | 0.1205 |
| **5** | 230 | 0.8 | 110 | 126 | 138 | 138 | 0.2618 |
| **6** | 150 | 0.294 | 145 | 144 | 156 | 156 | 0.2302 |
| **7** | 400 | 0.569 | 329 | 243 | 458 | 375 | 0.8464 |
| **8** | 260 | 0.563 | 269 | 250 | 298 | 260 | 0.9382 |
| **9** | 320 | 0.501 | 244 | 221 | 237 | 237 | 0.5667 |
| **10** | 200 | 1.943 | 212 | 221 | 221 | 148 | 0.1348 |
| **11** | 200 | 0.941 | 224 | 236 | 235 | 163 | 0.319 |
| **12** | 150 | 0.188 | 248 | 259 | 281 | 198 | 0.2881 |
| **13** | 200 | 1.9 | 224 | 232 | 257 | 173 | 0.2852 |
| **14** | 70 | 0.344 | 81 | 142 | 123 | 73 | 0.0919 |
| **15** | 150 | 0.321 | 221 | 282 | 487 | 283 | 0.2977 |
| **16** | 300 | 1.25 | 447 | 408 | 477 | 368 | 0.5955 |
| **17** | 160 | 0.92 | 216 | 261 | 313 | 257 | 0.4309 |
| **18** | 200 | 0.275 | 333 | 286 | 327 | 265 | 0.549 |
| **19** | 200 | 0.293 | 338 | 262 | 369 | 265 | 0.5421 |
| **20** | 350 | 1.41 | 209 | 256 | 188 | 156 | 0.1839 |
| **21** | 250 | 0.28 | 162 | 220 | 196 | 116 | 0.4531 |
| **22** | 200 | 0.279 | 245 | 268 | 327 | 203 | 0.3855 |
| **23** | 300 | 0.682 | 183 | 282 | 283 | 190 | 0.3558 |
| **24** | 479 | 0.73 | 385 | 319 | 366 | 308 | 1.4444 |
| **25** | 250 | 0.285 | 348 | 332 | 373 | 311 | 0.7864 |

Table 57: Test Pile Interpretation Methods

| Test No. | Dav. Qult | Dav. Δin. | Chin-K Qult | Chin-K Q @ 5% | Δin. 5%dia. | Corps Qult | Corps Δin. | WT feet | % Clay |
|---|---|---|---|---|---|---|---|---|---|
| **1** | 100 | 0.546 | 154 | 135 | 0.700 | 100.0 | 0.25 | 3 | 37 |
| **2** | 89 | 0.395 | 227 | 156 | 0.800 | 70.0 | 0.25 | 2.5 | 42 |
| **3** | 200 | 0.570 | 400 | 331 | 0.800 | 200.0 | 0.25 | 2 | 31 |
| **4** | 140 | 0.404 | 345 | 297 | 0.800 | 140.0 | 0.25 | 1.5 | 36 |
| **5** | 220 | 0.750 | 455 | 246 | 0.700 | 120.0 | 0.25 | 5 | 26 |
| **6** | 150 | 0.554 | 175 | 151 | 0.700 | 105.0 | 0.22 | 3 | 41 |
| **7** | 400 | 1.146 | 847 | 499 | 0.900 | 240.0 | 0.25 | 1 | 42 |
| **8** | 260 | 1.205 | 1042 | 360 | 0.700 | 137.0 | 0.25 | 5 | 37 |
| **9** | 320 | 0.883 | 533 | 397 | 1.000 | 230.0 | 0.25 | 9 | 52 |
| **10** | 100 | 0.418 | 278 | 157 | 0.800 | 115.0 | 0.32 | 3.5 | 35 |
| **11** | 150 | 0.529 | 467 | 216 | 0.800 | 105.0 | 0.27 | 3 | 39 |
| **12** | 100 | 0.192 | 334 | 220 | 0.800 | 121.0 | 0.28 | 2.5 | 27 |
| **13** | 100 | 0.420 | 318 | 177 | 0.800 | 90.0 | 0.28 | 3 | 29 |
| **14** | 70 | 0.359 | 132 | 92 | 0.700 | 50.0 | 0.25 | 2 | 40 |
| **15** | 150 | 0.598 | 324 | 251 | 0.900 | 135.0 | 0.25 | 7 | 29 |
| **16** | 150 | 0.600 | 526 | 219 | 0.900 | 75.0 | 0.25 | 13 | 32 |
| **17** | 160 | 0.714 | 257 | 206 | 0.800 | 134.0 | 0.27 | 7.5 | 31 |
| **18** | 200 | 0.832 | 410 | 313 | 0.800 | 190.0 | 0.25 | 6 | 26 |
| **19** | 200 | 0.825 | 313 | 213 | 0.800 | 164.0 | 0.30 | 7.5 | 29 |
| **20** | 275 | 0.620 | 216 | 200 | 0.800 | 195.0 | 0.35 | 2 | 49 |
| **21** | 250 | 0.719 | 375 | 328 | 0.800 | 210.0 | 0.25 | 3 | 41 |
| **22** | 200 | 0.669 | 315 | 264 | 0.800 | 180.0 | 0.25 | 2 | 28 |
| **23** | 300 | 0.639 | 350 | 292 | 0.800 | 212.0 | 0.32 | 1.5 | 35 |
| **24** | 479 | 1.728 | 714 | 348 | 0.800 | 190.0 | 0.25 | 11 | 42 |
| **25** | 250 | 1.070 | 637 | 415 | 0.800 | 230.0 | 0.25 | 7 | 38 |

