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CDOT FOUNDATION DESIGN PRACTICE AND LRFD STRATEGIC PLAN

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The Colorado Department of Transportation (CDOT) has adopted the AASHTO LRFD Bridge Design Specifications for the design of all highway bridges. This prompt implementation of the LRFD code was possible because of the history of LFD and ASD designs and the uniformity of the structural materials, concrete and steel. The implementation of geotechnical LRFD is hampered by the non-uniformity of the geological materials, soils and rocks, which requires the evaluation of the state-specific resistance factors. The process of evaluation and calibration of resistance factors is complex and time-consuming. It is based on the theory of probability and reliability. It requires the creation of a property database through a long-term data collection, the evaluation of property statistics, the formulation of probability density function for each property of each geological material, the determination of probability of failure, and the selection of reliability of index. In civil engineering designs the failure probability is determined by a code committee and, in the AASHTO code, the failure probability is determined to be 1/10,000.

Fostering a smooth process for the design of bridges and bridge foundations requires the implementation of both structural and geotechnical LRFD procedures. The FHWA has set the Year 2007 as the target time for the full implementation of the AASHTO LRFD Bridge Design Specifications and it is urgent for CDOT to devote effort to the implementation of geotechnical LRFD, which requires both financial and time commitment from all concerned parties, particularly the CDOT Central Administration. This study has found that the CDOT geotechnical practice is severely deficient in manpower and field test equipment. Without immediate remedial actions the quality of CDOT geotechnical services will be of great concern. The formation of an LRFD Committee with members from the CDOT Administration, Bridge Branch, Materials Laboratory (Geotechnical and Soil-Rockfall Programs), and Research Branch; structural and geotechnical engineering communities; and academia is recommended to strategize the implementation of the geotechnical LRFD procedures.

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EXECUTIVE SUMMARY

Load and resistance factor design (LRFD) was adopted by AASHTO to replace the allowable stress design method (ASD). As opposed to the ASD, where all uncertainties are accounted for in a factor of safety, the LRFD approach applies separate factors to account for each uncertainty in load and resistance. This provides a reliable and rational approach consistent with the safety requirement for highway structural and substructure designs. In the LRFD method, the external loads are multiplied by load factors, while the soil resistances are multiplied by resistance factors. A limit state is a condition beyond which a structural component ceases to fulfill it design function. The limit states, which must be evaluated in the AASHTO LRFD specification, include strength and service limit states. The strength limit state ensures that the design procedure provides adequate resistance (or margin of safety) against geotechnical and structural failures. The service limit state ensures that the function of the structure under normal service conditions performs satisfactorily (i.e., deformations is less than its tolerance). Hence, the foundation design procedure requires the estimation of the nominal response (ultimate strength and deformation) of the highway foundation when subjected to loading.

Foundation deformations can be evaluated from in-situ load tests and analytical methods (e.g., the finite element method and simple geotechnical analysis). Since the evaluation of foundation displacements by LRFD are performed in accordance with the service limit state, where load and resistance factors are both equal to unity, the methodologies used to estimate settlement and lateral deflection are identical for LRFD and ASD. The implementation of LRFD in CDOT design procedure with strength limit state requires 1) proper evaluation of soil strength and deformability, 2) establishment of the soil property database, and 3) evaluation and calibration of resistance factors.

Some of the methods employed in the prediction of the ultimate soil resistance are empirical, e.g., friction angle based on standard penetration test (SPT) blow counts, whereas others are rational based on classical theoretical soil mechanics (bearing capacity based on measured soil friction angle and cohesion). CDOT uses an empirical formula, Denver Magic Formula (DMF), to estimate the nominal strength of soils and rocks in the pier design as follow:

$$Q_{allowable} = 0.5\ N\ \text{kips/ft}^2 \text{ assuming a factor of safety of 2.5.}$$

The formula has been adopted extensively by the Denver geotechnical community in the design of drilled shafts and driven piles for many decades starting with F. H. Chen (1988). CDOT has adopted it in the deep foundation designs for many years and there has been a lack of documented drilled shaft failures. This implies that the method might be somewhat conservative for the deep foundation design in Colorado and some calibration is needed for the implementation of the method in the LRFD deep foundation design in order to evenly apply the risk factor to the different strength contributing factors. Because of the nature of SPT tests, the blow count most likely reflects the undrained shear strength of soils or soft rocks. It is important to note that CDOT is not the first one to apply this method in assessing the bearing capacity of deep foundations.

In fact, the following formulas for the allowable end bearing capacity were proposed by different researchers and practitioners assuming a factor of safety of 2.5:

1.5 N ksf (Meyerhoff, 1956),

1.0 N ksf (Terzaghi and Peck, 1967),

0.5 N ksf (Reese, Touma, and O'Neill, 1976), and

0.37 N ksf (Strounf and Butler, 1975)

The current DMF adopts a formula similar to the one proposed by Reese, Touma and O'Neill. This formula gears towards ASD, for it predicts the allowable soil and rock resistances using the SPT blow count (N) alone. There is nothing wrong with implementing DMF in the LRFD foundation designs. However, to implement DMF in the LRFD bridge substructure design in Colorado requires the calibration against Colorado soils and rocks. Two possible approaches are suggested. While it deviates somewhat from the recommended FHWA/AASHTO LRFD design approach, it can be calibrated and improved for use in deep foundation designs with data from PDA (pile driving analyzer) and full-scale load tests. For CDOT foundation design practice, two approaches are recommended:

First, stay the course of using DMF. This approach may exert less impact to the current CDOT foundation design practice. However, it still needs systematic calibration. A significant number of load tests are required to generate a database for the ultimate strength of Colorado piers and soils and rocks. The data should include SPT blow count N and soil/rock properties including index properties, strengths, and compressibility. When a sufficient database is established, the strength and compressibility can be related to the blow count, N, and, if necessary, index properties of soils and rocks and used in predicting the bearing capacity and settlement for different foundations. This field ultimate foundation capacity can be compared with the nominal foundation capacity calculated from analytical methods for the purpose of evaluating the resistance factors. Additionally, the correlation between the N value and ultimate strength can be used for a more reliable N value-based design method.

Second, adopt the FHWA/AASHTO recommended LRFD design practices with an attempt to evaluate the Colorado-specific resistance factors. While the approach allows us to gain some technical support from other states, some changes in geotechnical practice in CDOT are needed. This requires new field and laboratory equipment for the evaluation of the strength and deformability of Colorado soils and rocks and the associated resistance factors because of the distinctive characteristics of the subsurface material in the Colorado. Initially, the values of the resistance factors recommended in the AASHTO design specifications can be used, while a designer needs to exercise great caution by taking the unique properties of the Colorado soils and rocks into account. During the transition period the LRFD design can be compared to the ASD. Eventually

Colorado needs to have its own resistance factors for geotechnical and bridge substructure designs.

In sum, the implementation of geotechnical LRFD is logical and unavoidable. The current DMF-based design needs to move toward the LRFD-based DMF design during the transition and eventually the AASHTO LRFD procedures. Internally CDOT has already adopted the AASHTO LRFD Bridge Design Specifications in the Bridge Branch. To allow a smooth design process of bridge superstructures and substructures requires the implementation of geotechnical LRFD. Technically, the LRFD procedures are more rational than the ASD and LFD alternatives, for they apply a uniform risk to all design factors. The FHWA has recommended that all state DOTs implement the LRFD procedures by 2007.

Whichever approach CDOT chooses to enforce, the complete implementation requires significant monetary and time investments. New subsurface investigation procedures, laboratory and field testing equipment, and full-scale foundation load tests are needed. This laboratory/field testing equipment and full-scale testing will provide the information for the database needed for the LRFD implementation. Additionally, a geotechnical LRFD training is needed to facilitate a smooth transition from ASD to LRFD. Careful planning will minimize the potential delay of project delivery. This may require, initially, a parallel design effort using both ASD and LRFD procedures to avoid any delay in project delivery and also provide information for comparison and calibration.

IMPLEMENTATION PLAN

To successfully implement the LRFD procedures in geotechnical investigation and design, the following steps are strongly recommended:

1. Establish a geotechnical LRFD Committee, a think-tank group, to formulate the strategy and plan for an effective implementation. The committee membership should include some responsible persons from the CDOT Central Administration, Bridge Branch, Geotechnical Program, and Soil-Rock Fall Program; one representative from each of the structural and geotechnical consulting industries; and a couple from academia knowledgeable in LRFD, probability and reliability.
2. Hire additional geotechnical engineering and investigation staff required for the quality service delivery, public safety and implementation of LRFD procedures in geotechnical design and investigation.
3. Create a position(s) for foundation engineers, who are responsible for all foundation designs. The person(s) should have the overlapping capability of structural and geotechnical engineers to effectively communicate with both geotechnical and structural staff.
4. Upgrade the field investigation facility by purchasing CPT, PMT, GE, and VST equipment.
5. Activate these excellent laboratory testing apparatuses and place them in a production line to generate urgently needed laboratory data of soils and rocks for the formulation of resistance factors for Colorado-specific geological materials.
6. Select deep foundation as the first foundation type for geotechnical LRFD implementation and formulate detailed procedures for the evaluation of all necessary state-specific resistance factors for all geological materials.
7. Continue to practice DMF but establish an enhancement program for establishing resistance factors and calibration, while exploring the feasibility of using other alternative design methods.
8. Develop in-house full-scale load test capability to check the design recommendation and calibrate the geotechnical LRFD recommendation.

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1. INTRODUCTION

1.1 History of LRFD Development

The fundamental LRFD procedures and criteria were in place in Canada in early 1970's. Later in 1989, AASHTO decided to develop its own LRFD specifications. The 2002 AASHTO LRFD Bridge Design Specifications and the 2001 NHI lecture manual on "Bridge Substructure Design" detail the LRFD procedures and criteria for bridge substructure designs. If any state DOT were to adopt all the resistant factors and load factors as provided, it could simply accept the recommended values of load and resistance factors in its bridge substructure designs. This may not yield reasonable designs for the states with soil and rock properties distinctive from those used in the formulation of the resistance factors. Each state is likely to have some peculiar soils and rocks with properties significantly different because of the distinctive geological environment in which the soils and rocks are formed. Colorado soils and rocks can be quite different from those the code resistance factor values are based. This shows the need for the evaluation of the resistance factors specific to the Colorado soils and rocks.

The Colorado Department of Transportation (CDOT) uses a great number of drilled shafts in supporting bridge abutments and piers. The drilled shaft design becomes an important issue and the drilled shaft LRFD design procedures would assume a high priority in the LRFD implementation. Drilled shafts derive their capacities from end bearing and side friction. In the ASD design, a single factor of safety is used to address the uncertainty of both components of drilled shaft capacity. The Osterberg's Cell load tests show that they seldom reach their ultimate values simultaneously. So it is illogical to impose the same factor of safety. In the LRFD procedures, a uniform risk is applied to all design parameters. The risk is established by a specification committee, not by an individual design engineer. In the AASHTO specifications, the design risk is established at one ten thousandth (or 0.0001). This is much more logical and rational.

1.2 Significance of the Implementation of CDOT Geotechnical LRFD Procedures

The implementation of geotechnical LRFD procedures is critical to the success of the implementation of the AAASHTO LRFD Bridge Design Specifications. CDOT's Bridge Branch has been practicing the AASHTO LRFD procedures for some time. To facilitate the smooth and integrated LRFD bridge superstructure and substructure design, it is logical to accelerate the implementation of the geotechnical LRFD procedures. It is more difficult to implement the Colorado specific geotechnical LRFD because the Colorado specific geological materials make it mandatory to determine their resistance factors in advance of the geotechnical LRFD implementation.

The majority of superstructure construction materials like concrete and structure are manufactured under strict quality control. As a result, the Colorado concrete is quite similar in engineering properties, if not identical, to those from anywhere else in the country or the world. Their properties are quite uniform irrespective of where the concrete and steel are made and the evaluation of their Colorado-specific resistance factors becomes unnecessary. Thus, the implementation of the AASHTO LRFD structure code is much less time-consuming. Meanwhile, geological materials are of natural occurrence and their properties are quite random. Because of the spatial and manufacture process randomness, their properties can be quite different from those of other states and the Colorado-specific resistance factors become a necessity for the implementation of the geotechnical LRFD in Colorado.

The delay in the implementation of geotechnical LRFD is caused by the need for the evaluation of the Colorado-specific resistance factors. Many other states face similar problems. The response from nearly 30 out of 50 states shows that only a limited number of state DOTs have begun their effort toward the implementation of geotechnical LRFD. In fact, Colorado is ahead and can take a leadership role at least among the Rocky Mountain States. The FHWA recommended the year 2007 as the target time for the implementation of geotechnical LRFD.

1.3 Research Goal and Objectives

The goal of this study is to facilitate the CDOT implementation of geotechnical LRFD for bridge substructure and restructure design. To realize this goal requires the accomplishment of the following research objectives:

- Understand the current geotechnical investigation and design practices.
- Prioritize the areas for the implementation of geotechnical LRFD design procedures and identify desirable methods for the strength evaluation of high priority geostructures.
- Identify laboratory and field tests for evaluation of soil/rock properties.
- Identify the enhancement need for geotechnical investigation and testing facilities.
- Evaluate the sufficiency of geotechnical personnel resources.
- Establish the material property database for the formulation of the probability density function.
- Evaluate the resistance factor for each strength parameter of soils and rocks.
- Establish the field test database, if possible, for the calibration of resistance factors.

It is important to understand that the implementation of geotechnical LRFD is a tremendous task that requires significant time and monetary investment on the part of CDOT.

2. FUNDAMENTALS AND DEVELOPMENT OF LRFD SPECIFICATIONS

2.1 Development History of LRFD Specifications

The Ontario Ministry of Transportation (and Communication) decided in late 1970's to develop its own bridge design code rather than continue to use AASHTO Standard Specification for Highway Bridges. It also decided to base its specification on probabilistic limit states. Statistical Reliability of a bridge component is based on the mean values of the applied loads and resistance parameters and their standard deviations. In 1979 the first edition of the Ontario Highway Bridge Design Code (OHBDC) was released to the design community as North America's first calibrated, reliability-based limit state specification. The OHBDC was updated and released in 1983 and 1993 with the companion volume of commentary.

The LFD, and later LRFD, was adopted by AASHTO to replace the Allowable Stress Design Method (ASD) in the 1970's. As opposed to the ASD, where all uncertainty is embedded in the factor of safety (FS), the LRFD approach applies separate factors to account for uncertainties in loads and resistances based on the reliability theory. This provides a reliable and uniform approach for the design of highway structures and achieves more consistent level of safety in structure and substructure designs. In the LRFD method, external loads are multiplied by load factors while the soil resistances are multiplied by resistance factors.

In 1986 a group of five state bridge engineers from California, Colorado, Florida, Michigan and Washington met in Denver and drafted a letter to the Subcommittee on Bridges and Structures including their concerns that the AASHTO Specification was falling behind the times. This sowed the seed for the new LRFD Specification. In August 1986 the NCHRP Project 20-7/31 on "Development of Comprehensive Bridge Specification and Commentary" was initiated. The main project objectives include:

- Review of philosophy of safety and coverage of other specification.
- Review of ASSHTO documents for inclusion in a standard specification.
- Assess the feasibility of a probabilistic-based specification.
- Prepare an outline for a revised AASHTO Specification for Highway Bridge Design."

Three possibilities exist for the task on feasibility:

- Allowable stress design (ASD) treats each load on a structure with equal statistical variability.
- Load Factor Design (LFD) recognizes the difference in statistical variability among different loads by using different multipliers for different loads.
- Reliability-based design, such as the procedure adopted in OHBDC takes into account the statistical variability by using the mean and the standard deviation (or the coefficient of variation) of all loads and resistance parameters. Given a set of loads and resistance parameters the process can calculate the "probability of failure." This probability of failure is "not to exceed 0.0001." Alternatively the

process can target a quantity called “reliability index” which can be related to the probability of failure.

The NCHRP Project 12-33 entitled “Development of Comprehensive Specification and Commentary” was initiated in July of 1988. The project progress schedule was as follows:

- The first draft showing the extent of coverage and organization was released in April 1990.
- The second draft showing preliminary set of load and resistance factors was released in April 1991.
- The third draft with 2,000 comments was released in April 1992.
- The fourth draft was submitted in March 1993 and was accepted as a ballot item at the May 1993 meeting of the Subcommittee of Bridge and Structures. The process featured “two rounds of trial designs.”

This leads to the publication of the “Load and Resistance Factor Design Specification for Highway Bridges,” and further to the effort on the “North American Highway Bridge Design Specification.”

The objective of a design based on reliability theory, or probability theory, is to separate the load distribution from the resistance distribution such that the area of overlap is sufficiently small, say one in 10,000.The probability-based LRFD specification is advantageous in the following areas:

- More uniform level of safety throughout the system.
- Measurement of safety as a function of variability of loads and resistances.
- Designers will have an estimate of the probability of meeting or exceeding the design criteria during the design life.

The major disadvantages are the increased design effort, the start-up cost in re-training and the effort and time required during the transition.

The implementation of an LRFD design code requires:

- Establishment of a targeted transition period for changing from ASD to LRFD design.
- Overlapping knowledge in geotechnical and structural engineering for all design engineers.
- Requirement for all design geotechnical and structural engineers to enhance their knowledge in probability and reliability.

In a reliability-based code, a designer is required to calculate the value of “reliability index” provided by the design and compare it to the code-specified tolerable value. This effort requires a designer, geotechnical or structural, to possess a sound knowledge of probability and reliability. In principle it is possible, through the calibration of load and resistance factors by trial designs, to develop load and resistance factors so that the design process looks very much like the existing load factor design. In conclusion, an LRFD design code requires the development of the load factors and resistance factors such that the overlapping area between the load and resistance probability diagrams is no greater than the value accepted by a design code, such as one in 10,000.

2.2 ASD versus LRFD

As discussed earlier, the ASD procedure collectively accounts for the uncertainty of all design loads and resistances in one single factor of safety, while the LRFD procedure applies a load factor to each load and resistance factor to each resistance (or strength) parameter to account for the uncertainty in loads and resistances. The procedures are compared as follows:

2.2.1 Allowable Stress Design (ASD)

- ASD: $R_n/FS \geq \sum Q_i$
- Resistance ≥ Effects of Loads
- Limitations
  - Does not adequately account for the variability of loads and resistance
  - Does not embody a reasonable measure of strength
  - Subjective selection of factor of safety

2.2.2 Load and Resistance Factor Design (LRFD)

- LRFD: $R = \varphi R_n \geq \sum \eta_i \gamma_i Q_i = Q$

where R_n = nominal strength (e.g. ultimate bearing capacity); $\sum Q_i$ = nominal load effect; FS = factor of safety; R_n = nominal resistance; φ = statistically-based resistance factor; η_i = load modifier to account for ductility, redundancy and operational importance; γ_i = statistically-based load factor; Q_i = load effect.

- Limitations
  - Require the availability of statistical data and probabilistic design algorithms
  - Resistance factors vary with design methods
  - Require the change in design procedure from ASD

- Load and resistance can be modeled by a normal or log normal probability density function based on its distribution characteristics.

2.3 Fundamentals of LRFD

2.3.1 Probability Density Function, Probability of Failure, Reliability, and Reliability Index

Reliability-based design takes into account the statistical variability by using the mean and the standard deviation (or the coefficient of variation) and the probability density functions of all loads and resistance parameters. Given a load, Q (the sum of factored load), and resistances, R, load factors, γ_i's, and resistance factors, φ, can be calculated so that the design meets the required "probability of failure" as specified in the LRFD design guide. This probability of failure is selected by the code development professional through extensive research. It is chosen "not to exceed 0.0001" by AASHTO or other value specified by the code development group. This leads to the application of more rational design factors. Alternatively the process can target a quantity called "reliability index" related to the probability of failure.

An LRFD design code requires the development of the load factors and resistance factors such that the overlapping area between the load and resistance probability diagrams is no greater than one in 10,000, the code accepted probability of failure. So to formulate the LRFD procedures requires the probability density functions (pdf) for all loads and resistance parameters, as sampled in Figures 2.1 and 2.2 for load and resistance factor. This means all loads and resistance parameters are considered random variables and their pdf will most ideally derived from the data collection.

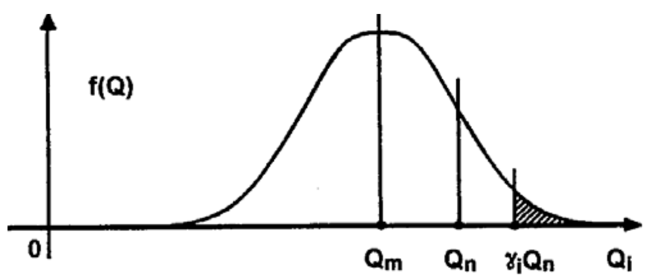

Figure 2.1 Probability Density Function for Load

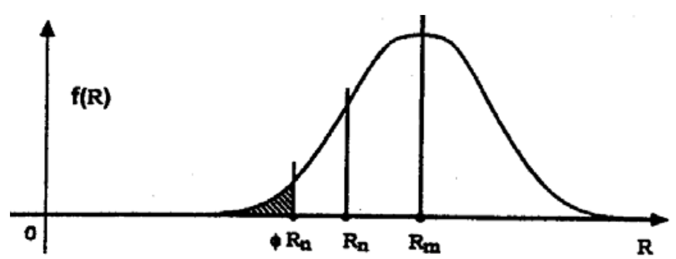

Figure 2.2 Probability Density Function for Resistance

Probability of failure as shown in Figure 2.3 is defined as the probability that a design load, $Q_{m,}$ ($=\sum\eta_i\ \gamma_i\ Q_i$) exceeds a selected value of material resistance, R_m ($= \varphi\ R_n$), or R – Q becomes negative:

$$\text{Probability of failure} = \Pr\ \{\varphi\ R_n \leq \sum\eta_i\ \gamma_i\ Q_i\}$$

where R_n = nominal strength (e.g. ultimate bearing capacity); $\sum Q_i = Q_n$ = nominal load effect; R_n = nominal resistance; φ = statistically-based resistance factor; η_i = load modifier to account for ductility, redundancy and operational importance; γ_i = statistically-based load factor; Q_i = load effect.

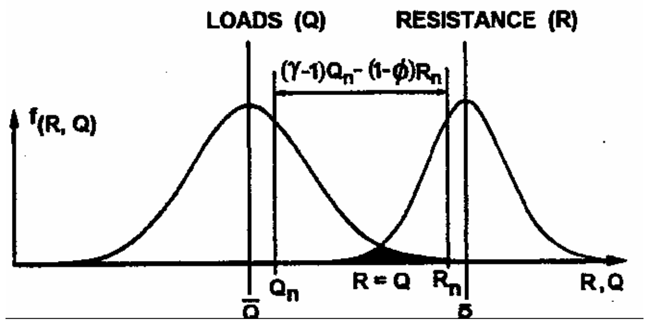

Figure 2.3 Probability of Failure

Reliability expresses the probability of success and is expressed as follow:

Reliability = 1 - Pr (failure)
= probability of success
= Pr $\{\varphi\ R_n \geq \sum \eta_i\ \gamma_i\ Q_i\}$

As shown in Figure 2.4, reliability index, **β,** is defined as the difference between the mean value of resistance and the mean value of design load divided by the standard deviation of the difference between resistance and design load, i.e.

$\boldsymbol{\beta} = (R_m - Q_m)/\sigma_{(R-Q)}$ ………………………………………………………. (1)

where $\sigma_{(R-Q)} = (\sigma_R^{\ 2} + \sigma_Q^{\ 2})^{1/2}$ = standard deviation of (R-Q).

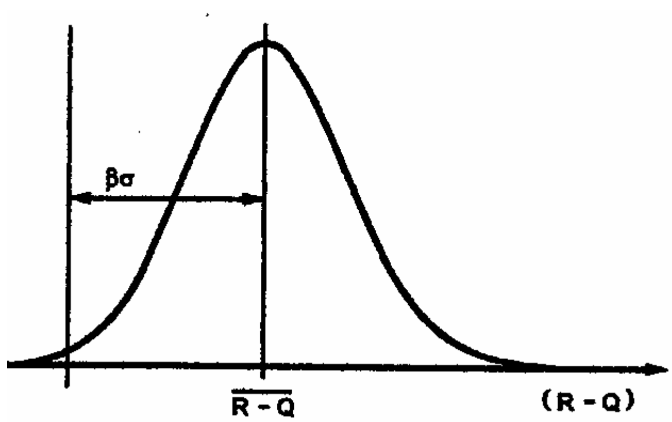

Figure 2.4 Definition of Reliability

For the probability of failure 1 x 10^{-4}, β equals to 3.57 for a lognormal distribution, and 3.72 in normal (or bell shape) distribution. In the pavement design, β of 2.5 is frequently adopted to define the designed service life, i.e. probability of failure of 1 x 10^{-2}.