# APPENDIX C GRAPHICAL COMPARISONS

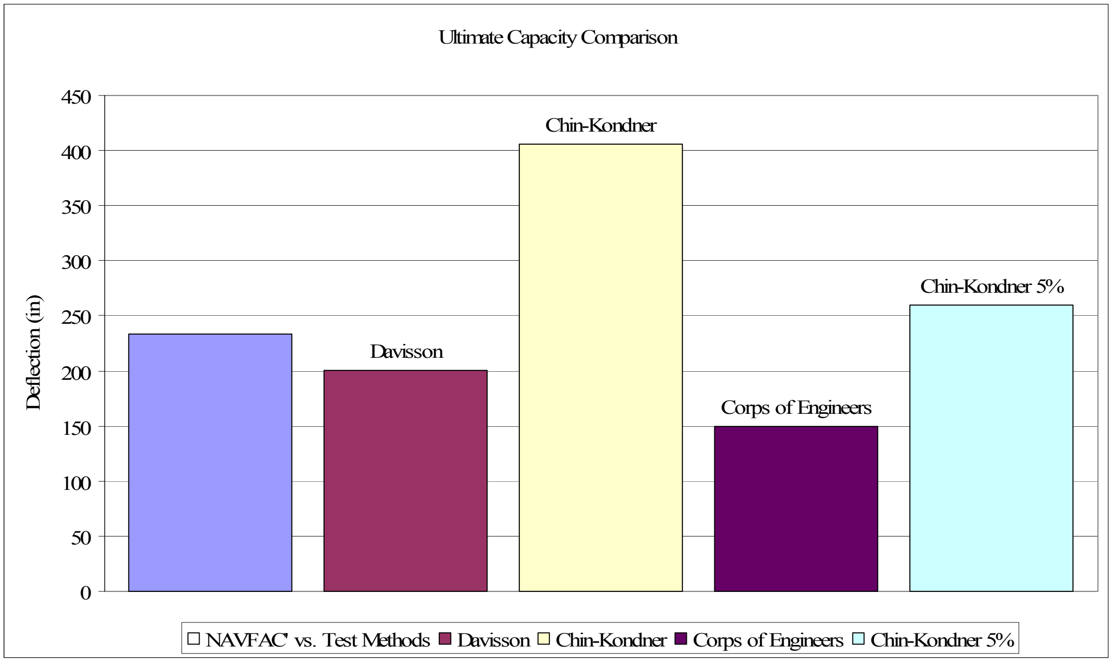


Figure 60: NAVFAC vs. Test Interpretation Methods

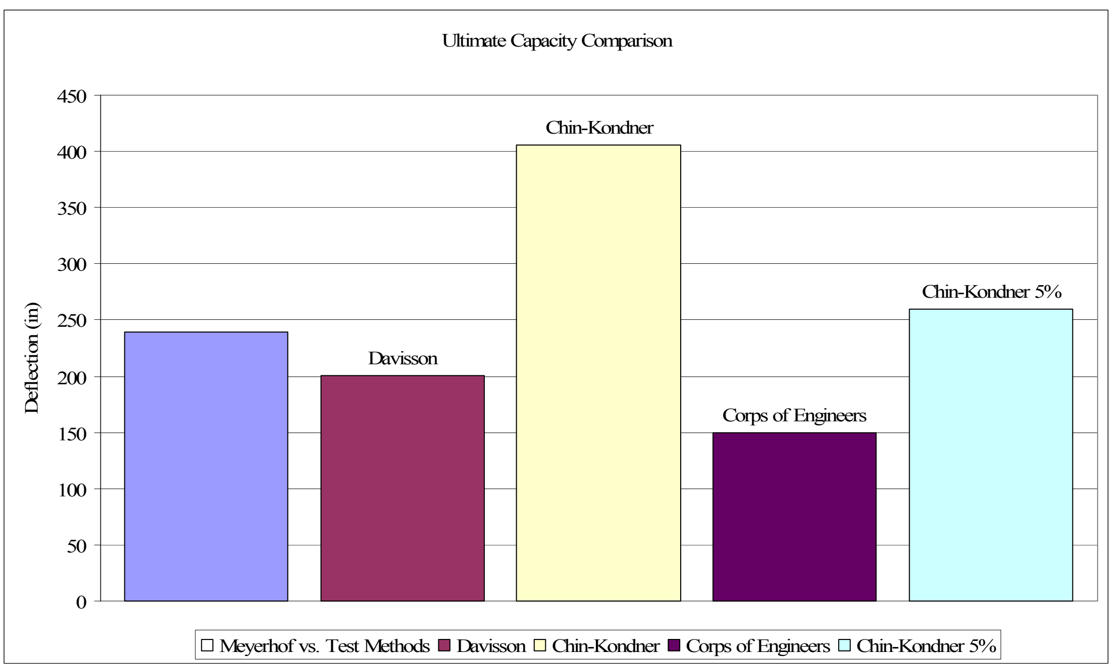


Figure 61: Meyerhof vs. Test Interpretation Methods

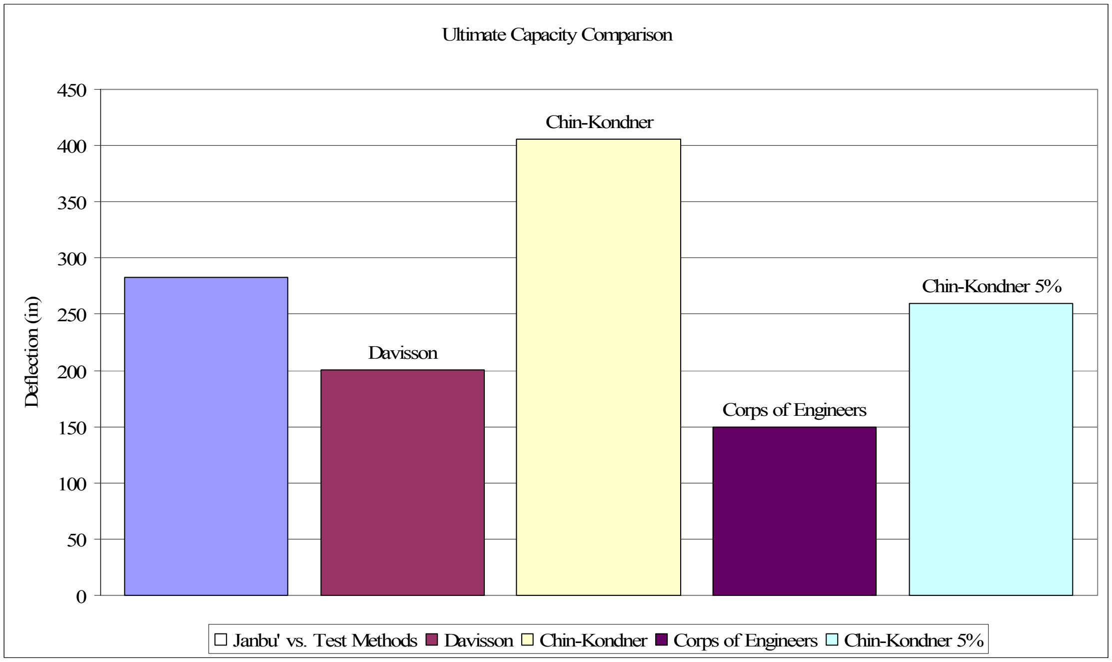


Figure 62: Janbu vs. Test Interpretation Methods

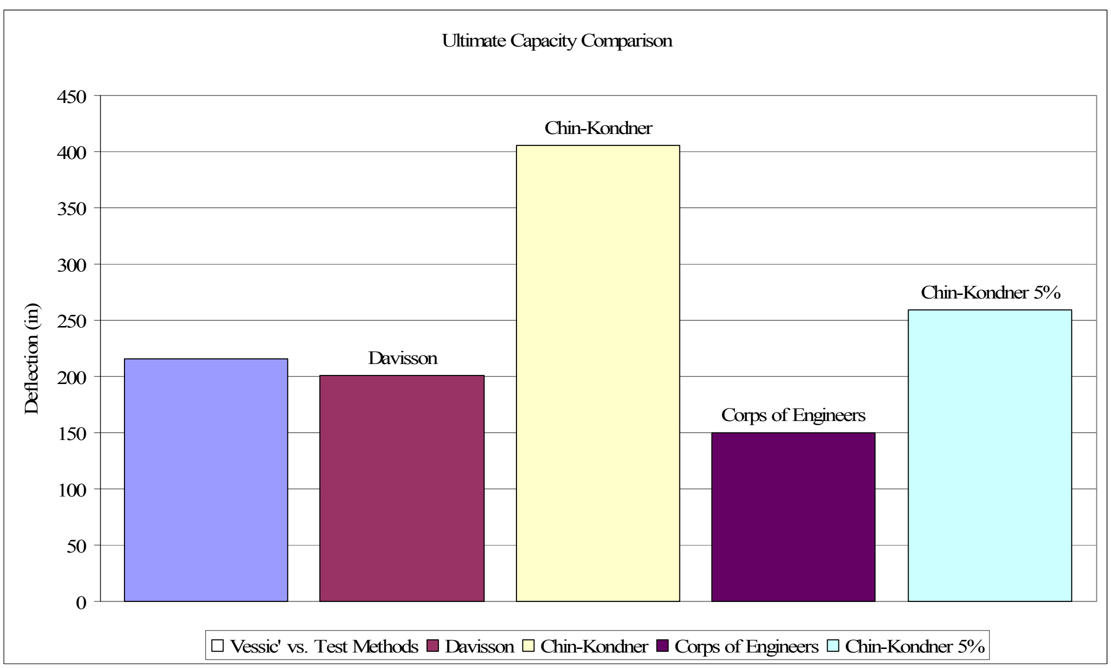


Figure 63: Vesic' vs. Test Interpretation Methods

# APPENDIX D SPSS OUTPUT EXAMPLE
# JANBU vs. ARMY CORPS METHOD

Table 58: SPSS Output

```