The designer will have to calculate the value of **β** for the design and then compare the value to the code specified tolerable value. Thus, the designer's sound knowledge of reliability theory will be very beneficial to the LRFD design decision process. It is also possible to develop load and resistance factors by trial designs through calibrating load and resistance factors. The above equation yields:

$$\boldsymbol{\beta} = (R_m - Q_m)/\sigma_{(R\text{-}Q)} \text{ and}$$

$$R_m = Q_m + \boldsymbol{\beta} \, (\sigma_R^{\,2} + \sigma_Q^{\,2})^{1/2} = \lambda \, R \, .$$

Reliability-based design requires that the factored resistance to be equal to or greater than the sum of factored loads, thus:

$$\varphi \, R = Q = \sum \gamma_i \, Q_i.$$

Thus, $R_m = 1/\varphi \, \lambda \sum \gamma_i \, Q_i$ and

$$\varphi = \lambda \, (1/R_m) \sum \gamma_i \, Q_i = \lambda \, (\sum \gamma_i \, Q_i) \, / \, (Q_m + \boldsymbol{\beta} \, (\sigma_R^{\,2} + \sigma_Q^{\,2})^{1/2}).$$

An acceptable value of reliability is specified in the code. A **β** value of 2.0 would imply that approximately 97.3% of the values being included under the bell-shaped curve. Both load and resistance factors must be evaluated. It can be accomplished by choosing the values for load factors and then calculate the value for the resistance factors. Some resistance factors and load factors specified in the AASHTO LRFD Bridge Design Specifications are included in Appendix C.

2.3.2 Limit state Before an appropriate procedure is selected for estimating the nominal strength in an LRFD design specification, the **limit state** must be defined. The limit state is a condition beyond which a structure or its component ceases to fulfill the function for which it is designed. Depending on the type of structures, the limit state includes, but not limited to, **service limit state** (settlement, lateral displacements, etc) and **strength limit state** (bearing capacity, etc). The service limit state is performance-based, which specified the tolerable performance limit for settlement, lateral displacement, and wall tilt, etc. The strength limit state ensures that the design procedure provides adequate resistance (or margin of safety) against structure and substructure failures. Hence, the design procedure of foundations requires the estimation of the nominal response (ultimate strength and deformation) of the highway foundation in service. Foundation deformations (e.g., load-settlement or p-y curve) are established from load tests and/or analytical methods, such as finite element analysis and rational geotechnical analysis).

2.3.3 Bias factor A **bias factor** is the ratio of measured "mean[1]" resistance to the predicted nominal resistance from the method defined by code. A bias factor reflects the effects of all potential sources of errors in evaluating the resistance and therefore is the product of the bias factors from all parameters involved in its evaluation. All parameters are considered random variables. The calibration of resistance factors requires the

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[1] The ratio of mean value to nominal value obtained by following a code-defined procedures. In LRFD method, all loads and resistances are considered "random variables" and, therefore, the term "actual value" does not carry much meaning.

evaluation of the average value of a bias factor (to account for uncertainty due to errors in predicted resistance) and the coefficient of variation of the random variable (to account for uncertainty due to variability in predicted resistance). The values of bias factors and coefficient of variation for the resistance from various types of tests on soils (e.g., SPT, angle of internal friction, PDA, static load tests) have been reported. Due to limited performance data, engineering judgments are used to estimate the statistical parameters of different resistances required in the calibration. Bias factors can be greater than 1.00 because of the variation in testing methods (never two different soils test methods yield the same strength) and the prediction methods for the nominal strength (engineering judgments will have to be exercised for the lack of definitive methods available for its estimate and even the rational methods are not necessarily accurate, like the classical bearing capacity theory for both shallow and deep foundations.) While the estimate from the result of large-scale tests is usually more reliable than the prediction from the empirical rules (like Denver Magic Formula, DMF), it could still vary among cases.

2.3.4 Calibration The process of assigning appropriate values to resistance factors and load factors is called calibration. The resistance factors developed for the 1997a LRFD specifications were calibrated using a combination of reliability theory, fitting to ASD, and engineering judgment. It is generally true in the ASD method that a higher FS is used for empirically based method as opposed to the more rational methods. Calibration by fitting to the ASD method was used in conjunction with reliability-based calibration to ensure that the designs were comparable with accepted engineering practice. In situations when sufficient data were not available, the reliability requirement is relaxed. Calibration using reliability theory is preferred because it permits the selection of a target reliability index that reflects the reliability and the failure probability. The value of resistance factor chosen for a particular design procedure and limit state from a reliability-based calibration can take into account the following factors:

- Variability of the soil and rock properties (geologic conditions),
- Method and extent of field exploration,
- Type, extent, accuracy, conditions, and suitability of in-situ and laboratory testing,
- Type of geotechnical problems, and
- Reliability of methods used for predicting resistance.

The calibration procedures involve the following tasks:

- Selection of geotechnical problems for LRFD development,
- Establishment of the statistical database for resistance parameters of soils and rocks,
- Formulation of resistance models (probability density function),
- Development of the reliability analysis procedures,
- Selection of target reliability index, and calculation of resistance factors.

3. SURVEY OF THE STATUS OF GEOTECHNICAL LRFD IMPLEMENTATION AT DIFFERENT STATE DOTS

3.1 Introduction

The Federal Highway Administration (FHWA) recommended the implementation of the AASHTO LRFD Bridge Design Specifications by the Year 2007. As shown in the survey, state DOT's are in the different stages of the LRFD implementation. Majority either has already implemented or are implementing the structural part of the AASHTO LRFD Bridge Specifications. However, most have not started the implementation of the geotechnical LRFD for bridge substructures and foundations, and are still practicing ASD or LFD procedures. Only very few have fully implemented the geotechnical LRFD procedures, like Oklahoma, South Carolina, Washington, and Florida, etc. It is encouraging to learn that, with only one exception, most respondents indicated the plan to implement geotechnical LRFD by 2007.

3.2 Survey Questionnaires

Under the CDOT sponsorship, the University of Colorado at Denver and Health Sciences Center (UCDHSC) sent the questionnaires with 29 questions to all state DOT's. It was very grateful that the majority of state DOT's (28 out of 50) responded to the questionnaires, although the returns were slower than expected. The respondents took time beyond their regular call of duty to answer the questionnaires. Their efforts are greatly appreciated. The questionnaires are shown in Table A.1. The distribution of questions is as follow: one question each about the size of staff, major geological materials, and organizational structure, two about laboratory and field test facilities, nine about geotechnical investigation, design and business practices, and nine about status of implementation of geotechnical LRFD procedures.

3.3. Survey Results and Discussion

The results of survey on the status of implementation of geotechnical LRFD procedures are shown in Table A.2. Discussions are divided into the following five areas: staff, facilities, geological materials, geotechnical investigation and design practices, and status of LRFD implementation.

3.3.1 Staff

The size of geotechnical staff varies widely among the respondents and it ranges from 2 in Rhode Island and 138 in N.Y. with an average of 32.27. The CDOT geotechnical staff of 20 covering geotechnical engineers, geologists, drilling crews, laboratory technicians, and CAD specialists is much smaller then the average among all respondents and also smaller than Kansas with 43 including two mechanics and Wyoming with 26. This shows that the CDOT Geotechnical and Soil-Rock Fall Programs are **grossly understaffed**. To assume the leadership role in the implementation of geotechnical LRFD among the Rocky Mountain States in the next few years, it is necessary to enhance the geotechnical staff.

3.3.2 Laboratory and Field Testing and Full-Scale Load Test Facilities

CDOT laboratory testing facility is comparable to the average of all respondents. This indicates Colorado has sufficient laboratory equipment for the operation at the current level. When implementing the geotechnical LRFD procedures, it mostly likely needs to enhance the laboratory testing facility to meet the challenge of increased workload.

In the subsurface investigation, most respondent states use standard penetration test (SPT), cone penetration test (CPT), and geophysical exploration (GE) in their subsurface explorations. Undisturbed sampling using Shelby tube is also frequently adopted. Occasionally, trenching and test pits are also used to expose the stratification of the subsurface conditions.

CDOT uses nearly strictly SPT in its exploration program. It is of general practice to begin an exploration program for a large project with geophysical exploration for an approximate delineation of subsurface conditions. The geophysical exploration is then followed by SPT and CPT for more detail identification of the subsoil layers through both visual inspection in SPT and the strength characteristics in both SPT and CPT. The former is not as ideal as a strength indicator, but it provides specimens for visual inspection. The latter is more ideal as a strength indicator, but it doesn't provide specimens for inspection or testing. Performing both in parallel would be very ideal practice for the situation where detail delineation of strength and stratification is necessary. For design pressure-meter tests will be most favorable for assessing the in-situ properties of soils and rocks for use in bridge substructure and foundation designs. PMT is performed in a borehole after the undisturbed sampling. It becomes obvious that CDOT is deficient in the following field test devices:

- Cone penetrometer,
- Pressure meter test device,
- Vane shear test, and
- Geophysical exploration equipment.

The implementation of geotechnical LRFD procedures requires full-scale load test results for calibration of the selected design methodology and the resistance factors. CDOT has in the past performed some deep foundation load tests. The load test effort accelerated because of the TREX project where a great number of drilled shafts are used as support for bridge piers and abutments. CDOT also uses a large number of driven piles as pier and abutment foundations. It is recommended to add the following to the CDOT geotechnical facilities list:

- Deep foundation load test facilities for vertical, lateral, and torsional loads.
- Expand the available PDA capability to cover CAPWAP capability for assessing the vertical load capacity and the load-settlement relationship of driven piles.

Very few states have the capability of performing own load tests. They are usually performed by load test companies. When implemented, CDOT would be able to perform a greater number of tests for LRFD design calibration at a much reduced cost.

3.3.3 Geological Materials

Geological materials vary widely because of different geological formations in different regions. Because of different soils and rocks the responding states have a wide variety of geotechnical problems and use different approach in their geotechnical investigation, design and testing.

3.3.4 Geotechnical Investigation and Design Practices

As shown in Section 3.3.2, CDOT uses nearly exclusively SPT in its subsurface exploration. In terms of geotechnical design, it uses DMF for deep foundation design assisted by PDA. In other words, the CDOT geotechnical investigation equipment is very deficient and it is in a great need of adding some new investigation equipment, like CPT, PMT and GE devices even before the implementation of geotechnical LRFD procedures.

Among the respondent states, about 35 percent of geotechnical investigation and design is contracted out to the geotechnical consulting firms. During the CDOT re-engineering, even higher percentage of geotechnical work was contracted out when Colorado was effluent in funding. Now the World has turned and Colorado is short on cash, it might be necessary to redistribute the engineering investigate and design fee and allocate a greater portion to the internal investigation.

The survey results on retaining walls, bridge foundations, and slope stabilization are summarized as follows:

- Five earth retaining mechanisms are investigated: reinforced concrete cantilever wall (28), gravity wall (5), MSE w/ block facing (24), MSE w/ full-height rigid panel (20), and Misc (5). Two types of MSE walls contribute 44% for total retaining wall practice, while RC cantilever walls still maintains a significant percentage of 28%.
- The bridge foundation support comes in three major types: drilled shaft, driven pile, and shallow foundation with 21, 50, and 22 percent of total bridge foundation practices. The deep foundation together contributes to 71 percent of bridge foundation support. Thus, the deep foundation constitutes the major foundation support for highway bridges.
- Slope stabilization mechanism comes in six different types: benching (20.2), retaining wall (16.1), geosynthetics (11.4), scaling (5), anchors (3.1), and micropile (0.4). Benching and retaining walls are as frequently used to stabilize the retaining walls and geosynthetic slope reinforcement contributes to 11.4% of the slope stabilization practice.

Note: The sum of percentage does not equal 100%. This is due to the data deficiency.

In summary, major geotechnical foundation practices include deep foundation; MSE walls, RC cantilever walls and shallow foundation constitute major foundation practices, which are the areas of emphasis in the implementation of geotechnical LRFD design procedures.

3.3.5 Status of LRFD Implementation

The survey result on the status of the LRFD code implementation is summarized as follows:

- Structural practices
  - LFD: 7 (26.9% of respondents)
  - ASD/LFD: 3 (11.5%)
  - ASD: 3 (11.5%)
  - LRFD: 6 (23.1%)
  - LFD/LRFD: 3 (11.5%)
  - ASD/LRFD: 2 (7.7%)
  - ASD/LFD/LRFD: 2 (7.7%)

  The above statistics reveal among the respondents to this question, only six states have completely implemented the structural LRFD. Fifty percent of the respondents either fully or partially implement the LRFD and other fifty percent have not begun their implementation effort. In other words, near one half of the respondent states have not implemented the AASHTO LRFD Bridge Design specifications for bridge superstructure. In this respect, CDOT Bridge Branch is ahead of absolute majority of the state DOT's in practicing the LRFD code for superstructure design. This seems natural for CDOT Bridge being one of the five original proponents of the LRFD code.

- Geotechnical practices
  - ASD: 16 (59.3% of respondents)
  - LFD: 3 (11.1%)
  - ASD/LFD: 2 (7.4%)
  - ASD/LRFD: 3 (11.1%)
  - LRFD: 3 (11.1%)

  The above statistics reveal that less than 22.2% of the respondent states either have implemented or begun to implement the geotechnical LRFD and more than three quarters of all respondents have not yet attempted the implementation of the geotechnical LRFD for bridge substructure design. This implies that the state DOT's are not rushing to the implementation LRFD code in their geotechnical designs. This may indicate that the implementation of geotechnical LRFD is experiencing significant difficulties. Reasons could include insufficient manpower, equipment and/or the realization of the need for state-specific (or regional) resistance factors for geological materials. All except one respondent intent to implement the geotechnical LRFD in 2007 and one state in 2008. The one exception indicated that they would not implement the geotechnical LRFD unless they were forced upon.

In summary, the implementation of the AASHTO LRFD Bridge Design Specifications is taking shape slowly, particularly the geotechnical LRFD for substructure design. To successfully implement the specifications requires: 1) intensive educational training program, 2) additional manpower, 3) additional equipment for geotechnical investigation and testing. All except two plan to implement the LRFD code by the end of 2007.

4. CDOT GEOTECHNICAL INVESTIGATION, DESIGN PRACTICES AND PERSONNEL RESOURCES

4.1 Geotechnical Personnel Resources

4.1.1 CDOT Geotechnical and Soil-Rockfall Program Organization

Currently, CDOT has twelve professionals working in the geotechnical program and eight in the soil/rock fall program with a total of 20 persons as follows:

- Geotechnical Program has 12 persons including H.C. Liu as the program manager with the following technical teams:
  - Five-geotechnical engineers including Liu.
  - Drilling crew of six technicians (3 senior and 3 junior drillers).
  - One AutoCAD specialist.
- Soils/Rock Fall Program has 8 persons including C. K. Su as the program manager with the following technical teams:
  - Four professional in total (two geotechnical engineers including Su, senior geologist, and geologist). The geologists specialize in rock fall problems.
  - Four soil lab technicians.

In summary, CDOT has seven geotechnical engineers, two geologists, six drillers, four soil laboratory technicians and one CAD specialist.

4.1.2 Geotechnical Personnel Resources Among Neighboring States and Comparison

It is unfortunate to find that only two neighboring states, Kansas and Wyoming, responded to the questionnaires. Kansas DOT has 41 persons in its geotechnical staff and Wyoming DOT 26 with an average between the two states of 33.5. The average from all respondents is 32.3. Both averages are much larger than the CDOT geotechnical staff of 20. These statistics show that CDOT is grossly understaffed. To provide quality and safe geotechnical service, it must have additional staff, particularly when CDOT is anticipating the accelerated effort in the implementation of the geotechnical LRFD.

4.2 Field and Laboratory Testing Facilities

4.2.1 CDOT Existing Geotechnical Investigation Facilities

While CDOT already has significant geotechnical investigation and testing equipment, to effectively implement the LRFD procedures in geotechnical investigation and design requires some additional equipment:

1. Field strength tests
   - Standard Penetration Test (SPT)
   - Cone Penetration Test (CPT)
   - Pressure Meter Test (PMT)
   - Vane Shear Test Device (VST), and
   - Geophysical exploration equipment.
2. Laboratory property and strength tests
   - Unconfined compression test (UCT)
   - Direct shear tests (DST)
   - Triaxial compression test (TCT)
   - CBR tests
   - Vheem Stabilometer test apparatus for R values.

3. Cyclic triaxial test apparatus for resilient modulus test
4. Laboratory compressibility (or swell) test
   - Oedometer tests
   - Denver swell tests
5. Index property and classification
6. Standard (or Modified) Proctor Compaction tests

CDOT has the adequate equipment for the laboratory index property, compaction, permeability, consolidation, and strength tests to maintain the status quo. When the demand for the geotechnical services exceeds the capability to deliver the investigation results for use in the bridge substructure and foundation designs, it is necessary to acquire additional equipment, particularly when seeking the implementation of geotechnical LRFD.

4.2.2 Recommended Additions to New Geotechnical Investigation Equipment

To enhance the quality of geotechnical service and accelerate the implementation of geotechnical LRFD, CDOT must address the problem of the shortage of its field geotechnical investigation and testing equipment. The following are the recommended purchases:

- Cone penetration test (CPT) facility,
- Pressure meter test (PMT) facility,
- Vane shear test (VST) facility,
- Geophysical exploration equipment (GEE), and
- Full-scale load test facility for determining the in-situ load capacity and deformation (or displacement) of drilled shafts and driven piles.

The survey shows that most state DOTs use CPT, PMT, GEE, and, sometime, VST in addition to SPT CDOT uses exclusively. Devices are there for an effective geotechnical investigation and it is puzzling that CDOT still does not use any of the above equipment. In general, the site investigation begins with geophysical exploration to delineate the subsurface strata and is followed by the joint use SPT and CPT to further define the subsurface condition. When grossly undesirable soils or rocks are detected, undisturbed sampling is followed to secure samples for laboratory study. Sometimes, it is necessary to perform vane shear tests to assess the undrained shear strength of clayey soils. If the field mechanical properties are needed, then PMT is performed. The CDOT subsurface investigation will be greatly enhanced with the above equipment purchase.

The full-scale load test facilities should include the equipment for performing load tests of piers and driven piles under vertical, lateral and torsional loads. The full-scale tests will provide the database urgently needed for the calibration of LRFD procedures in Colorado. CDOT has already begun to perform some full-scale tests of deep foundations with the out-of state technical assistance. If CDOT were to develop its own in-house capability for the full-scale tests, more load tests can be performed more cost effectively. In the long run, it would save significantly the cost for conducting such tests under a strict CDOT quality control. It is also recommended to expand the use of PDA to

CAPWAP to assess the vertical load capacity and performance of driven piles. To implement the above recommendation will require the addition of geotechnical staff.

4.2.3 Geotechnical Testing Facilities

The survey shows: 1) CDOT laboratory testing facilities are comparable to those of the questionnaire's respondents. Although some states do have much better facilitated soils testing laboratory; 2) Most state DOTs have much better field investigation facilities. SPT, CPT and GE (geophysical exploration) are routinely used in the subsurface exploration program. CDOT greatly lacks the subsurface exploration equipment, like CPT, GEE, VST, and PMT, as discussed above. To effectively implement the geotechnical LRFD code, CDOT needs to enhance its field exploration facilities.

4.3 Geotechnical Investigation and Design Practices

4.3.1 CDOT Geotechnical Investigation and Design Practices

Currently, CDOT geotechnical investigation and design use nearly exclusively the SPT. The in-house triaxial and direct shear test devices are idled. This could be the result of re-engineering and insufficient manpower to engage the equipment in the geotechnical investigation. To implement the geotechnical LRFD, CDOT needs to activate these apparatuses for the foundation investigation. Besides, as the nation's pavement design increasingly advances toward the mechanistic approach, the need for the capability to test base course, sub-base, subgrade and pavement materials for their resilient moduli will only increase with time. Thus, CDOT needs to acquire the quality cyclic triaxial test capability for the evaluation of the mechanical properties and strength of materials involved in the design and construction of the pavement.

CDOT has based most of its deep foundation design on DMF, where the allowable bearing capacity is assumed to be equal to N/2 with the inherent factor of safety of 2.0 to 2.5. The side frictional capacity is recommended as 10% of the end bearing capacity. First, DMF does not strictly belong to Denver. Many other researchers and practitioners (Meyerhof, 1959; Terzaghi and Peck, 1967; Reese, et al, 1976; and Stroud and Butler, 1975; Chen, 19????) had used similar approaches, as outlined in the Executive Summary. Chen promoted the use of the DMF in the design of drilled shafts in clay and clay shale in Colorado because of its simplicity. However, as the state-of-the-art of geotechnical engineering practices advances, more rational approaches have become available. In fact, the survey of the fifty states in the United States in which 28 states responded reveals that Colorado is the only state uses nearly exclusively the SPT blow count in the geotechnical investigation and design. While this simplistic approach has a good track record of success due to the lack of documented failures, it is certainly a good idea to check "how conservative it is or unconservative it could be." Besides, the DMF approach is geared toward the ASD procedures and needed to be expanded to address the settlement (or serviceability) issues.

The past Colorado experience indicates that the procedure can be either too conservative or unconservative and a more rigorous approach or a calibrated DMF is needed to enhance the rationality of the design method. Deficiencies of the DMF method are summarized in the following:

- The blow count N does not reflect the static property of soils because of the impact nature of the Standard Penetration Test.
- DMF is an ASD design based on the bearing capacity. The settlement has never been addressed. Most deep foundations in CDOT are used for bridge support and settlement becomes critical in the maintenance of sufficient bridge deck clearance. So, the approach needs enhancement for settlement assessment.
- The blow count N does not reflect well the effect of stress history on the strength of clayey soils or clay shale and the N-based DMF might not provide a rational estimate of end bearing strength.
- An effective settlement evaluation method using N value is needed, particularly for bridge foundation on clay and clay shale. The method eventually needs calibration for it effectiveness and reliability.
- After proper calibration against the Colorado soils and rocks, the N value can still be used for bearing capacity and settlement computations, while seeking more rational options.

4.3.2 Geotechnical Investigation and Design Practices in Neighboring States
Kansas and Wyoming are the only two states bordering Colorado that responded to the questionnaires. Neither of them exclusively uses the blow count approach in their design of bridge substructures and other foundations. Wyoming has begun to implement the geotechnical LRFD. In this respect, CDOT is behind and needs to plan for complete LRFD implementation.

4.3.3 Comparison and Recommendations
It was interesting to note that none of the states that returned the survey questionnaires indicated the use of blow count method (or DMF) as practiced in CDOT on a routine basis. Instead, most use the conventional foundation design methods. This points to two possible directions for the future foundation design practice in CDOT:

1) Enhance, calibrate and continue to use the DMF approach, and
2) Examine alternative approaches that are more rational for the foundation investigation and design.

No matter which direction CDOT eventually adopts, it is necessary to develop the in-house capability in the area of field investigation and testing and full-scale foundation load tests.

CDOT leads the PDA effort in Colorado, while local companies begin to develop PDA capability and beyond. To maintain the leadership role, CDOT will need to keep up with and advance beyond its current PDA practices and utilizes all available pile driving technology in the pile design. Field tests, full-scale tests, and pile driving technology will provide the information urgently needed for the development of resistance factors and calibration of foundation design methodology adopted in Colorado. In sum, the following areas are recommended for improvement in CDOT geotechnical investigation and design practices aiming at the full implementation of geotechnical LRFD:

- Hire additional geotechnical engineers and technicians to enhance its current practice, to maintain roadway safety, to perform full-scale load tests, and to prepare for the manpower need when the geotechnical LRFD implementation effort is initiated.

- Enhance the CDOT facilities for laboratory and field investigation and testing.
- Calibrate and enhance the DMF design approach for geotechnical LRFD procedures.
- Select the method for adoption in the implementation of CDOT geotechnical LRFD.
- Evaluate the Colorado-specific resistance factors for the implementation of the AASHTO LRFD Design Specifications for bridge substructures. During the transition, adopt the resistance factors from the AASHTO specifications with caution and use the full-scale load test, whenever possible and the ASD design as calibrators.
- Close communication with the Bridge Branch is needed to foster the smooth transition for the full implementation of the AASHTO LRFD Bridge Design Specifications.
- Close communication with the geotechnical industry and government agencies in Colorado for the geotechnical LRFD implementation.
- Retrain all geotechnical staff in CDOT and the Colorado geotechnical industry through continuing education.

The implementation of a new design concept usually takes time because of the time-consuming human adjustment. The CDOT Bridge is one of the five initiators and pioneers for the development of the LRFD code. It is only reasonable to expect CDOT to take pride in participating in the full implementation of the LRFD code. Besides using more rational procedures, the implementation will enhance CDOT technical communication with agencies at both federal and state levels.

4.4 Geotechnical LRFD Implementation

4.4.1 CDOT Status

CDOT needs to take pride in implementing the LRFD bridge design code. The CDOT Bridge Branch participated in the study on the need for the development of a LRFD bridge design code. It was one of the five pioneer states who met in Denver and decided that the United States needed its own rational bridge design code after the Canada developed and implemented its own LRFD code in 1970's. After a comprehensive study, the committee recommended the development of our own version. The committee action leads to the current AASHTO LRFD Bridge Design Specifications, which is recommended for implementation at all state DOTs. The effort took nearly twenty years. CDOT has adopted the LRFD procedures in the bridge superstructure design. To allow a smooth bridge design process in CDOT, it needs to implement the geotechnical LRFD procedures for bridge substructures. Besides the technical superiority of the LRFD procedures, CDOT will benefit significantly from practicing the national specifications, when the need occurred for the consultation with other state DOTs on technical design issues, when the same design specifications are practiced in all state DOTs.

Because of the distinctive geological materials, it is necessary to select the design methodology, and evaluate the resistance factors for state-specific geological materials when any state decides to adopt the geotechnical LRFD procedures. The evaluation of resistance factors will take time and money. This may be why state DOTs are slow and reluctant in the geotechnical LRFD implementation, as revealed in the survey results. This delay may reflect the implementation difficulty. The extent of geotechnical investigation depends on the following influencing factors:

- Importance of a project.
- Types of structures.
- Types of soils and rocks.
- Types of foundations.

To take advantage of collective wisdom, a geotechnical LRFD implementation committee can be formed to formulate the details. Different issues can be discussed openly in order to draw unbiased conclusions. The implementation strategy is then brought to a meeting with key representatives from engineering consulting companies and government agencies.

It is observed that CDOT geotechnical staff only provides design parameters to the Bridge Branch, which is responsible for the design of superstructures. It is unclear where the foundation design task is positioned. It is important to create a number of positions responsible for foundation design. The positions can be located in either Geotechnical Program or Bridge Branch.

In sum, the CDOT Geotechnical and Soil-Rockfall programs are much understaffed with 20 highly qualified engineers, geologists and experienced technicians. Collectively they are responsible for all subsurface investigation, laboratory soil testing, field investigation and roadway safety, like rock fall and slope slides, etc. The Soils Unit is equipped with excellent production-oriented laboratory testing equipment for index property, R-value (Hveem stabilometer), CBR, Standard Proctor, permeability, consolidation, unconfined compression, direct shear, and triaxial compression and resilient modulus tests, etc. Some high quality laboratory apparatuses, like triaxial test, resilient modulus test, and direct shear test apparatuses are left idle because of the lack of manpower after re-engineering effort. It is critical to the geotechnical LRFD implementation to activate these excellent laboratory testing apparatuses and place them in production line to generate urgently needed laboratory data of soils and rocks for the formulation of resistance factors for Colorado-specifically geological materials.

The field investigation still relies on the conventional Standard Penetration Test equipment while the equipment for cone penetration tests, geophysical exploration tests, and pressure meter tests are readily available and accepted as quality standard field test equipment in geotechnical consulting companies and government agencies. CDOT needs to accelerate its effort in the performance of field test and full-scale load tests.

4.4.2 Status of LRFD Implementation in Neighboring States

The Kansas DOT bases its structural design on LFD procedures and its geotechnical design on ASD procedures. The Wyoming DOT does not believe that there is a major advantage in switching over to the LRFD design procedures, in either bridge superstructure or substructural design and currently practices LFD for both. However, it had attempted the LRFD design procedures on a few structures. Although both DOTs have some suspicion about the benefit of switching over to the LRFD codes as yet, but had begun to pave the way for the eventual implementation of LRFD procedures in both geotechnical and structural designs.

Since CDOT is already practicing the LRFD code in structural designs, and has taken action to look into the feasibility of its implementation in geotechnical designs. In this comparison, CDOT is somewhat ahead in the overall LRFD implementation. CDOT is one of the five pioneer states in recommending the LRFD development, it seems reasonable to carry this pioneer spirit forward and take a strong initiative and leadership role in the implementation of the AASHTO LRFD Bridge Design Specifications. To complete this transition, CDOT needs to provide additional geotechnical human resource, field testing equipment, and full-scale load test capability to accelerate its LRFD implementation effort in the bridge substructure design. While the task is not simple, it is a worthwhile and rewarding effort.

5. STATUS AND NEED FOR IMPLEMENTATION OF GEOTECHNICAL LRFD AT CDOT

5.1 Introduction

The implementation of geotechnical LRFD procedures is critical to the adoption of the AAASHTO LRFD Bridge Design Specifications. The CDOT Bridge Branch has been practicing the AASHTO LRFD procedures for some time. To facilitate the smooth and integrated LRFD bridge superstructure and substructure designs, it is logical to accelerate the implementation of the geotechnical LRFD procedures. It is more difficult to implement the geotechnical LRFD because the need for the Colorado-specific geological materials make it mandatory to determine their resistance factors in advance of the geotechnical LRFD implementation.

The majority of superstructure construction materials like concrete and structure are manufactured under strict quality control. As a result, Colorado concrete is quite similar in engineering properties, if not identical, to concrete from anywhere else in the country or the world. Their properties are quite uniform irrespective of where the concrete and steel are made and the evaluation of their Colorado-specific resistance factors becomes unnecessary. Thus, the implementation of the AASHTO LRFD structure code is much less time-consuming. Meanwhile, geological materials are of natural occurrence and their properties are quite random. Because of the spatial distribution and manufacture process randomness, their properties can be quite different from those of other states and the Colorado-specific resistance factors become necessary for the implementation of the geotechnical LRFD in Colorado. This causes the delay in the implementation of geotechnical LRFD. Many other states face the same dilemma. The response from 28 states to the questionnaire shows that only a limited number of state DOT's have begun their effort toward the implementation of geotechnical LRFD. In fact, Colorado is ahead and can still provide the geotechnical experience to other states bordering Colorado. FHWA recommended the year 2007 as the target time for the implementation of geotechnical LRFD, as reflected in the responses to the question on the timing of the implementation.

As far as the implementation of geotechnical LRFD at the state level, among the states that responded to the survey, Oklahoma and South Carolina have implemented the geotechnical LRFD, and among the states that did not respond to the survey, at least Washington and Florida have implemented the LRFD procedures in retaining wall design and deep foundation design, respectively. The response from the states bordering Colorado, like Kansas and Wyoming, indicates CDOT is ahead in the structural LRFD implementation. CDOT can share its LRFD development experience with the states bordering Colorado.

5.2 CDOT Geotechnical Design and Investigation Practices and LRFD Needs

5.2.1 Colorado-Specific Shortcomings The resistance factors used in the AASHTO 1997a LRFD specifications were calibrated for typical subsurface (geologic) conditions and for methods widely used in predicting the nominal resistance. AASHTO (1991) summarizes special geologic conditions encountered in the U.S that need modification of resistance factors. Therefore, to ensure the successful implementation of geotechnical LRFD in Colorado, it is important to generate resistance factors for CDOT methods not calibrated in or accounted for in the LRFD specifications or generate the "Colorado-specific" resistance factors for the design methods outlined in the AASHTO code. Some current CDOT methods for predicting the nominal resistance need to be calibrated when implementing LRFD procedures. The use of the Denver Magic Formula (DMF based on blow count) to predict the allowable end bearing and side friction for deep foundation is an example. The method, while widely used in CDOT, has never been rigorously calibrated for Colorado geological materials for either the bearing capacity evaluation or settlement computation, critical to maintaining the clearance in highway bridges.

5.2.2 It Is Imperative, Not a Choice To improve the Colorado practice in substructure designs, two possible approaches can be taken:

1) Improve the current CDOT practice by formulating the correlation between the blow count and ultimate bearing capacity and the settlement assessment for deep foundation through using full-scale load tests and/or laboratory testing;
2) Adopt the ASHTO LRFD procedures and specifications after reevaluating the resistance factors for Colorado-specific geological materials;
3) Adopt the AASHTO LRFD code in the design while checking the design against the ASD or LFD design and evaluating the Colorado-specific resistance factors.

While the first approach imposes less impact to the Colorado's geotechnical practices, it does have the following drawbacks:

1. While the DMF works "magically" well (for both driven pile and drilled shaft), it deviates from the FHWA recommended AASHTO LRFD design approach. Besides, the lack of reported failures could be an indication of DMF procedures being excessively conservative and lack of safety uniformity among design cases.
2. N values are usually used directly in the design of shallow foundations in granular soils. Because of its inability in closely reflecting the stress history of claystone and clayshale, critical to the settlement and heave calculation, it is used only in bearing capacity estimation.
3. Generation of the resistance database for the Colorado-specific soils and rocks will be time-consuming.

The adoption of the AASHTO LRFD specifications will mean significantly revising the current CDOT design practice. While this will be more in line with the FHWA recommendations and allow us to gain technical support from other states in time of need, it has the following difficulties:

1. Require revision in CDOT geotechnical investigation practices,
2. Require changes in substructure design practices and, new test equipments, as outlined in Chapters 3 and 4.

3. Due to the unique characteristics of the subsurface materials in the Colorado like clayshale and expansive/collapsible soils, new databases are required for their engineering properties in an effort to evaluate the Colorado-specific resistance factors for its geotechnical LRFD implementation.

CDOT bridge engineers are practicing the LRFD design procedures, while it studies the feasibility and the need for the implementation geotechnical LRFD. The specifications are recommended for implementation at the state level by FHWA. While there may be some contributing factors like shortage of appropriate equipment for subsurface investigation and laboratory testing, insufficient manpower, etc, the main reason for the implementation delay is the lack of material strength database for evaluating the resistance factors for the Colorado-specific geological materials. This proposed study aims at examining the current CDOT practice in geotechnical investigation and design, determine the pros and cons of the practice, and recommend the best strategy for the implementation of the geotechnical LRFD procedures in Colorado. The implementation must not adversely affect the current Project Delivery System. To foster a smooth CDOT design mission, the implementation of the geotechnical LRFD procedures is imperative, not a choice.

5.3 Implementation of Substructure LRFD in Colorado

5.3.1 Introduction Colorado is one of the five states drafted a letter that laid the foundation for the development of the AASHTO LRFD Bridge Design Specifications. Having implemented it in the bridge superstructure design, the CDOT implementation of geotechnical LRFD for substructures will produce a unified approach to bridge and substructure design. The implementation of LRFD substructure design procedures will take a significant investment of time and money. This long-term effort for complete implementation will take about five years. The smooth transition and continuity are critical to the successful implementation that does not adversely affect the project delivery. Once the decision to implement the substructure LRFD procedures is made by the LRFD committee involving the CDOT Central Administration, Bridge Branch, and Geotechnical and Soil-Rock Fall Programs, etc, the data collection for soil and rock resistance and properties and full-scale load tests should be initiated to provide the critically needed database for calculating the resistance factors for Colorado-specific soils and rocks. A data center should be established as the repository of all laboratory, field and full-scale load test data. It will provide the database needed for the continual revision for the values of resistance factors and the continual comparison of these values with the database used in establishing the standard resistance factors in the AASHTO code.

5.3.2 Long-term Effort The complete implementation of LRFD design procedures for substructures is estimated to take five years. It covers the following tasks:

- Establish Substructure LRFD Implementation Committee to oversee the development of the substructure LRFD procedures.
- LRFD procedure training to psychologically prepare all design staff for the implementation of foundation LRFD procedures.
- Establish the LRFD data center as the repository for all data for the evaluation of resistance factors.

- Select the substructure type for implementation,
- Select the design method and equations needed for the design,
- Determine the laboratory and/or field test methods for the evaluation and collection of the design parameters.
- Purchase all necessary equipment for the laboratory, and field testing for soil and rock properties and the full-scale load test, if deemed necessary.
- Statistical analyses of all data to formulate the probability density functions (pdf) for all design parameters.
- Adopt the load factors, the associated database, and the pdf's for all loads from the AASHTO data bank.
- Evaluate the resistance factors corresponding to the risk factor of 0.0001, as selected by the AASHTO LRFD committee.
- Design the foundation with the newly obtained resistance factors and compare it with the ASD (LFD) design, or field full-scale load test results for the calibration of the design.
- Promulgation of the new design procedures throughout CDOT, consulting industry, and government agencies involved in the design of transportation structures.
- The above calibration effort will continue for a period of time till all design staff are satisfied with the design.
- Recommend the resistance factors and LRFD procedures for adoption.
- Expand the procedure to the next foundation type (S) recommended for LRFD implementation and repeat the above development cycle till the complete implementation of substructure LRFD design procedures.

The above listed tasks are to be enhanced by the LRFD Committee and divided into several different phases for execution.

5.3.3 Deep Foundation Recommended for Substructure LRFD Implementation

The national survey shows that the frequency of the foundation types used as bridge abutment and pier support is 21, 51, and 28% for drilled shafts, driven piles, and shallow foundations, respectively. The deep foundation (drilled shafts and driven piles) has 72% percent chance of being adopted as bridge abutment and pier support. Colorado uses a large number of drilled shafts to support bridges, noise barriers and sign/signal posts and another great number of driven piles for as bridge support. Thus, it seems reasonable to recommend the deep foundation as the first candidate for geotechnical LRFD implementation, which can then be subsequently extended to retaining walls, and other foundation types.

6. STRATEGIC PLAN FOR THE IMPLEMENTATION OF GEOTECHNICAL LRFD AT CDOT

Colorado was one of the five states that pioneered the development of LRFD bridge design code in a meeting that was held in Denver. Besides all the technical merits of the LRFD bridge design code, it seems to be reasonable to carry this pioneer spirit to fruition by fully implement the AASHTO Bridge Design Specifications. Since the Bridge Branch has already implemented the LRFD code, it also seems reasonable to accelerate the implementation of geotechnical LRFD. The following outlines the recommended strategy for the implementation of the substructure LRFD design procedures in three different areas: manpower, facility, and technical implementation strategy.

6.1 Manpower

As the survey shows, the CDOT Geotechnical Program and Soils-Rockfall Program are severely understaffed. The average from 28 states excluding Colorado is 32.27 and the average of Kansas and Wyoming is 34.5. This shows that CDOT geotechnical programs are severely understaffed to carry out even the current routine tasks. So it is recommended to drastically increase the geotechnical staff soon.

6.2 Laboratory, Field, and Full-Scale Load Test Facilities

6.2.1 Laboratory The survey shows that CDOT laboratory testing facility is comparable to those of the other respondent states is sufficient for carrying out the routine tasks at current level. However, when the work load increases due to the substructure LRFD implementation, there would be need to purchase additional equipment. It is noticed that triaxial test and direct shear test apparatuses are idle due to the lack of manpower. This situation needs correction.

6.2.2 Field Test and Exploration Equipment The survey shows that CDOT is very much behind the national average in terms of the types of field tests employed in the field subsurface investigation and its practice is far behind the state-of-the art practice in the field investigation. It is quite unfair to expect the quality subsurface investigation to provide the quality data for an effective bridge substructure design, when the Geotechnical Program does not even have minimum field test and exploration equipment. It is strongly urged to purchase the equipment for the following field test and exploration:

- Geophysical exploration,
- Cone penetration test,
- Menard pressure test, and
- Vane shear test.

6.2.3 Full-Scale Load Test Equipment The survey shows that very few states have the capability of performing full-scale load tests. However, this is before their plan for implementing the substructure LRFD is initiated. The state-specific resistance factors required the calibration of the design computation against a reliable calibration standard. In the case of substructure designs, nothing is more reliable than the full-scale load test results. Thus, it is most definite that CDOT needs to perform more full-scale load tests to provide the information for the calibration of the substructure designs. An Octerberg's cell load test, a bottom-up load test, costs, in an average, $100,000. If CDOT plans to perform 100 OC load tests. The total cost amounts to ten million dollars. The need for the top-down, lateral, and torsional load tests will significantly increase the expenditure. It seems to be wise and reasonable investment to develop the in-house capability for different kinds of full-scale load tests. This allows the in-house quality and cost control of load tests. CDOT may even be able to export the professional service to the neighboring states to recover some of the cost of the equipment purchase and development. An alternative will be for all the neighboring states to jointly purchase the equipment for the purpose of sharing the cost of equipment purchase and development.

6.3 Substructure Foundation for LRFD Implementation

Eventually all substructure foundations should be designed using the LRFD code. Initially CDOT can select the most frequently used foundation type for implementation. The survey shows the deep foundation (drilled shafts and driven piles) is used 72 times out of 100. Colorado also uses the deep foundation as the major foundation type for bridge substructures. Thus, it is recommended to select the deep foundation as the first foundation type for the LRFD code implementation and the implementation strategy in Section 6.4 is formulated with this in mind.

6.4 Implementation Plan and Strategy

The task for the implementation of substructure LRFD code is immense. It will take significant investment of time and money. Thus, it needs careful planning and execution. This implementation strategy should not be formulated singularly by the research group at the University of Colorado at Denver and Health Sciences Center (UCDHSC). It should be the collective effort of a Substructure LRFD Committee (or LRFD Committee) with members from all concerned parties to be identified. The following implementation plan and strategy is presented for your "target shooting" aiming at producing an executable plan and strategy.

Plan and Strategy for DOT Implementation of Geotechnical LRFD

- Close examination of the Plan and Strategy recommended for implementation by the UCDHSC research group,
- Establish Substructure LRFD Implementation Committee to formulate the substructure LRFD implementation pan and strategy. The committee should include members from all concerned parties, particularly the CDOT Central Administration, Bridge Branch, Geotechnical Program and Soil-Rock Fall Program, etc,
- LRFD procedure training to psychologically and technically prepare all design staff for the implementation of foundation LRFD procedures,
- Establish an LRFD data center as the repository for all data to be used in the evaluation of resistance factors and calibration of a design,
- Select the substructure type for implementation,
- Select the design method and equations needed for the design,
- Determine the laboratory and/or field test methods for the evaluation and collection of the design parameters,
- Hire additional geotechnical engineering staff,

- Purchase all necessary equipment for the laboratory, and field testing for soil and rock properties and the full-scale load test, if deemed necessary.
- Statistical analyses of all data to formulate the Colorado-specific probability density functions (pdf) for all design parameters.
- Compare the Colorado database of all design parameters with the corresponding AASHTO database to examine the difference,
- Adopt the load factors, associated database, and pdf's for all loads from the AASHTO data bank.
- Evaluate the resistance factors corresponding to the risk factor of 0.0001, as selected by the AASHTO LRFD committee.
- Compare the Colorado-specific resistance factors with the AASHTO resistance factors.
- Design the foundation with the Colorado-specific resistance factors and compare the design with the ASD (LFD) design, and/or full-scale load test results for the design calibration.
- Promulgation of the new design procedures throughout CDOT, consulting industry, and government agencies involved in the design of transportation structures.
- The above calibration effort will continue for a period of time till a satisfactory design is reached.
- Recommend the resistance factors and LRFD procedures for adoption.
- Expand the procedure to the next foundation type(s) recommended for LRFD implementation and repeat the above development cycle till the complete implementation of substructure LRFD design procedures.
- Technology transfer through an educational program and in-house training.

The complete implementation of the geotechnical LRFD procedures will require significant investment of time and money. The author of this report anxiously anticipates significant reference of this research report, particularly the plan and strategy, in the CDOT implementation effort.

7. DEEP FOUNDATION DESIGN

7.1 Introduction

In Colorado, most bridge abutments and piers are supported on drilled piers, rock socket piers, or driven piles. The national survey reflects the same trend with 72% goes for the deep foundations, drilled shafts (21%) and driven piles (51%). Thus, selecting piers and piles for geotechnical LRFD implementation is a reasonable choice. The implementation should start from single and then to group piers or piles.

CDOT has adopted DMF in the evaluation of vertical pier capacity. The lack of failure cases indicates the DMF approach could be conservative. CDOT also has used LPILE program in the in the design of pier under lateral loads with good success. However, with the intent of implementing geotechnical LRFD, it is necessary to assess the true (or ultimate) pier capacity through full-scale pier (or pile) load tests. It is very encouraging to learn in a meeting that CDOT is committed to performing in-situ pier and pile load tests for construction projects and has already done some as presented in the research report (Abu-Hejleh, etc, 2003). These field tests form an irreplaceable precious database for the calibration of nominal capacities based on the selected design methodology and formulation. The statistical sample size is increasing, it is still much too small to recommend for implementation. The sample size issue was discussed in a recent paper (McKay, 2004). However, the load test effort can not cease, but continue to allow the enhancement of database.

In an attempt to present an alternative methodology for the assessment of deep foundation capacity, the NIKE3D-SSI research group in the Center for Geotechnical Engineering Science at the University of Colorado at Denver and Health Science Center has devoted a good portion of its research effort on the finite element analysis of deep foundation. When calibrated and found effective, this method can be used to predict the bearing capacity, settlement, lateral deformation, torsional rotation of deep foundation. Results are shown in Appendix B.

So far, the research group has used the NIKE3D program developed at the Lawrence Livermore National Laboratory and is trying LSDYNA program from the LSTC, Inc. This finite element programs can become formidable design tools, when found to be effective, because numerical tests do not cost much to perform. It can also be used to generate the database for the deep foundation bearing capacity and performance for use in evaluating the resistance factors and performance limits. The following are the areas covered by this research group:

- Drilled Shafts in Hard Clays (completed). The study covers the performance of singular load of vertical, horizontal, moment, or torsional load. When the choice of material parameters and the delineation of soil strata are properly conducted, the comparison with the existing full-scale load tests shows some encouraging results as presented in later section of this report. This indicates that the numerical analysis may be an effective tool for assessment of deep foundation capacity and performance and can assist in generating and check the database for LRFD implementation. Excel programs are being finalized for use by design engineers to assess the bearing capacity and performance of short and long piers under lateral and torsional wind loads.

- Drilled Shafts in Sands (completed). The study assumed the sand to be elastic material. This is only qualitative and preliminary purpose for pier under singular load, of vertical, lateral, moment and torsional. Another study is being pursued with more rigorous effort in simulating the behavior of sand and thus, true behavior of deep foundation in sand.
- Drilled Shafts in Hard Clays under Combined Loads of vertical, horizontal, moment and torsional loads. (on going, completion expected May, 2005). This study aims to investigate the deep foundation performance under the combined load of torsional, axial, lateral and moment loads. The cross-load type effects are expected and this study intends to investigate the effect.
- Vertical Load Capacity of Drilled Shafts (ongoing, completion expected December, 2005). Osterberg cell (OC) load tests are being used throughout the country, in which the load is applied from the bottom of a deep foundation. The test provides the load-displacement curve for both end bearing and side friction. Problems needing investigation are two: the simultaneous failure in both end bearing and side friction is not observed and the effect of overburden on the true top-down side friction is not well understood. This study aims to answer these two questions and propose a viable mechanism with which the true top-down bearing capacity (end bearing plus side friction capacities) can be evaluated given the OC load test results.

The NIKE/SSI research group urgently needs the funding for research and the results of full-scale load tests for calibration of the numerical analysis results. When proven effective, the numerical analysis can be used to generate the urgently needed database for the Colorado-specific resistance factor evaluation. To evaluate the load capacity and performance limit for deep foundation, different methods are available depending on the loading conditions. Deep foundations are designed to resist any combination of vertical, lateral, moment and torsional loads. The design can follow the following steps:

- Define the loading condition.
- Survey available design tools including the numerical analysis, select a minimum of one method, and use it to determine the nominal capacity and the associated performance limit.
- Compare the analysis result with the full-scale test result to establish the confidence in the chosen analysis method.
- Establish a database for full-scale load tests for use in the calibration of the design method and recommended LRFD resistance factors.
- In case of driven piles under vertical load, design methods include Terzaghi and Meyerhoff bearing capacity equations and the capacity using PDA and CAPWAP. The design calculations can be compared to static load test results to gain confidence in the selected design method. It is well known that Meyerhoff’s method gives much higher capacity than Terzaghi’s method.
- During the transition, the ASD design can serve as a calibrator for the LRFD design.

This section reports the study on drilled shafts in hard clay under lateral load, moment, or torsional load. The parallel study on the piers under combined loading is progressing and is expected to complete in May, 2005. This study is much more involved and complicated. Discussion in this section on singular load includes:

- Material parameter selection,
- Brom's method and its comparison with numerical analysis method,
- LPILE method, and its comparison with numerical analysis method, and
- Finite element method of socketed piers in clay shale will be presented in Appendix B on Socketed Piers.