DATASET ACTIVATE DataSet4.
CROSSTABS
/TABLES=METHOD  BY LOADCAPACITY
/FORMAT= AVALUE TABLES
/STATISTIC=CHISQ ETA CORR
/CELLS= COUNT
/COUNT ROUND CELL .
```

Crosstabs

**Notes**

| | | |
|---|---|---|
| **Output Created** | | 4-APR-2008 19:01:07 |
| **Comments** | | |
| **Input** | **Active Dataset** | DataSet4 |
| | **Filter** | <none> |
| | **Weight** | <none> |
| | **Split File** | <none> |
| | **N of Rows in Working Data File** | 50 |
| **Missing Value Handling** | **Definition of Missing** | User-defined missing values are treated as missing. |
| | **Cases Used** | Statistics for each table are based on all the cases with valid data in the specified range(s) for all variables in each table. |
| **Syntax** | | CROSSTABS<br>/TABLES=METHOD BY LOADCAPACITY<br>/FORMAT= AVALUE TABLES<br>/STATISTIC=CHISQ ETA CORR<br>/CELLS= COUNT<br>/COUNT ROUND CELL . |
| **Resources** | **Processor Time** | 0:00:00.28 |
| | **Elapsed Time** | 0:00:00.32 |
| | **Dimensions Requested** | 2 |
| | **Cells Available** | 174876 |

```
[DataSet4]
```