7.2 Material Parameter Selection

7.2.1 Introduction

In a numerical analysis, material properties are needed for each material. A laterally loaded pile problem involves reinforced concrete for piles and surrounding soils. Concrete is modeled as an elastic material. Concrete properties are more consistent and predictable as compared to soils. Consequently, this chapter section is devoted to soil parameters for elastic soils with two parameters: Young's modulus (E_s) and Poisson's ratio (ν_s). Poisson's ratio of 0.35 and 0.45 are used for low and high moisture clay and clayshale, respectively.

Young's modulus of soil is difficult to evaluate. Section 7.2.2 synthesizes the literature survey on the elasticity of soils. Methods of determining E_s and its correlation with undrained shear strength of soils (c_u) are presented. In laterally loaded pile problems, the concept of subgrade reaction is usually used. Section 3.3 is devoted to the determination of coefficient of horizontal subgrade reaction (k_h) and its relation with undrained shear strength of soils. The relationship between E_s and $k_{h,}$ the ratios of E_s to c_u, the elasticity of clays, and finally the Ramberg-Osgood (RO) parameters for soils are all synthesized in Section 7.2.

7.2.2 Young's Modulus of Soils (E_s)

This section discusses the methods for evaluating the Young's modulus of soils, E_s. Laboratory and in-situ tests are commonly used to determine E_s. Sometimes, empirical correlations are also used to estimate Es using undrained shear strength of clays or shear modulus. The back calculations of E_s from full-scale lateral load tests are considered to provide the best fit value, E_s, for soils, which is calculated directly from the lateral load-deflection curves from the tests.

7.2.3 Methods for Determining E_s

Literatures give the following methods for determining E_s:

1) Back calculation from full-scale pile tests,
2) In-situ tests,
3) Laboratory tests,
4) Empirical correlations with undrained shear strength, and
5) Empirical correlations with shear modulus

7.2.3.1 Full-Scale Pile Tests
This method is accepted to be the most reliable method to determine Young's modulus of soils. Effects of piles installation and pile-soil interaction are taken into account automatically. Poulos and Davis (1980) recommended using ground-line deflection at working load to back analyze the secant modulus and use the linear part of load-deflection curve to back figure the tangent modulus. They also commented that a tangent modulus should give a more logical load-deflection relationship.

7.2.3.2 Laboratory Testing
Unconfined compression (UC) and unconsolidated undrained (UU) triaxial tests are frequently used in determining the modulus of elasticity and strength of clayey soils under undrained conditions, like in the design of laterally loaded piles. In the situation when E_s and undrained shear strength data are not available, E_s can be estimated using EQ (7.1) and results from the Oedometer test as recommended in the Canadian Foundation Engineering Manual (1985):

$$E_s = C\sigma_c^{'} \qquad (7\text{-}1)$$

where σ'_c is effective preconsolidation pressure from Oedometer test or Figure 7-1 and C is a constant:

- C = 80 for stiff clays,
- C = 60 for medium clays, and
- C = 40 for soft clays.

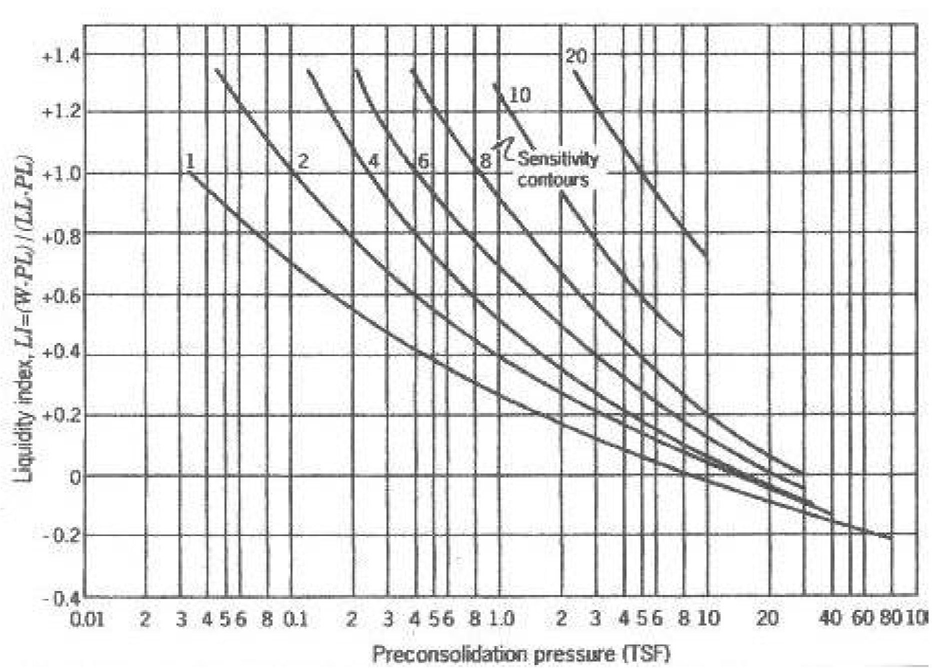


Figure 7-1 Effective Preconsolidation Pressure (σ'_c) vs. Liquidity Index (LI) (Prakash and Sharma, 1990)

7.2.3.3 Correlation with Shear Modulus

Shear modulus can be related to elasticity of soils as shown in EQ (7-2).

$$E_s = 2(1+\nu)G \qquad (7\text{-}2)$$

where E_s is modulus of elasticity of soil
G is shear modulus evaluated in laboratory or in-situ tests.
ν is Poisson's ratio

7.2.3.4 In-situ Tests

Cone penetration test (CPT)

Many in-situ tests can be used for determining elasticity of soils. The Cone Penetration Test (CPT), pressuremeters, and plate load tests are commonly used. E_s in tsf can be empirically related with the cone tip bearing resistance by

$$E_s = \alpha_c q_c \qquad (7\text{-}3)$$

where α_c is correlation factor depending on soil type and the cone bearing resistance and can be obtained from Table 7-1
q_c is the cone tip bearing resistance, tsf

Table 7-1 Correlation Factor α_c [a]

Soil Type	Resistance q_c, tsf	α_c
Lean clays (CL)	< 7 7 to 20 > 20	3 to 8 2 to 5 1 to 2.5
Plastic clays (CH)	< 20	2 to 6

[a] Adapted from Corps of Engineers (1990).

Pressuremeter Test (PMT)

The pressuremeter modulus may be evaluated from Corps of Engineers 1990 Manual

$$E_p = \frac{(1+\nu)\Delta P(R_{po} + \Delta R_{pm})}{\Delta R_p} \qquad (7\text{-}4)$$

where ν is Poisson's ratio
ΔP is change in pressure measured by the pressuremeter, tsf
R_{po} is radius of probe, inches
ΔR_{po} is change in radius from R_{po} at midpoint of straight portion of the pressure meter curves, inches
ΔR_p is change in radius between selected straight portions of the pressuremeter curve, inches

Plate load test (PLT)

The modulus of elasticity can be estimated from the results of plate load tests by

$$E_s = \frac{(1-\nu^2)}{(\frac{\Delta\rho}{\Delta q})} B I_w \qquad (7\text{-}5)$$

where E_s is elasticity of soils, psi
ν is Poisson's ratio
$\Delta\rho/\Delta q$ is slope of settlement versus plate pressure, inches/psi
B is plate diameter, inches
I_w is influence factor, for circular plates, $I_w = \pi/4$

7.2.3.5 Empirical Correlations with Undrained Shear Strength of Soil

Many previous studies showed that the elasticity of soil is strongly related to the undrained shear strength of clayey soils. Some empirical correlations between E_s and C_u are summarized as follows:

- In 1951, Skempton suggested that the secant modulus of the soils (E_{50}) was related to the unconfined compressive strength of soils (q_u). He proposed the ratio of E_{50} to q_u were in the range of 25 to 100. Consequently, E_{50} is in between 50 to 200 times c_u.
- Poulos (1971a) used elastic model to study behavior of piles subjected to lateral loads. In his study, he proposed to use 15 to 95 times c_u as the secant modulus of soils. The lower value is for soft clays and the higher value is for stiff clays.
- Banerjee and Davies (1978) reported the value of E_{50} was in between 100 to 180 times c_u for soft clay.
- Sullivan (1980) reported values of E_{50} for overconsolidated clays were 100 to 250 times c_u from UU triaxial tests.
- Davies and Budhu (1986) suggested the ratio of secant modulus to undrained shear strength for stiff clays in the range of 500 to 1000. In their review of values of E_{50}/c_u, they found that Simons (1976) reported the ratio ranges from 40 to 3000 for c_u from laboratory tests and Bjerrum (1972) gave the values of 500 to 1500 for c_u from vane shear tests.
- Bowles (1988) suggested using the elasticity of clays as functions of undrained shear strength of clays in finite element analyses. He gave the expressions to calculate E_s for different types of clays as shown below:
  - Normally consolidated sensitive clay:
    $$E_s = 200C_u \text{ to } 500C_u$$
  - Normally consolidated insensitive and lightly overconsolidated clay:
    $$E_s = 750C_u \text{ to } 1200C_u$$
  - Heavily overconsolidated clay:
    $$E_s = 1500C_u \text{ to } 2000C_u$$
- Corps of Engineers (1990), in Engineering Design series: Settlement analysis, gave the relationships between undrained modulus (E_u) and undrained shear strength of clays. EQ (7-6) shows this relation:

$$E_u = K_c c_u \tag{7-6}$$

where K_c is correlation factor and can be found in Figure 7-2.

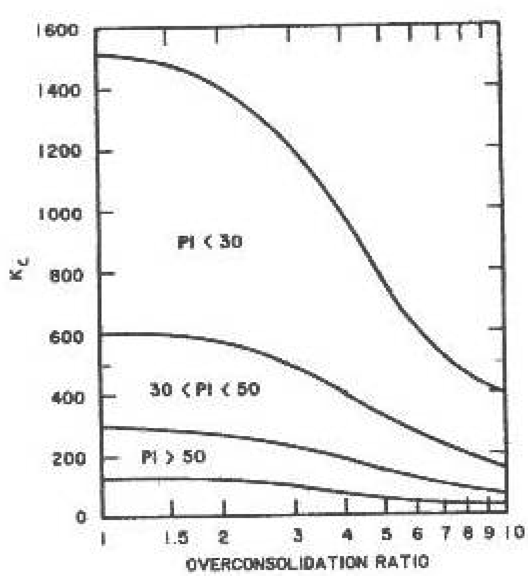


Figure 7-2 Correlation Factor K_c used in $E_u = K_cC_u$ (Corps of Engineers 1990)

Few studies had been carried out to relate initial modulus of elasticity with undrained shear strength. Matlock et al (1956) and Reese et al (1968) studied the relationships between the initial or tangent modulus of clays (E_i) and undrained shear strength from unconfined compression tests. Matlock et al found that E_i were in range of 40 to 200 times the undrained shear strength of clays from Lake Austin. Reese et al studied stiff clays at Manor, Texas and found that E_i were in between 100 and 200 times c_u. Plot of ratio of E_i to c_u along the depth from these two studies is shown in Figure 7-3.
Poulos and Davis (1980) reported the ratio of E_i to c_u ranged from 250 to 400 using lower and higher values for soft and stiff clays respectively. These empirical relations were supposed to use in laterally loaded piles problems.

In 1989, Stokoe found that values of E_i were 2000 times undrained shear strength from laboratory test results of soils at small strain levels.

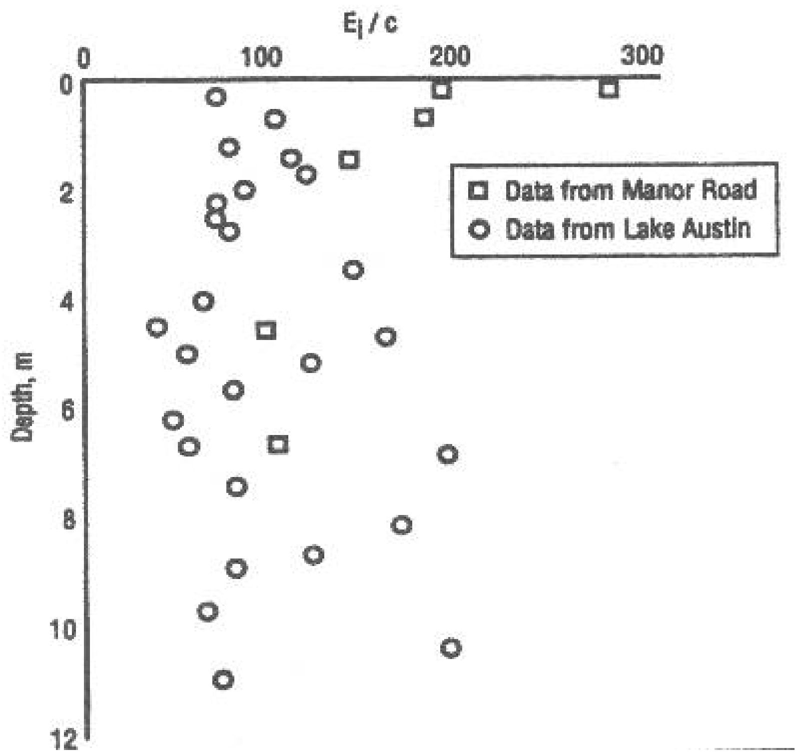


Figure 7-3 Plot of Ratio of E_i to c_u from Unconfined Compression Tests of Clays (Reese et al 2000)

7.2.4 Empirical Correlations of E_s with Shear Modulus (G)

A few studies had been carried out to determine correlations of shear modulus and undrained shear strength of clays. Seed and Idriss (1970) studied soil moduli and damping factors for dynamic analyses. In their study, they collected ratio of in-situ shear modulus to undrained shear strength of saturated clays from many sources and plotted into one graph (Figure 7-4). The ratios of G/c_u depend on shear strain levels.

In 1988, Weiler studied shear modulus of clays at small strain levels. He provided the ratios of maximum shear modulus to undrained shear strength of clays obtained from CU triaxial tests. These ratios depend on plasticity index (PI) and overconsolidation ratio (OCR) of clays and are shown in Table 7-3. Correlations of shear modulus and undrained shear strength from various studies can be summarized in Table 7-4.

Table 7-2 Modulus of Elasticity from Undrained Shear Strength

No	Study	Correlation	Remarks
1	Skempton (1951)	E_{50} = 50 to 200c_u	c_u from unconfined compression tests
2	Poulos (1971)	E_{50} = 15 to 95c_u E_{50} = 40c_u	Lower values for soft clays and higher values for stiff clays For average value

3	Bjerrum (1972)	E_{50} = 500 to 1500c_u	c_u from vane shear tests
4	Simons (1976)	E_{50} = 40 to 3000c_u	c_u from laboratory tests
5	Banerjee and Davies (1978)	E_{50} = 100 to 180c_u	For soft clays and E_{50} assumed to vary linearly with depth
6	Sullivan (1980)	E_{50} = 100 to 250c_u	For OC North Sea clays and c_u from UU triaxial tests
7	Davies and Budhu (1986)	E_{50} = 500 to 1000c_u	For stiff clays c_u from standard triaxial undrained test
8	Bowles (1988)	E_s = 1500 to 2000c_u E_s = 750 to 1200c_u E_s = 200 to 500c_u	For heavily overconsolidated clays For normally consolidated insensitive and lightly OC clays For normally consolidated sensitive clays
9	Corps of Engineer (1990)	$E_u = K_cC_u$	E_u is undrained modulus and K_c is correlation factor from Figure 3-2
10	Matlock et al (1956)	E_i = 40 to 200c_u	E_i is initial or tangent modulus and c_u from unconfined compression tests
11	Reese et al (1968)	E_i = 100 to 200 c_u	c_u from unconfined compression tests
12	Poulos and Davis (1980)	E_i = 250 to 400c_u	Lower values for soft clays and higher values for stiff clays
13	Stokoe (1989)	E_i = 2000c_u	From soil samples subjected to very small strains

Table 7-3 Values of $K_m = G_{max}/c_u$ (Weiler, 1988)

Plasticity Index	Overconsolidation Ratio, OCR		
	1	2	5
15-20	1100	900	600
20-25	700	600	500
35-45	450	380	300

Table 7-4 Shear Modulus from Undrained Shear Strength

No	Study	Correlation	Remark
1	Seed and Idriss (1970)	$G = KC_u$	G is in-situ shear modulus K is factors depending on strain levels and can be determined from Figure 7-4.
2	Weiler (1988)	$G_{max} = K_mC_u$	G_{max} is maximum shear modulus K_m is factors from Table 3-3 c_u is undrained shear strength from CU tests

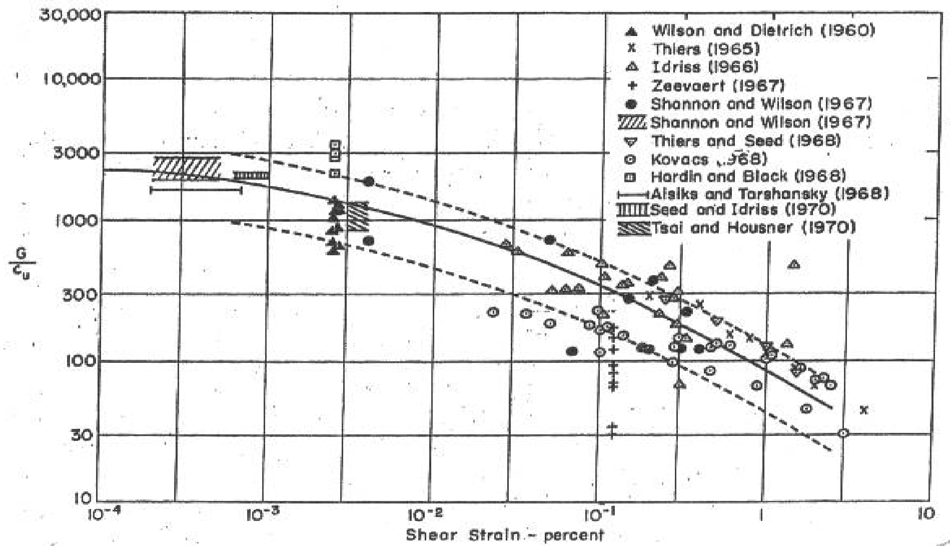


Figure 7-4 Ratio of Shear Modulus (G) to c_u for Saturated Clays (Seed and Idriss 1970)

7.2.5 Coefficient of Horizontal Subgrade Reaction (k_h)

The concept of subgrade reaction has been using in predicting lateral deformations of piles under lateral loading. This concept is based on Winkler's study in 1867. In his study, he modeled soil behavior by independent series of linear springs along the depth of the soils. The Winkler model is shown in Figure 7-5. Coefficients of horizontal subgrade reaction (k_h) represent stiffness of springs. When pile subjected to lateral load, pile will deform laterally from the original position y. The resistance from soils is represented by pressure q can be related to lateral deformation y by EQ (7-7).

$$q = k_h y \qquad (7\text{-}7)$$

where q is soil reaction in force per square length
k_h is coefficient of horizontal subgrade reaction in force per cubic length
y is lateral deflection of pile in length.

Evaluation of k_h is required for solving piles under lateral loadings. There are many ways to evaluate k_h. Among these are:

1) Full-scale lateral-loading tests on piles
2) Plate load tests
3) Empirical correlations with soil properties

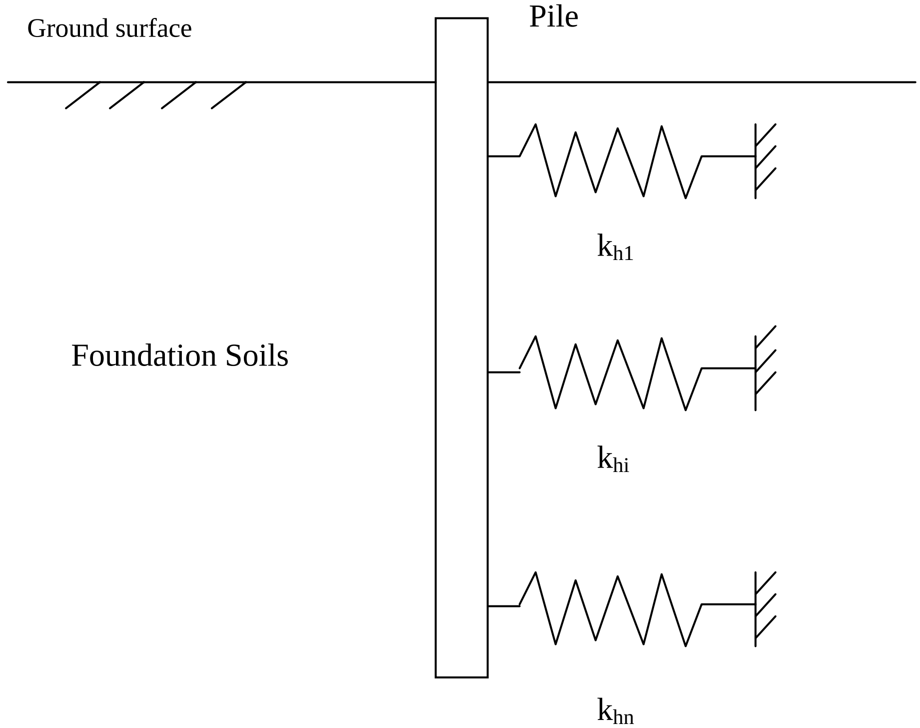


Figure 7-5 Soils Modeled by Winkler's Assumptions

7.2.6 Methods for Determining k_h

7.2.6.1Full-Scale Lateral-Loading Tests on Piles

The lateral load test may be the best method for determining the represented in-situ horizontal subgrade reaction modulus of soils, k_h, where a pile is instrumented to measure soil pressures and associated deflections. The test results are used to calculate k_h. However, the method is both expensive and time-consuming.

7.2.6.2 Plate Load Tests

Plate load tests can be undertaken on both horizontal and vertical plates. These tests may be tested using either circular or square plates. Many studies had been carried out to evaluate k_h from plate load tests. Terzaghi (1955) is the pioneer to evaluate coefficients of horizontal subgrade reaction. He suggested determining k_h from the results of horizontal plate load tests using 1-feet wide square plate. He proposed the expression to calculate k_h from the coefficient of vertical subgrade reaction from 1 ft x 1ft plate (k^*_{s1}). The expression is shown in EQ (7-8).

$$k_h = \frac{k^*_{s1}}{1.5D} \tag{7-8}$$

where k_h is coefficient of horizontal subgrade reaction in ton per cubic feet
k^*_{s1} is coefficient of vertical subgrade reaction and given in Table 7-5.
D is pile diameter in feet

In 1961, Vesic studied behavior of infinite beam resting on isotropic elastic soil. He proposed an expression to determine coefficient of vertical subgrade reaction k for infinite piles from the plate load tests using square plate with a width D. The k_s can be applied to relatively long piles with a width D subjected to lateral loads.
Broms (1964a) suggested using Vesic's expression to calculate k_h for relatively long piles by assuming k_s and k_h values are the same. Broms (1965) proposed the expression to calculated k_h for relatively long piles from plate load tests as shown in EQ (7-9).

$$k_h = \frac{0.4 k_0 B}{D} \tag{7-9}$$

where k_0 is the coefficient of vertical subgrade reaction for a square or circular plate with the side or diameter of B
D is pile diameter

Table 7-5 Values of k^*_{s1} in ton/cu. ft for Square Plates, 1 ft x 1 ft and for Long Strips, 1 ft Wide [a], Resting on Pre-compressed Clay (Terzaghi 1955)

Consistency of clay	Stiff	Very Stiff	Hard
Values of q_u, ton/sq. ft	1-2	2-4	> 4
Range of k^*_{s1}, square plates	50-100	100-200	> 200
Proposed values, square plates	75	150	300 [b]

[a] For rectangular plates with width 1 ft and length L ft: $k_{s1} = k^*_{s1}$ (L + 0.5)/1.5L
[b] Higher values should be used only if they were estimated on the basis of adequate test results.

7.2.7 Empirical Correlations
There were a few studies on relationships between k_h and soil properties. As mentioned in previous section, Terzaghi (1955) proposed the procedure for calculating k_h from plate load test. In 1970, Davisson used nondimensional solutions to determine lateral load capacity of piles. He suggested the expression to relate k_h with undrained shear strength of clay (c_u) and pile diameter (D) as shown in EQ (7-10).

$$k_h = \frac{67 c_u}{D} \tag{7-10}$$

It is easy to show that Terzaghi and Davisson's expressions are the same. From Table 3-5, k^*_{s1} can be converted to q_u and then c_u as shown in EQ (7-11).

$$k^*_{s1} = 50 q_u = 100 c_u \tag{7-11}$$

Substitute EQ (7-11) into EQ (7-8), we get EQ (7-12).

$$k_h = \frac{67c_u}{D} \qquad (7\text{-}12)$$

The difference between Terzaghi and Davisson is Terzaghi suggested to use representative value of k_h for a range of q_u or c_u while Davisson used typical value of k_h for given value of c_u. Table 3-6 summarizes k_h formulae from both Terzaghi and Davisson.

Table 7-6 Coefficient of Horizontal Subgrade Reaction (k_h) from c_u

No	Study	Correlation	Remark
1	Terzaghi (1955)	$k_h = \frac{k_{s1}^*}{1.5D}$	k^*_{s1} from Table 7-5 and D is pile diameter
2	Davisson (1970)	$k_h = \frac{67c_u}{D}$	c_u is undrained shear strength D is pile diameter

7.2.8 Relationship Between E_s and k_h

Many attempts were devoted to study the relationships between elasticity and coefficient of subgrade reaction. Based on Skempton's works in 1951, Broms (1964a) mentioned that the coefficient of subgrade reaction for cohesive soils is approximately proportional to the unconfined compressive strength of the soil. Work by Terzaghi (1955) also showed the relation of coefficient of subgrade reaction with unconfined compressive strength of clays. Many studies show that elasticity of soils can be related to undrained shear strength of the soils (Section 7.2.2). It is possible to relate E_s with k_h.

Vesic (1961) proposed the coefficient of vertical subgrade reaction in terms of elasticity of soil, Poisson's ratio of soil, bending stiffness of pile, and pile width as shown in EQ (7-13).

$$k_s = \frac{0.65}{D}\left(\frac{E_s D^4}{E_p I_p}\right)^{\frac{1}{12}} \frac{E_s}{1-\nu^2} \qquad (7\text{-}13)$$

where k_s is coefficient of vertical subgrade reaction
E_s is modulus of elasticity of soil
E_p is modulus of elasticity of pile
I_p is moment of inertia of pile section
ν is Poisson's ratio of soil
D is pile width or diameter

Based on Timoshenko and Goodier (1951)'s theory of elasticity, Broms (1964a) suggested to calculate coefficient of subgrade reaction from circular or square plate load tests and relate k_h with secant modulus of elasticity of clays as shown in EQ (7-14).

$$k_0 = \frac{1.25E_{50}}{B(1-\nu^2)} \quad (7\text{-}14)$$

where k_0 is coefficient of subgrade reaction from circular plate load test
E_{50} is the secant modulus of elasticity of the soils
B is plate diameter or width
ν is Poisson's ratio of the soils

Substitute k_0 from EQ (7-14) into EQ (7-9), we get

$$k_h = \frac{0.5E_{50}}{D(1-\nu^2)} \quad (7\text{-}15)$$

Pike and Beikae (1984) reviewed the solutions of piles subjected to lateral loads from the previous studies. Based on Beikae's work in 1982, they proposed the relationships between k_h and E_{50} for clays as shown in EQ (7-16a) to (7-16c).

$$k_h = \frac{2.3E_{50}}{D} \quad (7\text{-}16a)$$

$$k_h = \frac{2.0E_{50}}{D} \quad (7\text{-}16b)$$

$$k_h = \frac{1.8E_{50}}{D} \quad (7\text{-}16c)$$

EQ (7-16a), (7-16b), and (7-16c) are for Poisson's ratio of 0.0, 0.33, and 0.5, respectively.

Table 7-7 Relationships between Elasticity and Coefficient of Subgrade Reaction for Clays

No	Study	Relationship	Remark
1	Vesic (1961)	$k_s = \frac{0.65}{D}\left(\frac{E_s D^4}{E_p I_p}\right)^{\frac{1}{12}} \frac{E_s}{1-\nu^2}$	For relatively long piles
2	Broms (1964a)	$k_h = \frac{0.5E_{50}}{D(1-\nu^2)}$	For relatively long piles
3	Pyke and Beikae (1984)	$k_h = \frac{2E_{50}}{D}$	For $\mu = 0.33$

7.2.9 E_s for Numerical Models

As shown in the preceding sections, modulus of elasticity of soils can be determined by many methods. Among those, the most reliable method is to perform pile load test in the field and back analyze the soil modulus. If the full-scale pile test is not performed, the evaluation of E_s from in-situ or laboratory testing is recommended. If the undrained shear strength is available, the empirical relationships between E_s and c_u can be used with caution. E_s is a function of undrained shear strength of clays. Coefficients of horizontal subgrade reaction can be related to both undrained shear strength and elasticity of soils. Consequently, E_s, k_h, and c_u are interrelated. From previous sections, a flow chart, Figure 7-6, is drawn for determination of Es. Figure 7-6 clearly shows that that method IV and V can be regrouped to calculate k_h from c_u. It is widely accepted that undrained shear strength of clays is important parameter for analyses of piles under lateral loads in clay. The subsequent sections are devoted to the evaluation of undrained shear strength, C_u/σ_{vc}' and E_s/c_u ratios, and E_s for analyzing pile performance in clays.

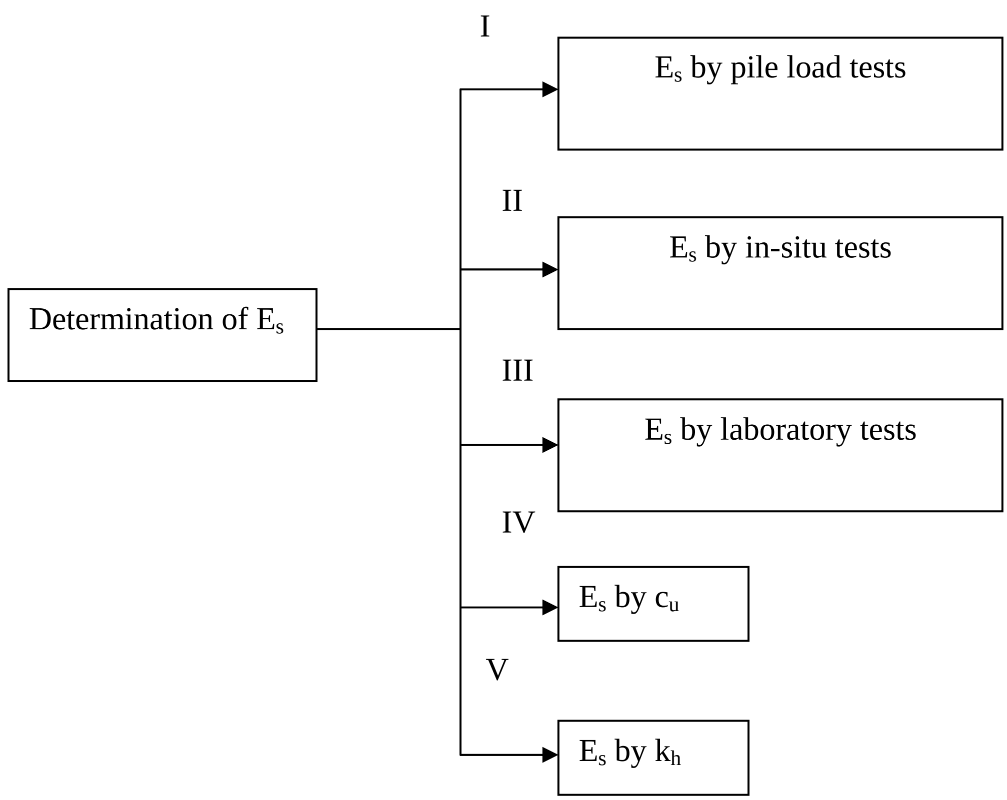


Figure 7-6 Flow Chart for Determination of E_s

7.2.10 Estimation of c_u

7.2.10.1 In-situ Tests

Knowing N value from Standard Penetration Tests, undrained shear strength of clays can be estimated from Table 7-8. In case that CPT (Cone Penetration Tests) data is available, c_u can be estimated from Table 7-9.

Table 7-8 Approximated Undrained Shear Strength of Clays from SPT

SPT Penetration N Values [a]	Estimated Consistency	Estimated Range of c_u, ksf
< 2	Very Soft (extruded between fingers when squeezed)	< 0.25
2 - 4	Soft (molded by light finger pressure)	0.25 – 0.50
4 - 8	Medium (molded by strong finger pressure)	0.50 – 1.00
8 - 15	Stiff (readily indented only with great effort)	1.00 – 2.00
15 - 30	Very stiff (readily indented by thumbnail)	2.00 – 4.00
> 30	Hard (indented with difficulty by thumbnail)	> 4.00

[a] The Canadian Foundation Engineering Manual does not recommend the relationship with N.
Source: Prakash and Sharma (1990) and Terzaghi and Peck (1967), Design Manual NAVFAC DM7.1 (1982), and Canadian Foundation Engineering Manual (1985).

Table 7-9 Undrained Shear Strength of Clays from CPT [a] (McCarthy 2002)

General Description	q_c/p_a from CPT	Consistency Index, CI = 1.0 – LI [b]	Approximated c_u, tsf
Very soft	< 5	< 0.5	< 0.25
Soft	5 to 15	0.5 to 0.75	0.25 – 0.5
Medium			0.5 – 1.00
Stiff	15 – 30	0.75 – 1.0	1.00 – 2.00
Very stiff	30 – 60	1.0 – 1.5	2.00 – 4.00
Hard	> 60	> 1.5	> 4.00

[a] Data compiled from various sources.
[b] $LI = \frac{(w - PL)}{(LL - PL)}$ is liquid index.

The undrained shear strength can also be evaluated from the result of vane shear tests as follow (Coduto, 1999):

$$c_u = \frac{6\lambda T_f}{7\pi d^3} \qquad (7\text{-}17)$$

where c_u is undrained shear strength in psf
T_f is torque at failure in lb-ft
d is vane diameter in ft
λ is correction factor for time effect from Figure 7-7

Pressuremeter can be used for determining undrained shear strength of clays as shown in EQ (7-18) provided by Roberson (1986).

$$c_u = \frac{P_L - P_0}{5.5} \tag{7-18}$$

where P_L is the maximum pressure in kg/cm^2
V_L is the maximum volume corresponding to P_L (Figure 7-8)
P_0 is the initial pressure of the pressuremeter in kg/cm^2 and the corresponding volume is V_0 (cm^3)

Alternatively with EQ (7-4), the pressuremeter modulus, E_p, can be estimated from the slope of the linear portion of the pressure-volume curve in (Figure 7-8) by

$$E_p = 2.66(V_0 + V_m)\left(\frac{\Delta P}{\Delta V}\right) \tag{7-19}$$

where V_0 is initial volume of the measuring cell
V_m is volume change read on the volumeter at a pressure corresponding to the mean pressure in the pseudoelastic range (estimated from mean of V_0 and V_f)
$\Delta P/\Delta V$ is slope of the pressure-volume curve as the Section AB, where Point A refers to the initial stage and Point B the upper limit of linear portion.

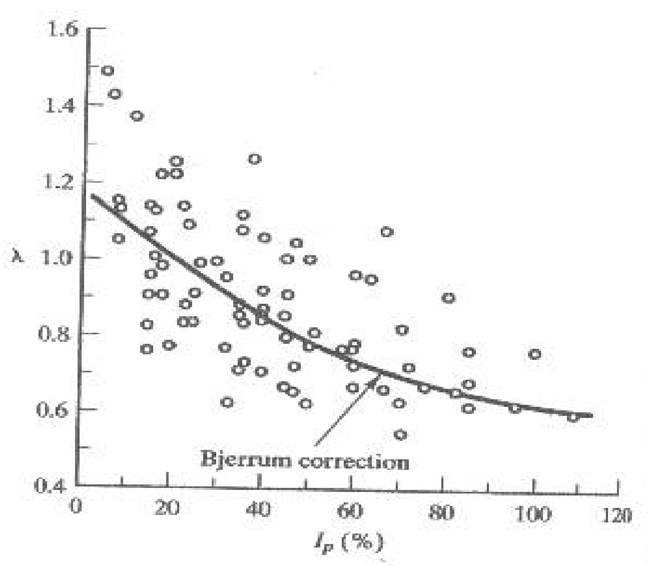


Figure 7-7 Correction Factor, λ for Vane Shear Tests (Coduto, 1999)

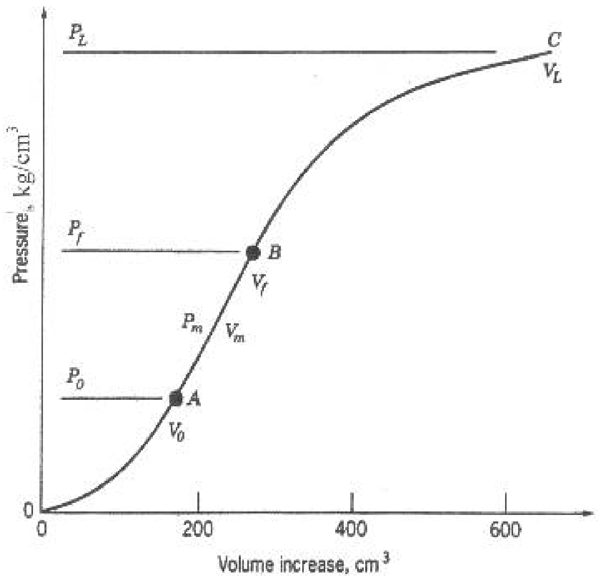


Figure 7-8 Idealized Pressure-Volume Curve from Menard-type Prebored Pressuremeter Test (Prakash and Sharma, 1990)

7.2.10.2 Laboratory Tests

Many laboratory tests can yield the undrained shear strength of clays and consolidated undrained (CU) triaxial tests are preferred (Bardet 1997) including isotropically-consolidated undrained triaxial compression tests (CIUC). Chen and Kulhaway (1993) indicated that unconsolidated undrained (UU) triaxial tests gave a better estimate of undrained shear strength than unconfined compression tests (UC). However, because of their simplicity, UC tests are more frequently used commercially than CU and UU tests. This section is devoted to the empirical, laboratory-based correlations of c_u with others soil properties. c_u from undrained shear strength can be estimated from plasticity index (PI) and the effective preconsolidation pressure (σ'_c) (Design Manual NAVFAC DM 7.1 1982).

$$c_u = \sigma'_c(0.11 + 0.0037PI) \qquad (7\text{-}20)$$

where σ'_c is determined from Oedometer tests or Figure 7-1. For normally consolidated natural clays, the effective vertical overburden pressure (σ'_v) can be replaced with σ'_c in EQ (7-20) to estimate c_u (Skempton 1948; Bjerrum and Simons 1960).
Based on the database for CIUC and UU tests, Chen and Kulhawy (1993) suggested the correlation between σ'_c, atmospheric pressure (p_a) and liquidity index, LI, (See also Figure 7-1 or Table 7-9):

$$\sigma'_c = p_a 10^{(1.11-1.62LI)} \qquad (7\text{-}21)$$

The undrained shear strength of clays can also be estimated from the effective overburden stress (σ'_{vc} or σ'_{vo}) and overconsolidation ratio (OCR). Ladd et al. (1977) and

Jamiolkowski et al. (1985) suggested the following relation between c_u/σ'_{vc} and OCR as follows

$$\frac{c_u}{\sigma'_{vc}} = S(OCR)^m \qquad (7\text{-}22)$$

where $S = \left(\frac{c_u}{\sigma'_{vc}}\right)_{nc}$ is the ratio for normally consolidated clay and m is the strength increase exponent.

Ladd et al. (1977) found that the average value for five different clays from the direct simple shear tests is 0.8. Jamiolkowski et al. (1985) reported that $S = 0.23 \pm 0.04$ for clays having plasticity index < 60%. Davies and Budhu (1986) suggested using S = 0.3 for many clays. EQ (7-22) can be plotted on both semi-log scale as shown in Figure 7-9 and log-log scale as shown in Figure 7-10.

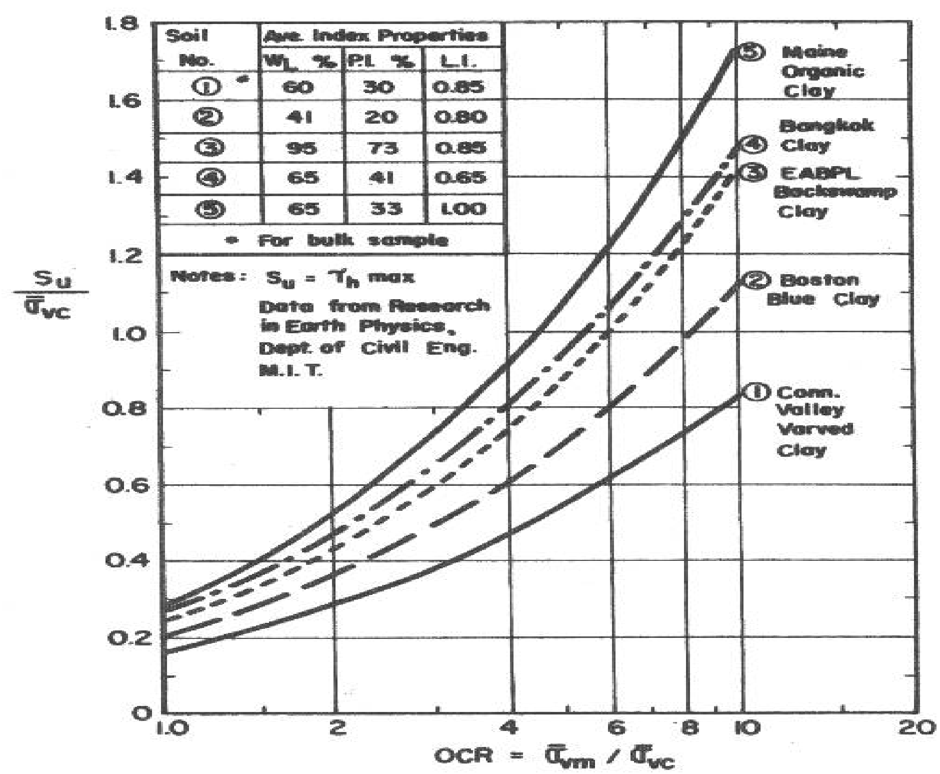


Figure 7-9 Variation of Undrained Shear Strength Ratio with OCR for Five Soils (Foott and Ladd 1973).

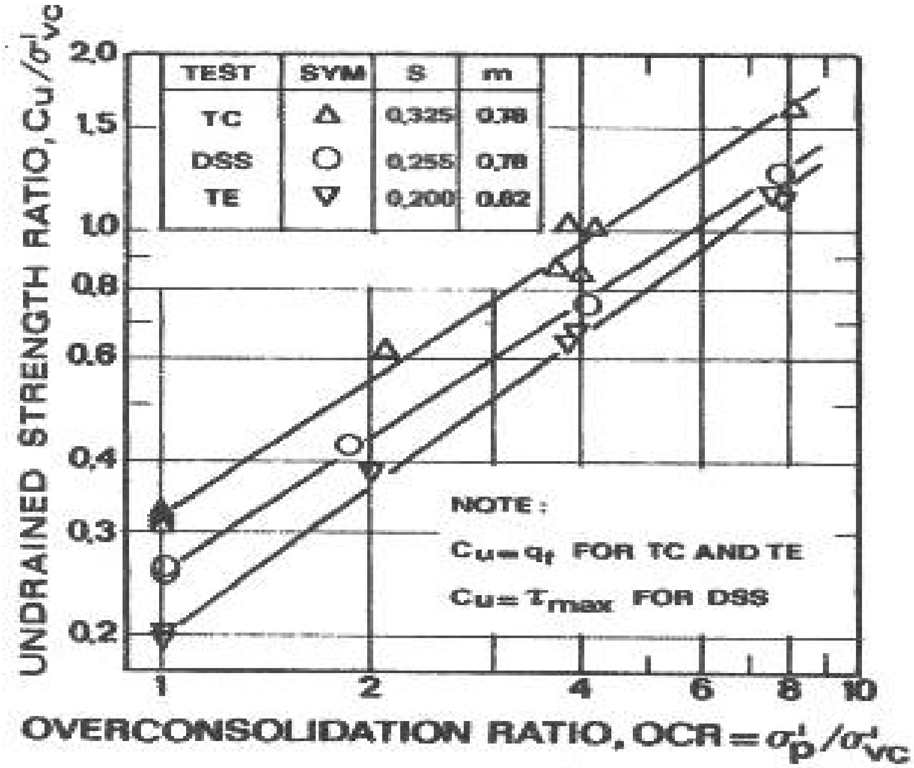


Figure 7-10 Unddrained Shear Strength Ratio vs. OCR for Marine Clay (Jamiolkowski et al. 1985).

Chen and Kulhawy (1993) studied the undrained shear strength interrelationships among CIUC, UU, and UC tests. A database for CIUC and UU tests was developed as shown in Table 7-10 and plotted in Figure 7-11, where S and m equal to 0.4 and 0.58, respectively with $R^2 = 0.77$ as shown in EQ (7-23). This curve or equation can be used to estimate undrained shear strength of clays at particular OCR and overburden pressure.

$$\frac{c_u}{\sigma'_{vc}} = 0.4(OCR)^{0.58} \qquad (7\text{-}23)$$

Table 7-10 Database for CIUC test [a]

Site	Soil description	Depth, m	OCR [b]	c_u/σ'_{vc} [b]	Source
Boston	Medium blue clay	12.2	4	0.8	Ladd and Lambe (1963)
		18.3	1.8	0.48	
		27.4	1	0.32	
Lagunillas	High plasticity clay	6.2	1.3	0.4	Ladd and Lambe (1963)
		6.4	1.3	0.41	
Kawasaki	High plasticity clay	20.5	1.2	0.49	Ladd and Lambe (1963)
		25	1.2	0.45	
		35	1.2	0.48	
Beaumont	High plasticity clay with fissures and slickensides	0 to 8	7.5	1.58	Mahar and O'Neill (1983)
Montgomery	Light grey sandy clay with desiccation	9 to 17	4	1.14	Mahar and O'Neill (1983)
Hamilton	Firm to stiff grey silty clay	3 to 6	3.2	1.16	Ismael and Klym (1978)
		6 to 9	2.2	0.73	
		11	1.2	0.78	
	Firm to stiff grey silty clay (surface desiccated and fissured)	4.6	3.2	1.18	
		7.2	2.5	0.73	
		10.8	1.5	0.78	
		15.2	1.1	0.6	
Lackland	Expansive black to grey clay	0 to 3	5	1.48	Johnson and Stroman (1984)
		3 to 6	4.8	1.12	
	Fissured expansive clay shale	6 to 9	6.5	1.6	
		9 to 12	8.5	1.47	
Rio de Janeiro, Guanabara Bay	Soft grayey clay	2 to 4	2.1	0.62	Ramalho-Ortigao et al. (1983)
		4 to 6	2	0.51	
		6 to 8	1.7	0.46	
		8 to 10	1.7	0.44	
South Padre Island	Medium to stiff clay	6 to 12	1.2	0.6	Focht and Drash (1985)
		15 to 18	1.1	0.51	
		8.2	1.2	0.58	
		14.6	1.2	0.46	
		19	6.4	0.52	

Table 7-10 Database for CIUC test (Cont.) [a]

Site	Soil description	Depth, m	OCR [b]	c_u/σ'_{vc} [b]	Source
St. Alban	Soft to medium silty clay	2.1	2.1	0.99	Roy et al. (1982)
		5.5	2.3	0.85	
		7.2	2.4	0.93	
Boston	Lean and moderately sensitive		18	1.7	D'Appolonia (1972)
	blue clay		12	1.4	
			3.2	0.72	
			1	0.32	
Laboratory	Overconsolidated kaolinite		9	1.28	Duncan and Seed (1966)
result			9	1.03	
Hackensack	Varved clay	7.9 to 15	1.8	0.6	Saxena (1978)
Valley					
Santa Barara	Firm Pleistocene clay	20 to 60	1.6	0.36	Quiros and Young (1988)
Channel	Hard silty clay	100 to 140	1.2	0.26	
Lakeland	Cohesive slimes	0 to 33	1.1	0.41	Ladd (1991)
San Francisco	Soft grey clay (New Bay Mud)	6 to 10	1.4	0.43	Clough and Denby (1980)
Bay Mud		10 to 15	1.3	0.44	
San Francisco	Sandy clay	6 to 9	1.4	0.55	Clough and Denby (1980)
	Soft grey clay	9 to 12	1.2	0.49	
Boston	Marine illitic blue clay		4	0.91	Kinner and Ladd (1973)
			2	0.55	
			1	0.31	
Anacostia		4 to 6	2.1	0.46	Mayne and Frost (1986)
		6 to 9	2.1	0.32	
Tuckerton	Dark grey silty clay	16	8	2.03	Koutsoftas and Fischer
		17	5.2	1.17	(1976)
	Dark grey plastic clay	18 to 23	4	1.16	
Ottawa	Leda clay - moderately OC clay	6 to 9	3.1	1.08	Coates and Mcrostie
	with high plasticity and	9 to 12	2.2	1.02	(1963)
	sensitivity	12 to 15	2	0.95	Eden and Crawford (1957)
		15 to 18	2	0.7	
		18 to 21	1.6	0.68	

Table 7-10 Database for CIUC test (Cont.) [a]

Site	Soil description	Depth, m	OCR [b]	c_u/σ'_{vc} [b]	Source
Madingley	Grey fissured Gault clay with heavily OC clay	3 to 4	20	2.33	Windle and Wroth (1977a)
		4 to 6	18	2.27	
		6 to 7	14	2	Coop and Wroth (1989) Windle and Wroth (1977b)
Southeastern Texas	Very stiff clay with high plasticity	15.2	6.5	0.87	Endley et al. (1979)
		18.3	5.8	0.75	
		21.3	2.9	0.64	
Empire	Fine grey clay	36.6	1.2	0.27	Cox et al. (1979)
Chicago	Hard silty clay	3.7	17	2.5	Holtz and Baker (1972)
		9	20	2.22	
		10	22	2.35	
		11.6	39	2	
Gulf of Mexico	Soft plastic clay	0 to 6	3.5	0.86	Fenske (1956)
	Firm to stiff plastic clay eith silt and sand	18 to 32	1.1	0.34	
		32 to 50	1.2	0.29	
Skabo	Plastic clay with high salt content	10.6 to 16	1.2	0.45	Ladd and Lambe (1963)
Gota Valley	Lilla Edet clay - marine late-glacial plastic clay with high sensitivity	4 to 6.8	12	1.44	Bjerrum and Wu (1960)
		10 to 12.3	1.8	0.76	
		16.2 to 18	1.5	0.52	
		10.8 to 13	1.5	0.32	
Drammen	Soft silty clay with thin seams of silt and fine sand	5 to 12	1.3	0.34	Simons (1960)
		18	1.1	0.33	
Sault Ste. Marie	Varved glacial lake clay with floccuent structure	9	1.2	0.23	Wu (1960) and Wu et al. (1962)
Kars	Cemented Leda clay	2.5 to 6	7	1.47	Raymond (1972)
		6 to 12	2.5	0.72	

[a] Adapted from Chen and Kulhawy (1993).
[b] Values were taken from either single point or an average over a certain depth.