**Case Processing Summary**

| | Cases | | | | | |
|---|---|---|---|---|---|---|
| | Valid | | Missing | | Total | |
| | N | Percent | N | Percent | N | Percent |
| **METHOD * LOADCAPACITY** | 50 | 100.0% | 0 | .0% | 50 | 100.0% |

| Chi-Square Tests | | | |
|---|---|---|---|
| | Value | df | Asymp. Sig. (2-sided) |
| **Pearson Chi-Square** | 50.000(a) | 24 | .281 |
| **Likelihood Ratio** | 69.315 | 24 | .011 |
| **Linear-by-Linear Association** | 19.901 | 1 | .000 |
| **N of Valid Cases** | 50 | | |
| a 94 cells (100.0%) have expected count less than 5. The minimum expected count is .50. | | | |

| Directional Measures | | | |
|---|---|---|---|
| | | | Value |
| **Nominal by Interval** | **Eta** | **METHOD Dependent** | 1.000 |
| | | **LOADCAPACITY Dependent** | .637 |

| Symmetric Measures | | | | | |
|---|---|---|---|---|---|
| | | Value | Asymp. Std. Error(a) | Approx. T(b) | Approx. Sig. |
| **Interval by Interval** | **Pearson's R** | -.637 | .066 | -5.730 | .000(c) |
| **Ordinal by Ordinal** | **Spearman Correlation** | -.667 | .078 | -6.197 | .000(c) |
| **N of Valid Cases** | | 50 | | | |
| a Not assuming the null hypothesis. | | | | | |
| b Using the asymptotic standard error assuming the null hypothesis. | | | | | |
| c Based on normal approximation. | | | | | |

## APPENDIX E STATEMENT OF CHI-SQUARED HYPOTHESIS

The null hypothesis and the alternate hypothesis are created for each comparison to determine if a statistical difference exists. The hypothesis statements presented for each comparison group as follows:

1). $H_o$ no difference exists between the NAVFAC method and the Davisson Offset method, $H_1$ a difference exists between the NAVFAC method and Davisson Offset method.

2). $H_o$ no difference exists between the NAVFAC method and the Chin-Kondner Extrapolation, $H_2$ a difference exists between the NAVFAC method and the Chin-Kondner Extrapolation.

3). $H_o$ no difference exists between the NAVFAC method and the Chin-Kondner Extrapolation at 5%, $H_3$ a difference exists between the NAVFAC method and the Chin-Kondner Extrapolation at 5%.

4). $H_o$ no difference exists between the NAVFAC method and the Army Corps method, $H_4$ a difference exists between the NAVFAC method and the Army Corps method.

5). $H_o$ no difference exists between the Meyerhof method and the Davisson Offset method, $H_5$ a difference exists between the Meyerhof method and Davisson Offset method.

6). $H_o$ no difference exists between the Meyerhof method and the Chin-Kondner Extrapolation, $H_6$ a difference exists between the Meyerhof method and the Chin-Kondner Extrapolation.

7). $H_o$ no difference exists between the Meyerhof method and the Chin-Kondner Extrapolation at 5%, $H_7$ a difference exists between the Meyerhof method and the Chin-Kondner Extrapolation at 5%.

8). $H_o$ no difference exists between the Meyerhof method and the Army Corps method, $H_8$ a difference exists between the Meyerhof method and the Army Corps method.

9). $H_o$ no difference exists between the Janbu method and the Davisson Offset method, $H_9$ a difference exists between the Janbu method and Davisson Offset method.

10). $H_o$ no difference exists between the Janbu method and the Chin-Kondner Extrapolation, $H_{10}$ a difference exists between the Janbu method and the Chin-Kondner Extrapolation

11). $H_o$ no difference exists between the Janbu method and the Chin-Kondner Extrapolation at 5%, $H_{11}$ a difference exists between the Janbu method and the Chin-Kondner Extrapolation at 5%.

12). $H_o$ no difference exists between the Janbu method and the Army Corps method, $H_{12}$ a difference exists between the Janbu method and the Army Corps method.

13). $H_o$ no difference exists between the Vesic' method and the Davisson Offset method, $H_{13}$ a difference exists between the Vesic' method and Davisson Offset method.

14). $H_o$ no difference exists between the Vesic' method and the Chin-Kondner Extrapolation, $H_{14}$ a difference exists between the Vesic' method and the Chin-Kondner Extrapolation.

15). $H_o$ no difference exists between the Vesic' method and the Chin-Kondner Extrapolation at 5%, $H_{15}$ a difference exists between the Vesic' method and the Chin-Kondner Extrapolation at 5%.

16). $H_o$ no difference exists between the Vesic' method and the Army Corps method, $H_{16}$ a difference exists between the Vesic' method and the Army Corps method.

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