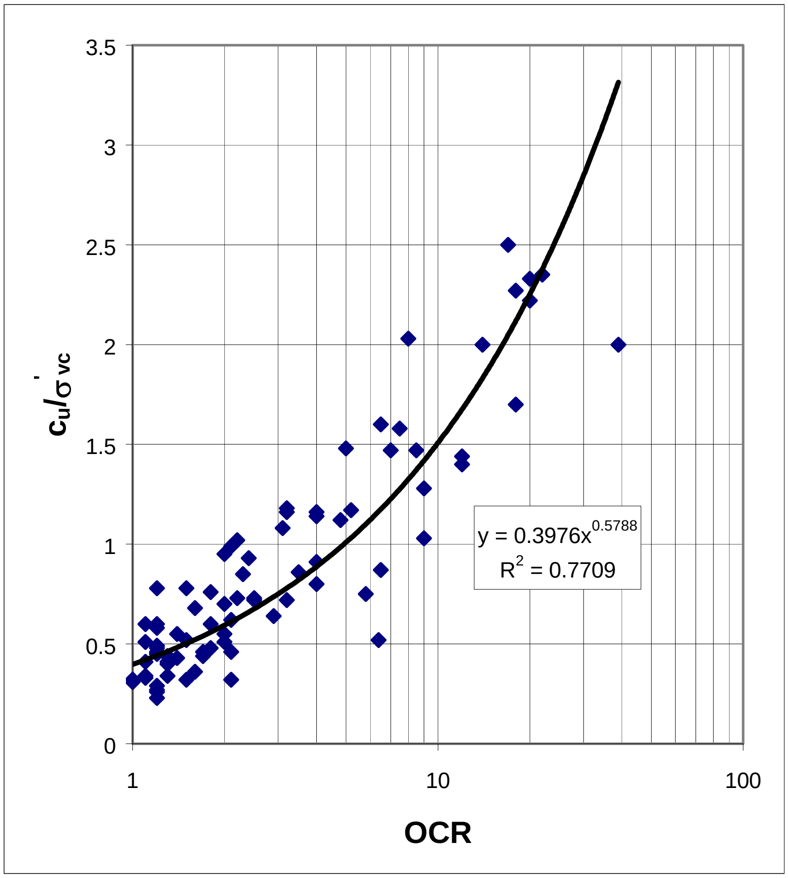


Figure 7-11 Undrained Shear Strength Ratio vs. OCR from developed from the database in Chen and Kulhawy 1993

7.2.10.3 Analysis of E_s/c_u Ratio

In section 7.2.2, relationships between E_s and c_u were given. From Table 7-2, the ratio of E_{50} to c_u can be grouped into two ranges based on values as shown in Table 7-11. E_{50}/c_u ratio by Simons (1976) is not included in Table 7-11 because the ratio is out of range. Ratios by Corps of Engineers are also not shown because this method requires more detailed information of soils. Note that grouping was done regardless of type of clays and test procedures. For each range, the common range (i.e. the range that can be found in every sub-range except ratios by Poulos) of E_{50}/c_u can be obtained and shown in Table 7-12.

From Table 7-2, the range and average values of E_i/c_u can be shown in Table 7-13. The common range for E_i/c_u ratio is 100 to 200. It is obviously that E_i/c_u is greater than E_{50}/c_u. The common range of E_{50}/c_u from range I give the reasonable values compare with E_i/c_u. By the same token, the average range of E_i/c_u is suggested. The ratios of E_{50}/c_u or E_i/c_u are approximate and can be used as a guide when only undrained shear strength is available.

Table 7-11 Ratio of E_{50} to c_u for Clays [a]

Range No	Sub-range No	Study	Ratio of E_{50}/c_u	Remark
I	1	Poulos (1971)	15 to 95	Piles in uniform, elastic soils
	2	Skempton (1951)	50 to 200	Bearing capacity of clays and c_u from unconfined compression tests
	3	Banerjee and Davies (1978)	100 to 180	For soft clays and E_s vary linearly with depth.
	4	Sullivan (1980)	100 to 250	For OC clays and c_u from UU triaxial tests
II	1	Bjerrum (1972)	500 to 1500	c_u from vane shear tests
	2	Davies and Budhu (1986)	500 to 1000	For stiff clays and c_u from UU triaxial tests
	3	Bowles (1988) [b]	200 to 2000	Lower values for NC clays and higher values for heavily OC clays

[a] Ratio of E_s/c_u by Simons (1976) and Corps of Engineers are not shown.
[b] Elasticity by Bowles is assumed to be secant elasticity.

Table 7-12 Overlapping Ranges and Average Values of E_{50}/c_u

Range No	Overlapping range of E_{50}/c_u	Average values of E_{50}/c_u
I	50 to 180	115
II	180 to 500	340
III	500 to 1000	750

Table 7-13 Ratio of E_i/c_u

Study	Minimum Value of E_i/c_u	Maximum Value of E_i/c_u
Matlock et al (1956)	40	200
Reese et al (1968)	100	200
Poulos and Davis (1980)	250	400
Average	130	265 [a]

[a] Round off to whole number

The Ratio of E_s/c_u (E_s is the secant modulus of elasticity at a designated strain level) depends on shear stress level as shown in Figure 7-12. E_u in Figure 7-12 represents the secant modulus of elasticity of clays. The E_u/c_u ratios were plotted versus the shear stress level (τ_h/c_u) from CK_0U direct simple shear tests on seven NC clays. From Figure 7-12, E_u/c_u ratios decrease with increasing values of shear stress ratios. E_u/c_u also decrease with increasing plasticity and organic content of clays (soil 1 to soil 7). This trend can also be seen in Figure 7-2.

The ratios of E_s to c_u are dependent of OCR. Figure 7-13 shows E_u/c_u (E_u is secant modulus) were plotted with OCR at two shear stress levels. The values of E_u/c_u can be either increased or decreased for low OCR. But for high OCR, ratios of E_u/c_u tend to decrease with OCR (See also Figure 7-2).

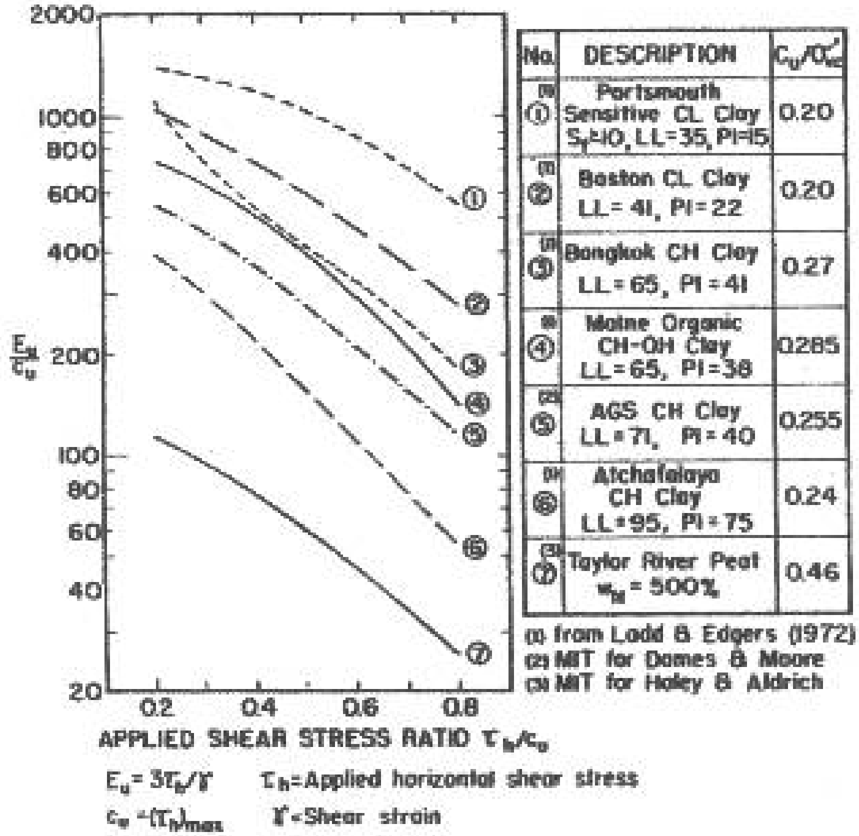


Figure 7-12 E_u/c_u vs. Shear Stress Ratio (Ladd et al. 1977)

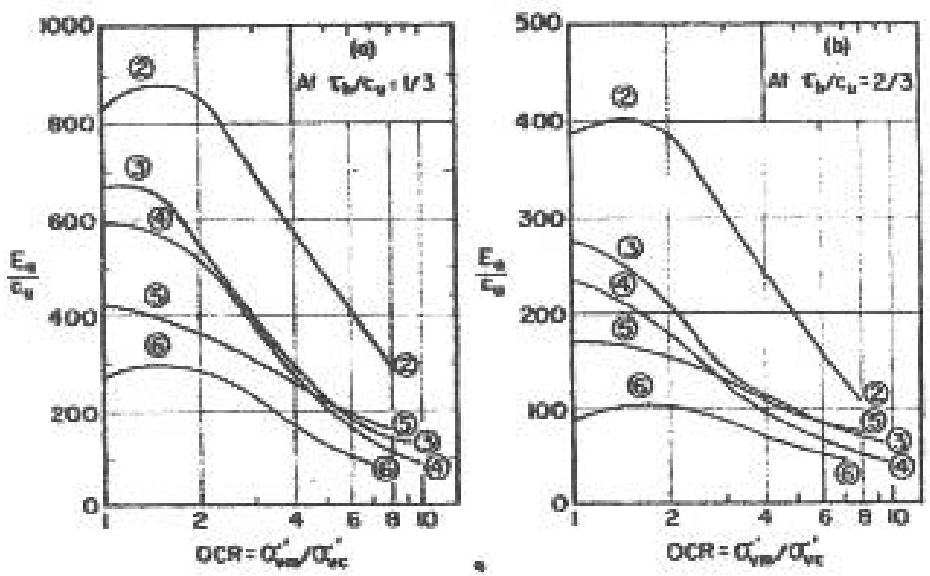


Figure 7-13 E_u/c_u vs. OCR at Two Shear Stress Levels (Ladd et al. 1977)

E_s/c_u depends on many factors: shear stress, plasticity of clays, organic content, and stress history (OCR). These factors need to be considered in selecting the ratio of E_s/c_u. Based on analyses of E_s/c_u in this section, these recommendations for selecting E_s/c_u can be drawn below:

1) In case of no c_u data, the undrained shear strength of clays can be estimated by procedures in Section 7.2.10. Then E_s/c_u or E_i/c_u can be estimated from 2).
2) In case of only undrained shear strength data from laboratory or in-situ tests are available, the values of E_{50}/c_u of 50 to 180 and the values of E_i/c_u of 130 to 265 can be used with great care.
3) If c_u, OCR, and PI data is available, elasticity proposed by Corps of Engineers (1990) is recommended.

7.2.10.4 Modulus of Elasticity for Clays

If possible, lateral pile load tests are recommended and the elasticity of soils is back-figured. These E_s are most reliable values for developing the numerical model, which needs to be calibrated before being used to predict behavior of piles under lateral loads. Data from field tests are used for calibrating numerical models. Then values of E_s can be back-figured. Jamiolowski et al. (1985) mentioned that back-analysis from full-scale loading tests are the most reliable assessment of geotechnical design parameters for piles under lateral loads (Poulos and Davis 1980). Figure 7-14 shows the E_s versus c_u plot of piles in clays, where E_s values were back-figured from a number of published pile-test results. It is seen that E_s increases with increasing c_u for both driven piles and drilled shafts until C_u reaches some limits whereon E_s become constant. For drilled shafts, this limit is 135 kN/m^2 and 95 kN/m^2 for driven piles. At the same value of C_u, E_s is larger for driven piles than drilled shafts.

Soil moduli from laboratory tests usually are much lower than those from back-figured (Jardine et al. 1986, 1985, 1984, and Poulos and Davis 1980). This may be caused by samples disturbance and nonhomogeneities of natural soils. The limitation of laboratory methods increases when dealing with heavily OC clays, conditions at small strains (Jamiolkowski et al. 1985), and fissured stiff clays (Callanan and Kulhawy 1985). In-situ tests also have their own limitations with the greatest being the interpretation of data based on empirical correlations. Table 7-14 shows comparison of advantages and limitations of in-situ versus laboratory testing for cohesive soils. The earliest in situ test method is plate load test. E_s from plate load tests were 1.8 to 4.8 times greater than those determined from undrained triaxial tests (Marland 1971a). Poulos and Davis (1980) commented that E_s from plate load tests at various depths gave satisfactory load-deflection predictions for piles.

Pressuremeter tests (PMT) give stress-strain curve that have potential for use in predicting performance of both axially and laterally loaded foundations (Huang 1995). Fryman, et al. (1975) reported the use of pressuremeter test results produced good agreement between calculated and measured behavior. Self-boring pressuremeter (SBP) gives soil moduli for use in the calculation of both vertical settlement and lateral deformation of soils. SBP can shear soils in the horizontal direction and yields a horizontal shear modulus (Jamiolkowski et. al. 1985).

Dilatometer tests (DMT) can also be used in determining E_s for clays. Some studies were reported to apply DMT with laterally loaded piles (Davidson and Boghrat 1983 and Robertson et al. 1989).

The direct methods for determining E_s of soils are laboratory testing, in-situ tests, and back-analysis. Each method has advantages and limitations. Many factors should be taken into consideration on selecting one of these methods. Cost-benefit, resources, times, nature of problems, and nature of soils at site play the important roles in the decision.

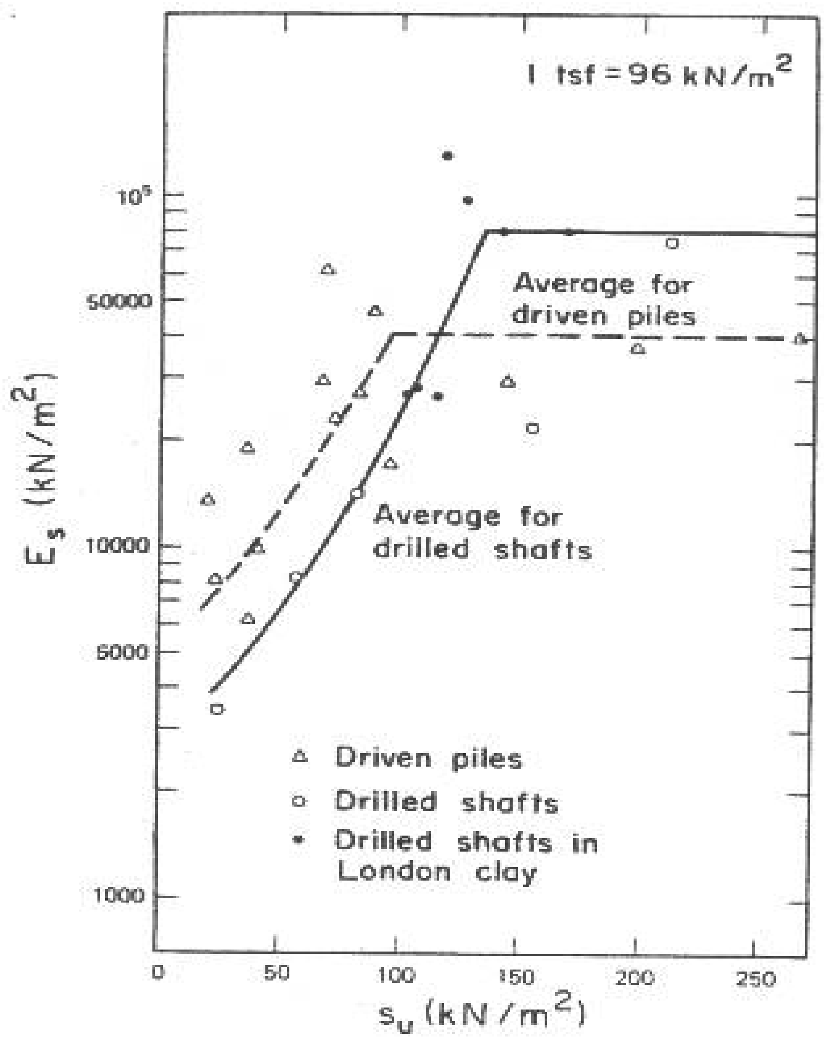


Figure 7-14 Back-figured E_s for Piles in Clay (Poulos and Davis 1980)

Table 7-14 Comparison of Advantages and Limitations of In-situ versus Laboratory Testing for Cohesive Soils [a]

In Situ Testing	Undisturbed Laboratory Testing
Advantages	
1. Response of large volume of soil. 2. Response to natural environment, i.e., in situ temperature and no stress relief. 3. More economical and less time-consuming. 4. Semi-continuous profile. 5. Can be carried out in soil in which undisturbed sampling is still impossible or unreliable.	1. Known soil type, nature, and physical features. 2. Well-defined boundary conditions. 3. Well-defined stress paths. 4. Strain rates can be specified. 5. Strictly controlled drainage conditions.
Limitations	
1. Unknown effects of advancing devices in the ground. 2. Poorly defined stress and strain boundary conditions. 3. Cannot control drainage conditions. 4. Strain fields are nonuniform and strain rates are higher than those anticipated in the foundation.	1. Expensive and time-consuming. 2. Effects of sample disturbance may be difficult to identify and minimize. 3. Small, discontinuous test specimens. 4. Discontinuous nature of information obtained. 5. Unavoidable stress relief.

[a] Compiled from Ladd et al. 1992 and Jamiolkowski et al. 1985.

7.2.11 Ramberg-Osgood (RO) Parameters for NIKE3D

This section presents the method for the evaluation of Ramberg-Osgood (RO) parameters and their sensitivity. There are five important parameters in the Ramberg-Osgood model: reference shear stress (τ_y), reference shear strain (γ_y), stress coefficient (α), stress exponent (r), and bulk modulus (K). Figure 7-15 shows its typical backbone curve. RO parameters can be determined using RAMBO software. Input information for RAMBO includes soil type (sand or clay) and density.

Table 7-15 shows output from RAMBO for clay with density of 3.80 slug/ft^{3,} where V_s is shear wave velocity from Cross-Hole test and G_{max} is maximum shear modulus. If shear wave velocity data are provided, G_{max} and other parameters can be found. Shear modulus (G) can be estimated by using 85 to 90 % of G_{max}. Shear modulus can be related to bulk modulus by EQ (7-24).

$$K = \frac{2G(1+\nu)}{3(1-2\nu)} \qquad (7\text{-}24)$$

where ν is Poisson's ratio.

The effects of RO parameters on the shape of backbone curve are investigated. The backbone stress-strain curve for monotonic loading of Ramberg-Osgood is described by EQ (7-25):

$$\frac{\gamma}{\gamma_y} = \frac{\tau}{\tau_y}\left(1 + \alpha\left|\frac{\tau}{\tau_y}\right|^{r-1}\right) \qquad (7\text{-}25)$$

where γ is shear strain
τ is shear stress
γ_y is reference shear strain
τ_y is reference shear stress
α is stress constant and $\alpha \geq 0$
r is stress exponent and $r \geq 1.0$

The typical stress-strain (backbone) curve for G_{max} = 1436400 psf, τ_y = 515.34 psf, γ_y = 0.000359, α = 1.257, and r = 2.441 is shown in Figure 7-15, where the slope of the backbone curve is secant shear modulus (G) defined by τ/γ at given shear strain level, G_{max} is the maximum value of shear modulus defined by τ_y/γ_y, reference shear strain defined as the strain at the point of maximum curvature in backbone curve, and reference shear stress is the shear stress at reference γ_y.

Table 7-15 Output from RAMBO for Clay with ρ = 3.80 slug/ft^3

V_s (ft/sec)	G_{max} (psf)	α	R	γ_y	τ_y (psf)
100	38000	1.257	2.441	0.0003588	13.6
150	85500	1.257	2.441	0.0003588	30.7
200	152000	1.257	2.441	0.0003588	54.5
250	237500	1.257	2.441	0.0003588	85.2
300	342000	1.257	2.441	0.0003588	122.7
350	465500	1.257	2.441	0.0003588	167.0
400	608000	1.257	2.441	0.0003588	218.1
450	769500	1.257	2.441	0.0003588	276.1
500	950000	1.257	2.441	0.0003588	340.8
550	1149500	1.257	2.441	0.0003588	412.4
600	1368000	1.257	2.441	0.0003588	490.8
650	1605500	1.257	2.441	0.0003588	576.0
700	1862000	1.257	2.441	0.0003588	668.0
750	2137500	1.257	2.441	0.0003588	766.9
800	2432000	1.257	2.441	0.0003588	872.5
850	2745500	1.257	2.441	0.0003588	985.0
900	3078000	1.257	2.441	0.0003588	1104.3
950	3429500	1.257	2.441	0.0003588	1230.4
1000	3800000	1.257	2.441	0.0003588	1363.4
1050	4189500	1.257	2.441	0.0003588	1503.1
1100	4598000	1.257	2.441	0.0003588	1649.7
1150	5025500	1.257	2.441	0.0003588	1803.0
1200	5472000	1.257	2.441	0.0003588	1963.2
1250	5937500	1.257	2.441	0.0003588	2130.2
1300	6422000	1.257	2.441	0.0003588	2304.1
1350	6925500	1.257	2.441	0.0003588	2484.7
1400	7448000	1.257	2.441	0.0003588	2672.2
1450	7989500	1.257	2.441	0.0003588	2866.4
1500	8550000	1.257	2.441	0.0003588	3067.5
1550	9129500	1.257	2.441	0.0003588	3275.4
1600	9728000	1.257	2.441	0.0003588	3490.2
1650	10345500	1.257	2.441	0.0003588	3711.7
1700	10982000	1.257	2.441	0.0003588	3940.1
1750	11637500	1.257	2.441	0.0003588	4175.3
1800	12312000	1.257	2.441	0.0003588	4417.2

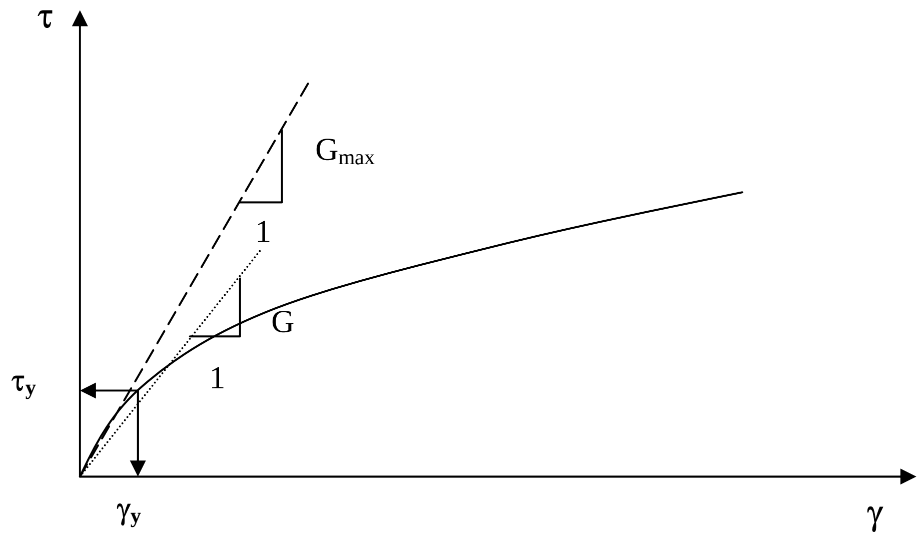


Figure 7-15 Typical Backbone Curve

Effect of γ_y on Backbone Curves

This section discusses the effect of reference shear strain on backbone curves. Using the same data as the base curve, Figure 7-16, except γ_y, two new backbone curves can be drawn with $\gamma_y = 0.01$ and 0.0001 respectively. Figure 7-17 shows as the reference shear strain increases; the backbone curve flattens itself, which also results in the decrease of G_{max} when τ_y remains constant and γ_y increases or τ_y/γ_y decreases.

Effect of α

Effect of α (alpha) can be seen from Figure 7-18. As α increases, the backbone curve flattens. The initial tangent shear modulus G_{max} remains the same, but the curve beyond reference point become flatter as α increases.

Effects of r on Backbone Curves

Stress exponent (r) is varied from 2.441 in base curve to 1.75 and 3.9. Figure 7-19 shows that the effect of r is similar as the effect of α, but at a much greater degree. The above sensitivity study can assist in the calibration process. Besides, the further study on the RO parameters for different soils will be useful to the finite element analysis of geostructure behavior and soil-structure interaction.

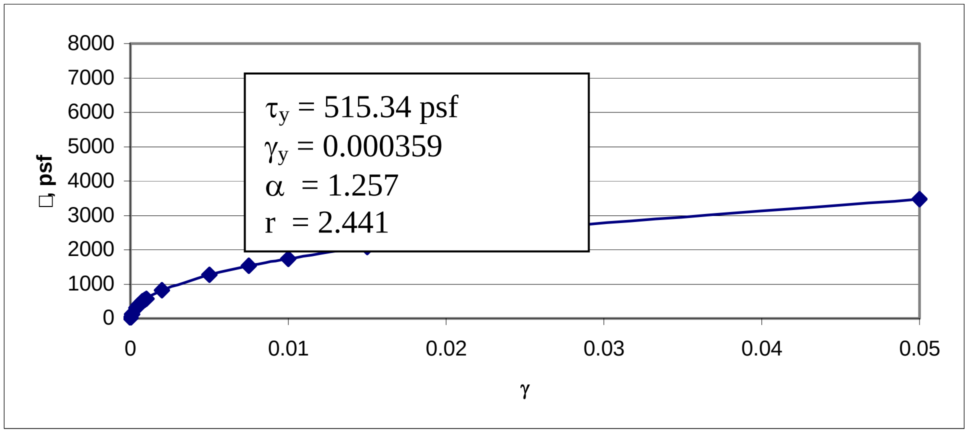


Figure 7-16 Typical Stress-Strain Curve (Base Curve)

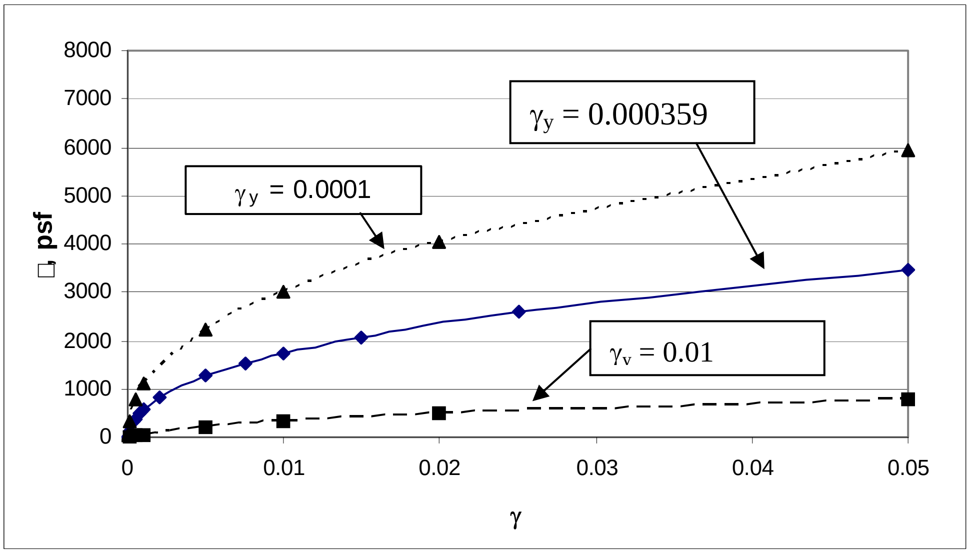


Figure 7-17 Effects of γ_y on Backbone Curves

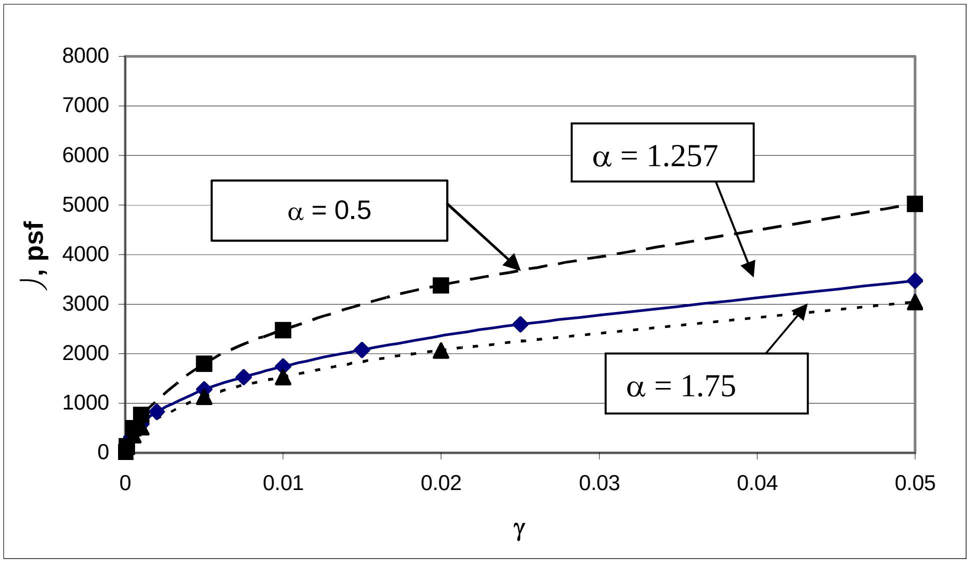


Figure 7-18 Effects of α on Backbone Curves

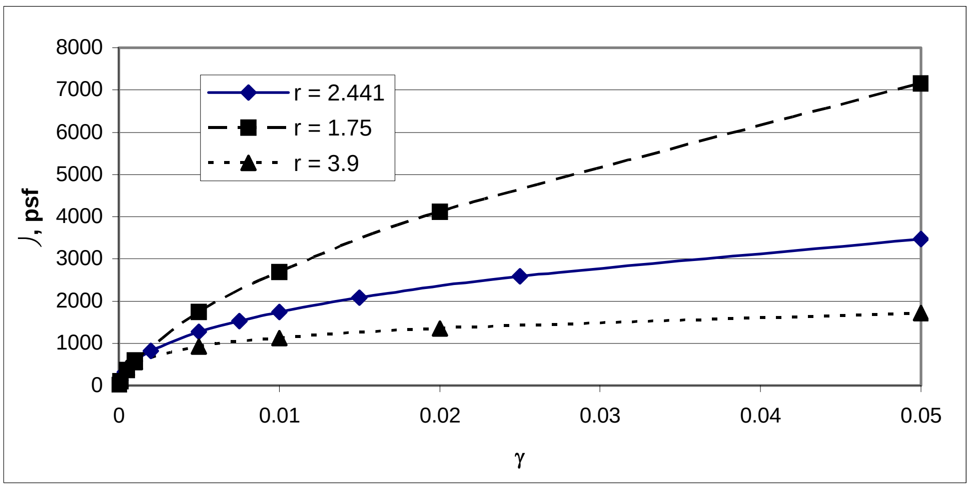


Figure 7-19 Effects of r on Backbone Curves

7.3 Broms Method

7.3.1 Fundamental Theory and Assumptions

In this section, Broms theory and assumptions are presented. Broms method is used to predict the behavior of laterally loaded piles. It is based on the subgrade reaction concept to calculate lateral deflections of piles.

7.3.1.1 Fundamental Theory

Broms proposed his theory in 1964. The objectives were to calculate the ultimate lateral resistances (P_{ult}), lateral ground-line deflections (y_0) at working loads, and maximum bending moments (M_{max}) in pile. The working load is defined at 1/3 to ½ of the ultimate lateral resistance (P_{ult}). Broms proposed the failure mechanism for relatively short and long piles. Figures 7-20-a and b show the failure modes for long and short piles, respectively. For long piles, the failures take place when pile section yields (or cracks) at the location of the maximum bending moment. For short piles, the piles are considered to fail when they rotate and the soils along the piles fail.

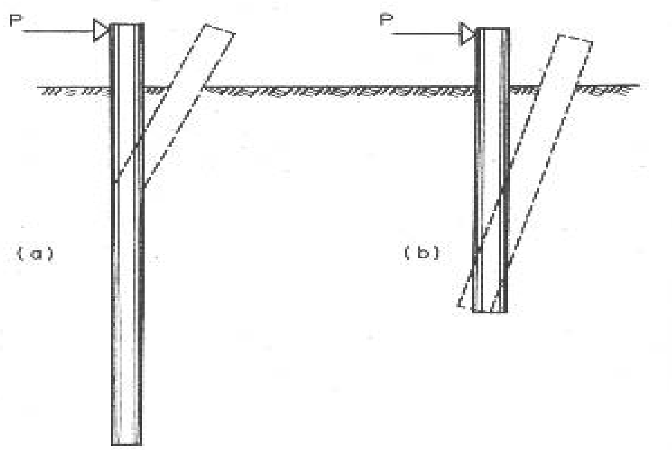


Figure 7-20 Failure Modes for Short and Long Piles (Broms 1964a)

Soil reaction distributions and maximum are shown in Figures 7-21 and 22, respectively. The ultimate lateral load and the maximum bending moment can be calculated from soil reaction distribution. The soil reaction and bending moment distribution for short and long piles are shown on Figures 7-22 and 7-22, respectively.

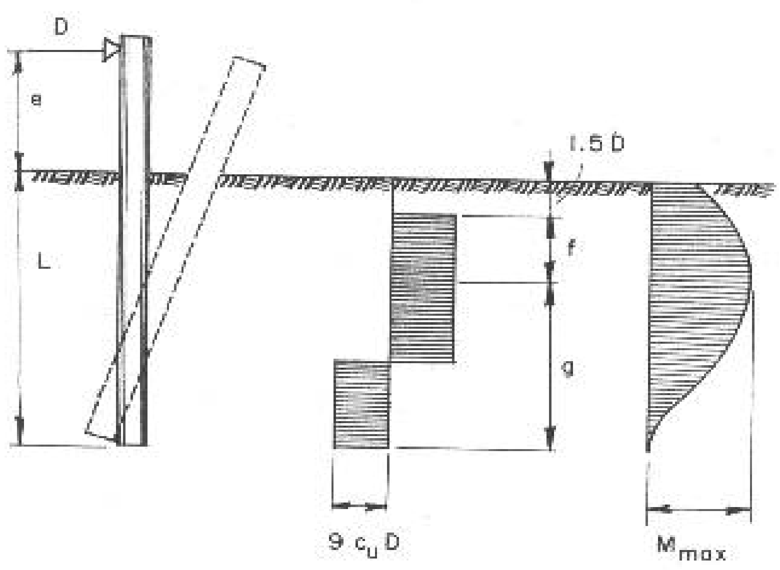


Figure 7-21 Soil Reaction and Bending Moment Distribution for Short Piles (Broms 1964a)

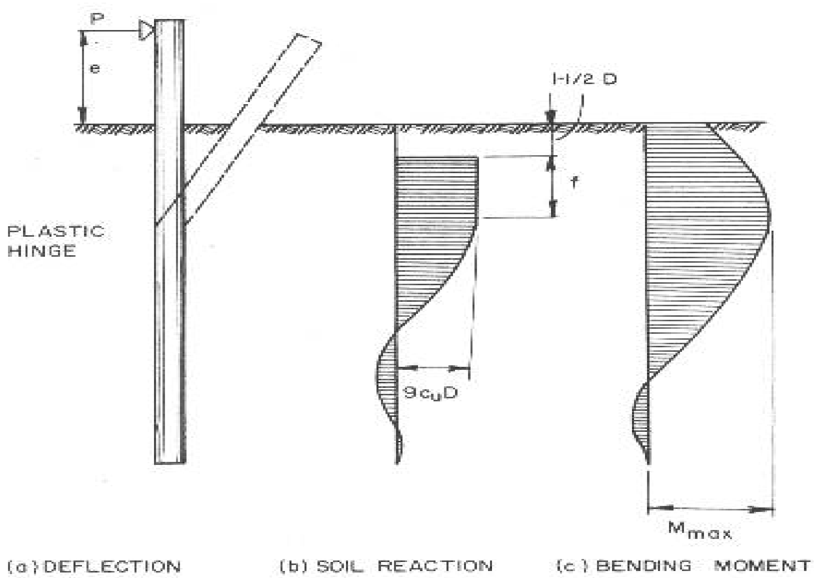


Figure 7-22 Soil Reaction and Bending Moment Distribution for Long Piles (Broms 1964a)

For short piles, soil reaction distributions are assumed to be linear with zero soil pressure from ground surface to depth of 1.5D. The point of rotation is the point at which soil reaction changes to the opposite sign. The point of rotation is located at mid-way of the distance '*g*' in Figure 7-21. Bending moment distributions can be calculated from soil reaction distributions. In Figure 7-21 and 7-22, the distance '*f*' is the distance from depth of 1.5D to the depth of maximum bending moment. c_u is undrained shear strength of clay.

For long piles, the soil reaction distribution is assumed to be constant ($9c_uD$) up to the depth of maximum bending moment. Then the distribution is nonlinear below this depth. Behavior of piles can be specified by dimensionless length βL ($\beta = \sqrt[4]{\frac{k_h D}{4EI}}$), where L is length of pile below ground surface, EI the effective stiffness of piles, k_h the coefficient of horizontal subgrade reaction, and D pile diameters. In general, βL is greater than 2.25 for long piles and less than 2.25 for short piles. Also in case of short piles, M_{max} are less than M_{ult}, for long piles, M_{max} and M_{ult} are equal. Flow chart for determining behavior of piles is shown on Figure 7-23.

7.3.1.2 Assumptions

Broms method is simple method for solving plies under lateral loads. Consequently, the following simplified assumptions are made:

1. Working load is 1/3 to ½ of ultimate lateral resistance.
2. At working loads, lateral forces vary linearly with lateral deflections.
3. Short pile (or rigid pile) fails when pile rotate as a unit and soil around pile fails.
4. Long pile (flexible pile) fails when plastic hinge is formed and M_{max} reaches at M_{ult}.

In addition to general assumptions, there are following assumptions for piles in cohesive soil:

5. Broms theory can be applied to driven piles into saturated clays.
6. The coefficients of subgrade reaction in both vertical and horizontal directions are the same.
7. The coefficient of subgrade reaction is constant with depth.

One needs to take note of the above assumptions and limitations when using Broms theory.

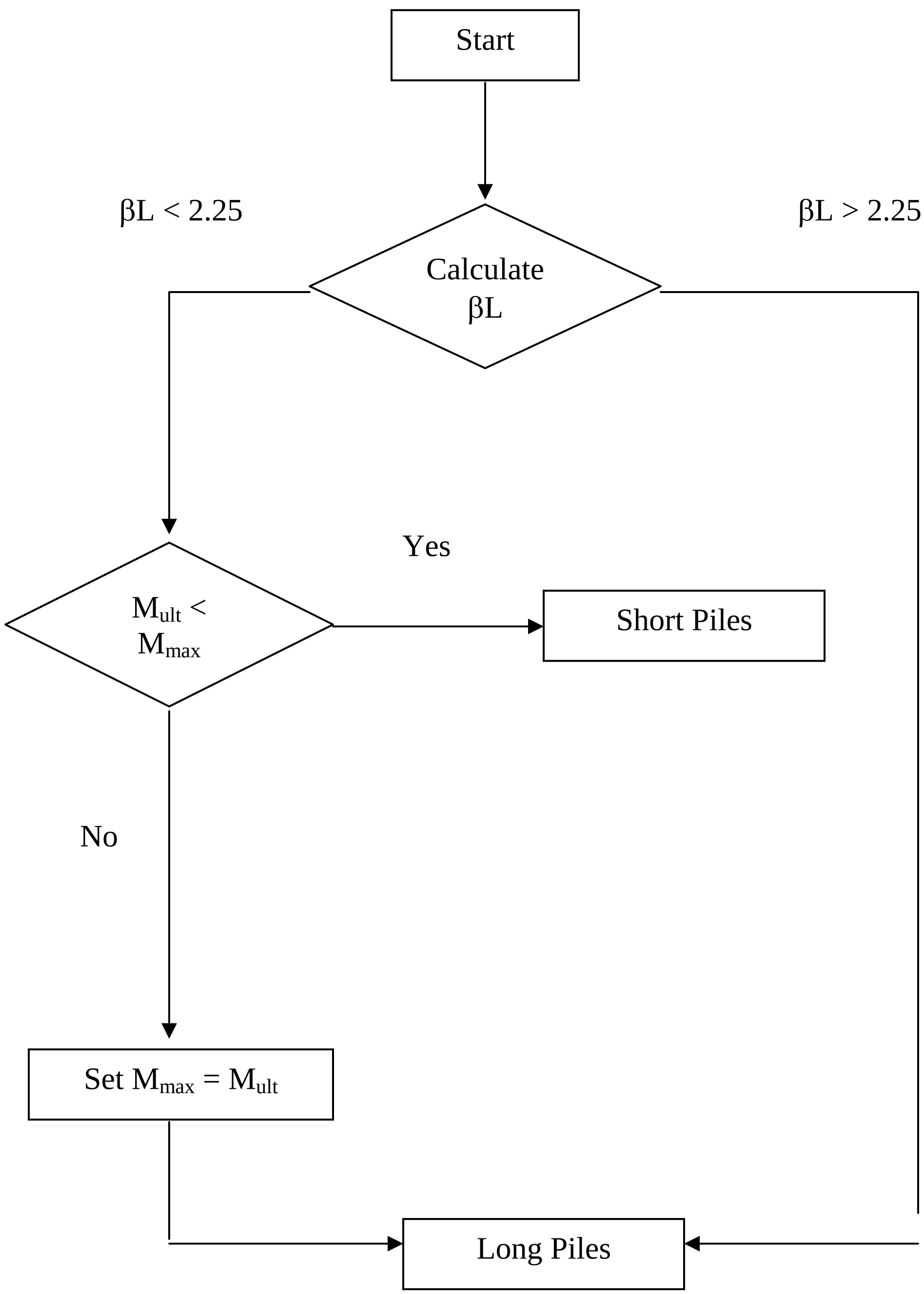


Figure 7-23 Flow Chart for Determining Behavior of Piles

7.3.2 Lateral Ground-line Deflections (y_0) at Working Loads

This section discusses equations and graphical solutions for calculating y_0. In some situations, lateral deflections at working loads control the design of laterally loaded piles or drilled shafts. As mentioned before, working load equals to 1/3 to ½ of P_{ult}. In the range of working loads, the lateral deflections are assumed to vary linearly with the applied loads Figure 7-24 shows the pile subjected to the lateral load. The lateral deflections at working load of piles can be calculated using both equations and graphical solutions. The lateral deflections of short piles depend on the magnitude of loads, pile dimensions, and soil properties. For short piles with $\beta L < 1.5$, the equation for calculating y_0 is shown in EQ (7-26).

$$y_0 = \frac{4P(1+1.5\frac{e}{L})}{k_h DL} \qquad (7\text{-}26)$$

where P is lateral load
e is the eccentricity or distance from ground surface to the point of load application
L is the length of pile below ground surface
k_h is the coefficient of horizontal subgrade reaction
D is pile diameter

For long piles with $\beta L > 2.5$, y_0 can be determined from

$$y_0 = \frac{2P\beta(e\beta+1)}{k_\infty D} \qquad (7\text{-}27)$$

where k_∞ is the coefficient of subgrade reaction for long piles.

For piles with βL up to 5.0, Broms provided graphical solutions for lateral deflections as shown on Figure 7-25. In this graphs, ratio of e to L (e/L) is required. The calculation of lateral deflections is based on the subgrade reaction concept. The important soil property is the coefficient of subgrade reaction. This soil parameter will be discussed later.

7.3.3 The Ultimate Lateral Resistances of Piles (P_{ult})

Lateral deflections require the values of ultimate lateral loads of piles (P_{ult}). Ultimate lateral loads are based on hypothesized soil reaction distributions. The calculation of P_{ult} is based on behavior of the pile. The procedure to determine

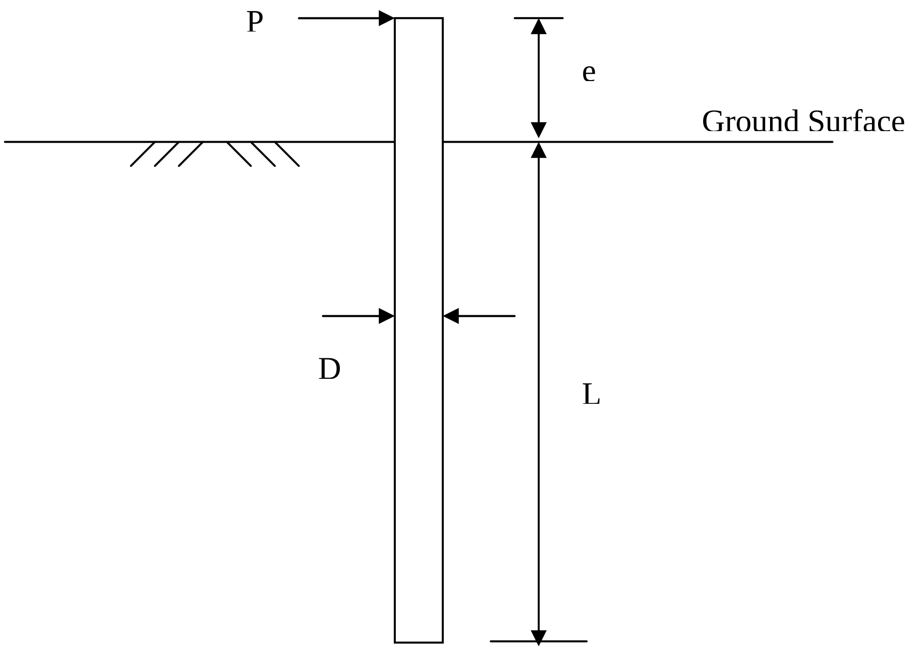


Figure 7-24 Pile Subjected to the Lateral Load

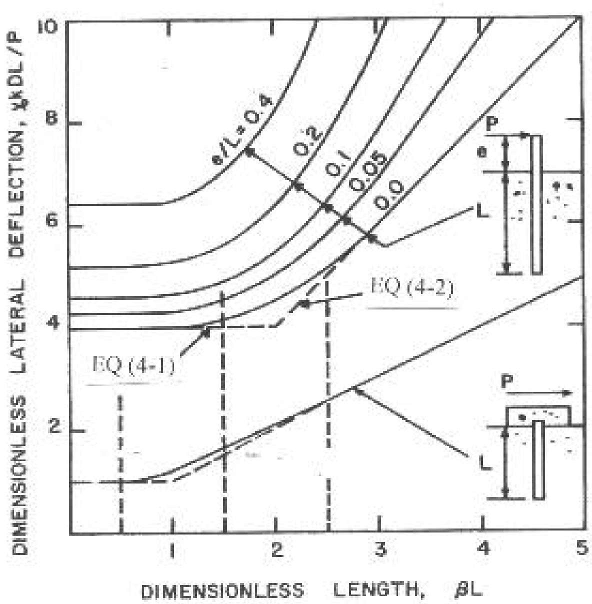


Figure 7-25 Lateral Deflections at Ground Surface (Broms 1964a)

pile behavior was given in the previous section. The ultimate lateral resistance of pile depends on pile behavior. P_{ult} for short and long piles are different depending on failure mechanism and behavior.

7.3.3.1 Short Piles

Short piles fail when they rotate as a unit and the soils around pile fail. Failure mechanism is shown in Figure 7-20. Based on the soil reaction distribution, the bending moment distribution can be determined. The distance *f* in Figure 7-21 can be calculated from:

$$f = \frac{P_{ult}}{9 c_u D} \tag{7-28}$$

The maximum bending moment can be determined by

$$M_{\max} = P_{ult}(e + 1.5D + 0.5f) \tag{7-29}$$

M_{max} is also function of the distance *g* as follow

$$M_{\max} = 2.25 c_u D g^2 \tag{7-30}$$

The pile length needs to be checked with EQ (7-31):

$$L = 1.5D + f + g \tag{7-31}$$

The solution procedure is iterative and follows the following steps:

1. Assume '*f*' and calculate P_{ult} from EQ (7-28).
2. Calculate M_{max} and '*g*' from EQ (7-29) and (7-30), respectively.
3. Compute calculated L from EQ (7-21) and compare with known L. If the calculated L is not equal to known L, value of '*f*' needs to be adjusted.
4. Repeat from step 1 until calculated L is equal to known L.

Figure 7-26 shows the flow chart of the solution procedures for short piles.

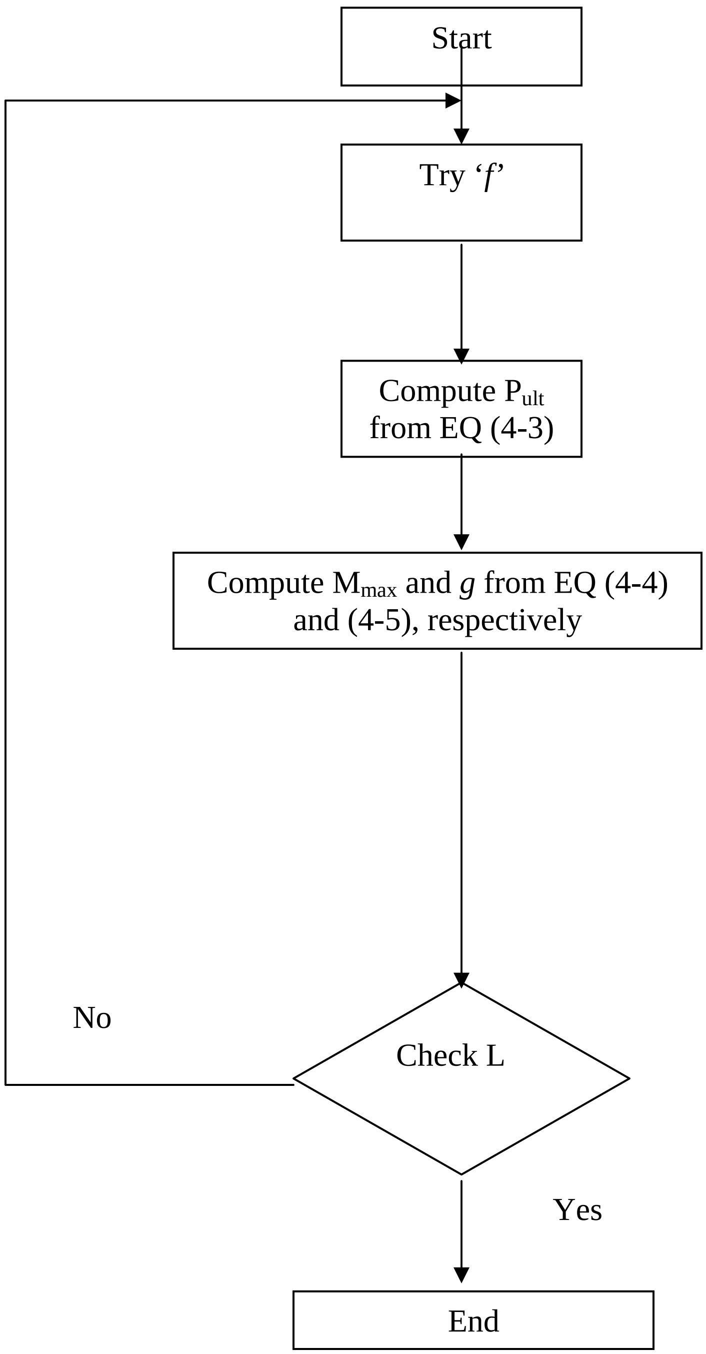


Figure 7.26 Solution Procedures of P_{ult} for Short Piles

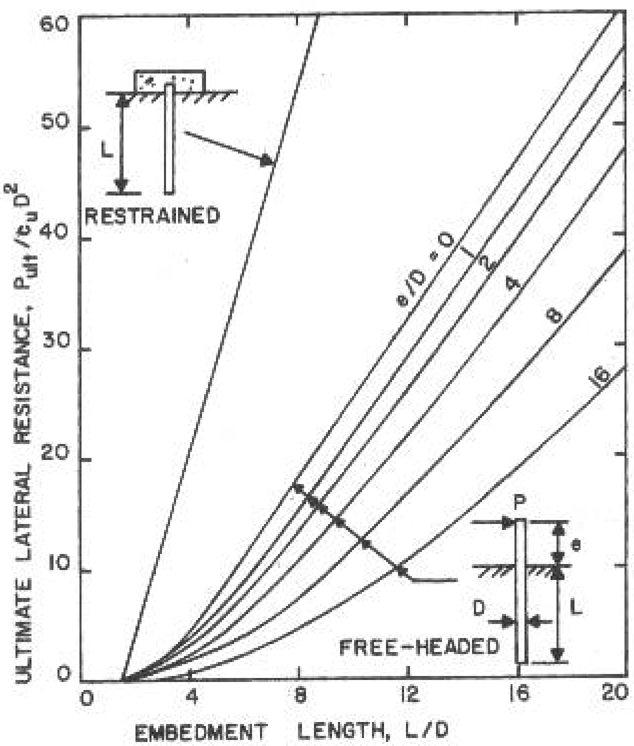


Figure 7-27 Graphical Solution of P_{ult} for Short Piles (Broms 1964a)

7.3.3.2 Long Piles

Failure mechanism of long piles can be seen from Figure 7.20. Soil reaction and bending moment distributions are shown on Figure 7-22. For long piles, the ultimate lateral resistances can be expressed in term of the distance *f*.

$$P_{ult} = 9c_u Df \tag{7.32}$$

The maximum bending moments in the piles are equal to ultimate moments of pile sections and can be related to P_{ult} and '*f*' as shown in EQ (7-33).

$$M_{\max} = M_{ult} = P_{ult}(e + 1.5D + 0.5f) \qquad (7\text{-}33)$$

From EQ (7-32) and (7-33), substitute P_{ult} from EQ (7-32) into EQ (7-33) and rearrange the terms. Then we get

$$4.5c_u Df^2 + 9c_u D(e + 1.5D)f - M_{ult} = 0 \qquad (7\text{-}34)$$

EQ (7-34) is a quadratic equation. This equation is solved for *f* as shown in EQ (7-35).

$$f = \frac{-b \pm \sqrt{b^2 - 4ac}}{2a} \qquad (7\text{-}35)$$

where $a = 4.5c_uD$

$b = 9C_uD\,(e+1.5D)$

$c = -M_{ult}$

Using positive number of f, P_{ult} can be determined by either EQ (7-32) or (7-33). Broms also provided the graphical solutions for P_{ult}. Solutions for short and long piles are shown on Figures 7-27 and 7-28, respectively.

7.3.4 Coefficient of Horizontal Subgrade Reaction (k_h)

The coefficient of horizontal subgrade reaction is the most important soil parameter for calculating lateral deflections using Broms method. Broms assumes the vertical and horizontal coefficients of subgrade reaction to be the same. He proposed the method to estimate the coefficient of subgrade reaction for long piles based on plate load test results. The coefficient of subgrade reaction for long pile with diameter or width D can be determined:

$$k_\infty = \frac{\alpha K_0}{D} \tag{7-36}$$

where α is defined as:

$$\alpha = 0.52\left(\frac{K_0 D^4}{EI}\right)^{1/12},$$

"K_0" is the coefficient for circular plate,

"EI" is the stiffness of the circular plate.

The coefficient α can be approximately determined by

$$\alpha = n_1 n_2 \tag{7-37}$$

where n_1 and n_2 can be found from Table 7-16 and 7-17, respectively.

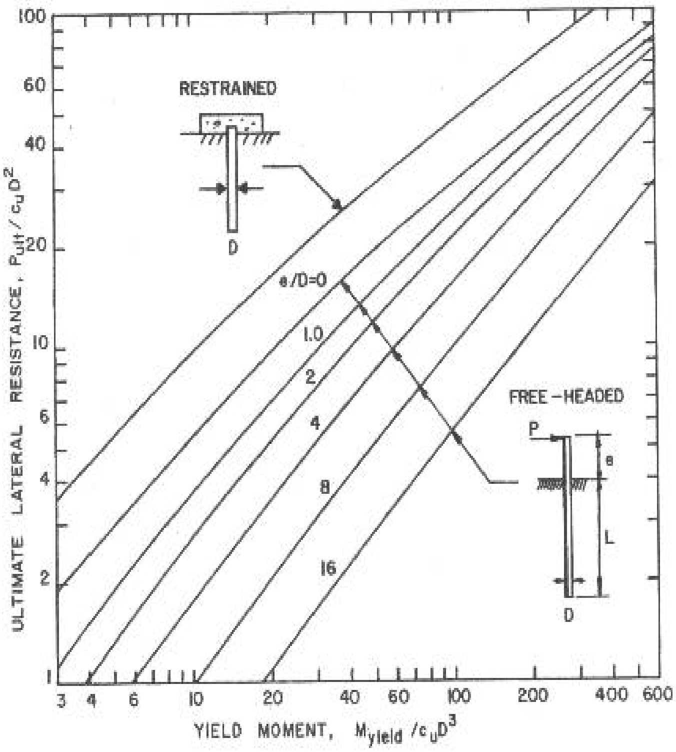


Figure 7-28 Graphical Solution of P_{ult} for Long Piles (Broms 1964a)

Table 7-16 The Coefficient n_1 (Broms 1964a)

Unconfined Compressive Strength q_u, tsf	**Coefficient n_1**
Less than 0.5	0.32
0.5 to 2.0	0.36
Greater than 2.0	0.40

Table 7-17 The Coefficient n_2 (Broms 1964a)

Pile Material	**Coefficient n_2**
Steel	1.00
Concrete	1.15
Wood	1.30

The coefficient K_0 can be determined from plate load tests. The coefficient of subgrade reaction is defined by q/d_0, where q is the intensity of the applied load and d_0 is the deflection of the circular plate. For Poisson's ratio of 0.5, K_0 can be computed with known secant modulus of elasticity E_{50} by

$$K_0 = 1.67 E_{50} \tag{7-38}$$

Skempton (1951) found that E_{50} was approximately equal to 25 to 100 times the unconfined compressive strength of clay. The coefficient K_0 can be expressed in term of q_u as

$$K_0 = 40q_u to 160q_u \qquad (7\text{-}39)$$

K_0 from EQ (7-39) can be used for determining k_∞ in EQ (7-40).

The coefficients of horizontal subgrade reaction can also be estimated using Terzaghi (1955) and Davisson (1970) methods are discussed earlier in this chapter.

7.3.4.1 Back-figured k_h

The coefficient of horizontal subgrade reaction can be back figured from the load-deflection curves of pile lateral load tests. Here the results from Dunnavant and Reese & Welch test will be analyzed. Only the linear portion of load-deflection curves was used. Loads up to 180 and 80 kips were used for Dunnavant and Reese & Welch data, respectively. Pile parameters from Dunnavant and Reese & Welch are shown in Table 7-18 and the back calculated k_h values are 278 and 464 tcf, respectively.

Table 7-18 Pile Parameters from Dunnavant and Reese & Welch Tests

Pier or Pile Parameters	**Dunnavant**	**Reese & Welch**
Diameter (D), ft	6.0	2.5
Penetrated Length (L), ft	37.5	42.0
Effective Stiffness (E_pI_p), lb-in^2	5.65×10^{12}	2.01×10^{11}
Eccentricity (e), ft	0.92	0.25

Ground-line deflection (y_0) results from Dunnavant and Reese & Welch computed from back-figured k_h are shown in Tables 7-19 and 7-20, respectively. Lateral loads versus ground-line deflections are plotted for Dunnavant and Reese & Welch and shown on Figure 7-29 and 7-30, respectively.

Table 7-19 Ground-line Deflections from Back-figured k_h for Dunnavant Data

Lateral Load P (kips)	y_0 by Broms using back-figured k_h (in)	y_0 from Dunnavant Result (in)
0	0.000	0.000
25	0.013	0.022
65	0.034	0.037
85	0.044	0.040
135	0.070	0.066
180	0.093	0.093
250	0.130	0.296
300	0.156	0.589
350	0.182	0.909
400	0.208	1.291
410	0.213	1.634

Table 7-20 Ground-line Deflections from Back-figured k_h for Reese & Welch Data

Lateral Load P (kips)	y_0 by Broms using back-figured k_h (in)	y_0 from Reese & Welch result (in)
0	0.000	0.000
22	0.034	0.026
32	0.049	0.040
43	0.066	0.075
63	0.096	0.183
83	0.127	0.461
100	0.153	0.927

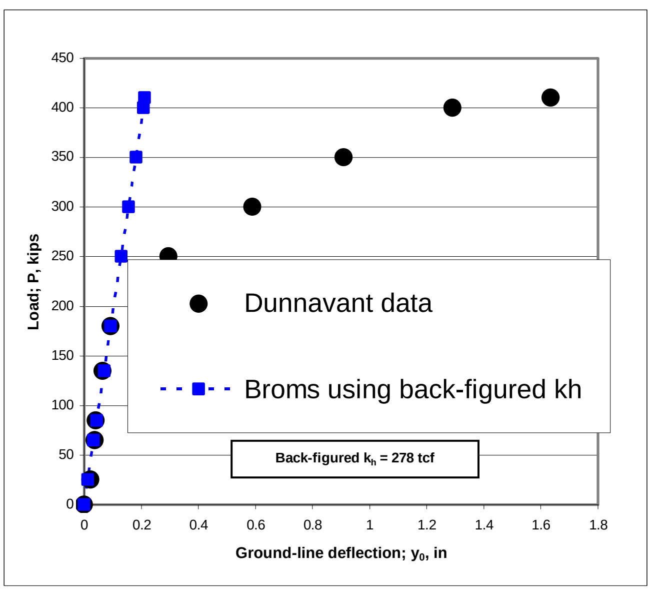


Figure 7-29 Ground-line Deflections from Back-figured k_h for Dunnavant Data

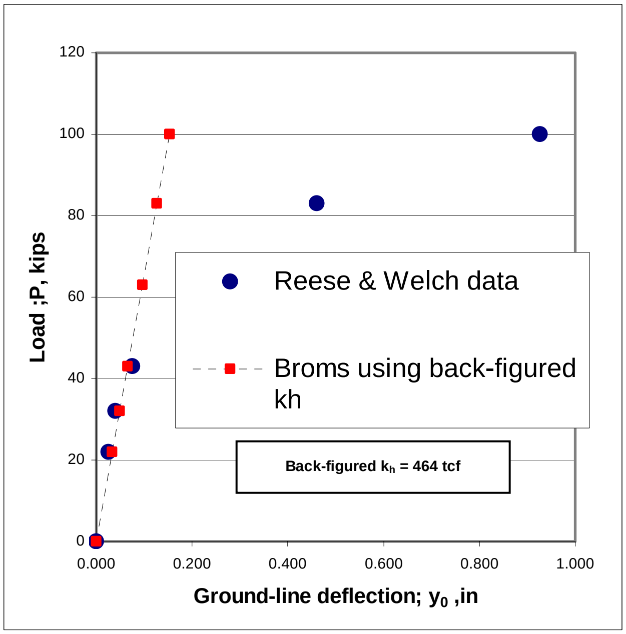


Figure 7-30 Ground-line Deflections from Back-figured k_h for Reese & Welch Data

7.3.4.2 k_h from Terzaghi and Davisson Methods

The methods for evaluating k_h using Terzaghi and Davisson methods were presented earlier in this Chapter. This section summarizes the results from both methods using Dunnavant and Reese & Welch data. For Dunnavant data, soil parameters were divided into two sets based on soil layers used in determining soil parameters. The first set is based on averaging soil properties over 40 ft. The second set uses the averaged soil properties over top 15 ft. This 15-ft layer was recommended in Broms method as "critical depth (L_c)" or the depth wherein any change in k_h will not affect lateral ground-line deflection or the maximum bending moment by more than 10%. L_c is defined as L/Lβ or 1/β. Table 7-21 shows k_h using Terzaghi and Davisson methods on Dunnavant data.

For Reese & Welch data, soil parameters were also divided into two sets. The first and second sets are based on averaging soil properties over 42 and 20 ft, respectively. The 20-ft layer was the value recommended by Reese & Welch as significant depth. The resulting k_h using both Terzaghi and Davisson methods is shown in Table 7-22.

From Table 7-21, k_h from set No.1 is closer to back-figured k_h than ones from set No. 2. k_h using Terzaghi and Davisson are not much different. From Table 7-22, k_h from set No.1 and 2 are not much different. This is because of the undrained shear strength along the depth for Reese & Welch site are almost uniform. Anyway, for both Dunnavant and Reese and Welch data, back-figured k_h are much higher than those from the Terzaghi and Davisson methods.

Table 7-21 k_h Using Terzaghi and Davisson for Dunnavant Data [a]

Set No	Soil property	Terzaghi	Davisson [b]
1	c_u, tsf	1.48	1.48
(average over 40 ft)	k_h, tcf	16.67	16.53
2	c_u, tsf	0.97	0.97
(average over 15 ft)	k_h, tcf	8.33	10.83

[a] k_h from back analysis is 278 tcf

[b] $k_h = 67c_u/D$

Table 7-22 k_h Using Terzaghi and Davisson for Reese & Welch Data [a]

Set No	Soil property	Terzaghi	Davisson [b]
1	c_u, tsf	1.24	1.24
(average over 42 ft)	k_h, tcf	40	33.23
2	c_u, tsf	1.06	1.06
(average over 20 ft)	k_h, tcf	40	28.41

[a] k_h from back analysis is 464 tcf

[b] $k_h = 67c_u/D$

7.3.5 Comparisons Between Broms Method and Test Results

This section compares the results of Broms method using k_h from different methods, and with test results from both Dunnavant and Reese & Welch tests.

7.3.5.1 Dunnavant's Pier Test

Using k_h from Table 7-21, ground-line deflections can be calculated. The results from parameter sets 1 and 2 are shown in Tables 7-23 and 7-24, respectively. Data in Tables 7-23 and 7-24 are plotted in Figures 7-31 and 7-32, respectively. The results show the best method for evaluating k_h is back calculation and set 1 provides better result than set 2.

7.3.5.2 Reese & Welch's Pile Test

This section presents the results using Broms method and k_h from different methods and compare with Reese & Welch test data. From Table 7-22, k_h from set 1 and 2 are about the same. Only results from set 1 are presented. Ground-line deflections from parameter set 1 are shown on Table 7-25. Data from Table 7-25 are plotted on Figure 7-33. From the results, the best method for evaluating k_h is back analysis of k_h. The coefficients of horizontal subgrade reaction from set 1 are better than k_h from set 2.

Table 7-23 Ground-line Deflections by Broms Method Using Parameter Set 1 for Dunnavant Data

Lateral Load, P	y_0 in inches using Broms and k_h by			y_0 from
(kips)	Terzaghi set 1 (in)	Davisson set 1 (in)	back-figured k_h (in)	Dunnavant data (in)
0	0	0	0.000	0
25	0.17	0.17	0.013	0.022
65	0.43	0.43	0.034	0.037
85	0.56	0.57	0.044	0.040
135	0.9	0.9	0.070	0.066
180	1.19	1.2	0.093	0.093
250	1.66	1.67	0.130	0.296
300	1.99	2.01	0.156	0.589
350	2.32	2.34	0.182	0.909
400	2.65	2.68	0.208	1.291
410	2.72	2.74	0.213	1.634

Table 7-24 Ground-line Deflections by Broms Method Using Parameter Set 2 for Dunnavant Data

Lateral Load, P	y_0 in inches using Broms and k_h by			y_0 from
(kips)	Terzaghi set 2 (in)	Davisson set 2 (in)	back-figured k_h (in)	Dunnavant data (in)
0	0.00	0.00	0.000	0.000
25	0.33	0.26	0.013	0.022
65	0.86	0.66	0.034	0.037
85	1.13	0.87	0.044	0.040
135	1.79	1.38	0.070	0.066
180	2.39	1.84	0.093	0.093
250	3.32	2.55	0.130	0.296
300	3.98	3.06	0.156	0.589
350	4.65	3.57	0.182	0.909
400	5.31	4.08	0.208	1.291
410	5.44	4.19	0.213	1.634

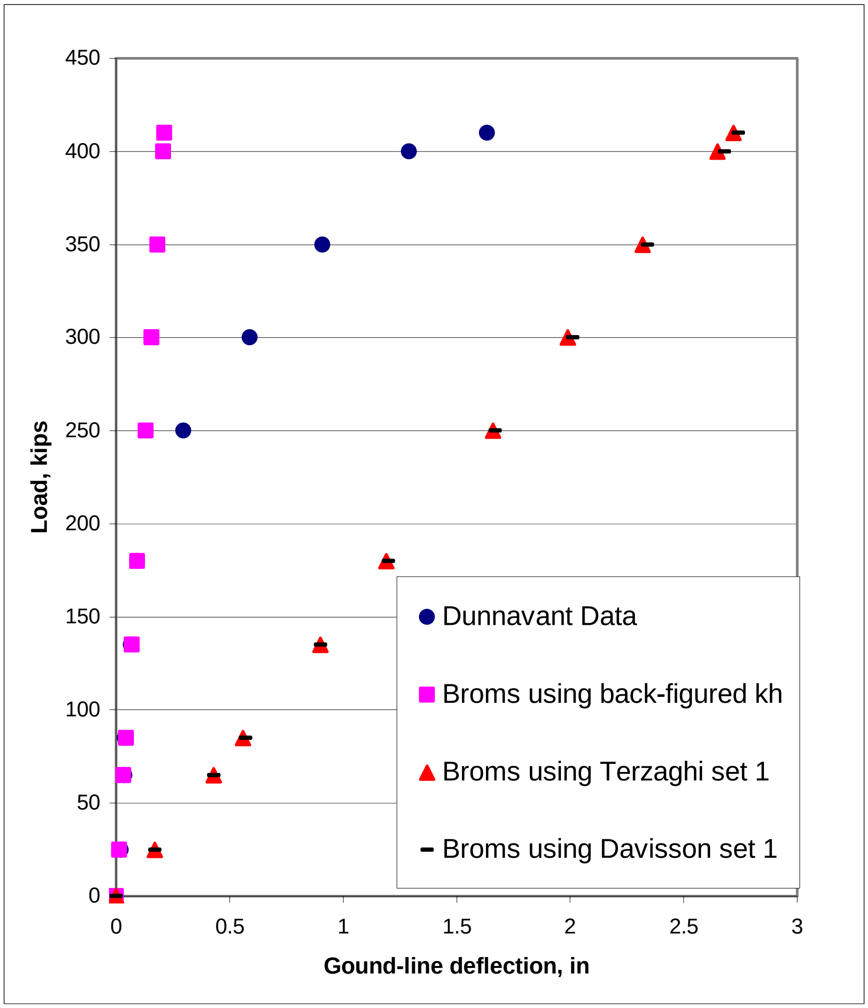


Figure 7-31 Ground-line Deflections by Broms Using Parameter Set 1 for Dunnavant Data

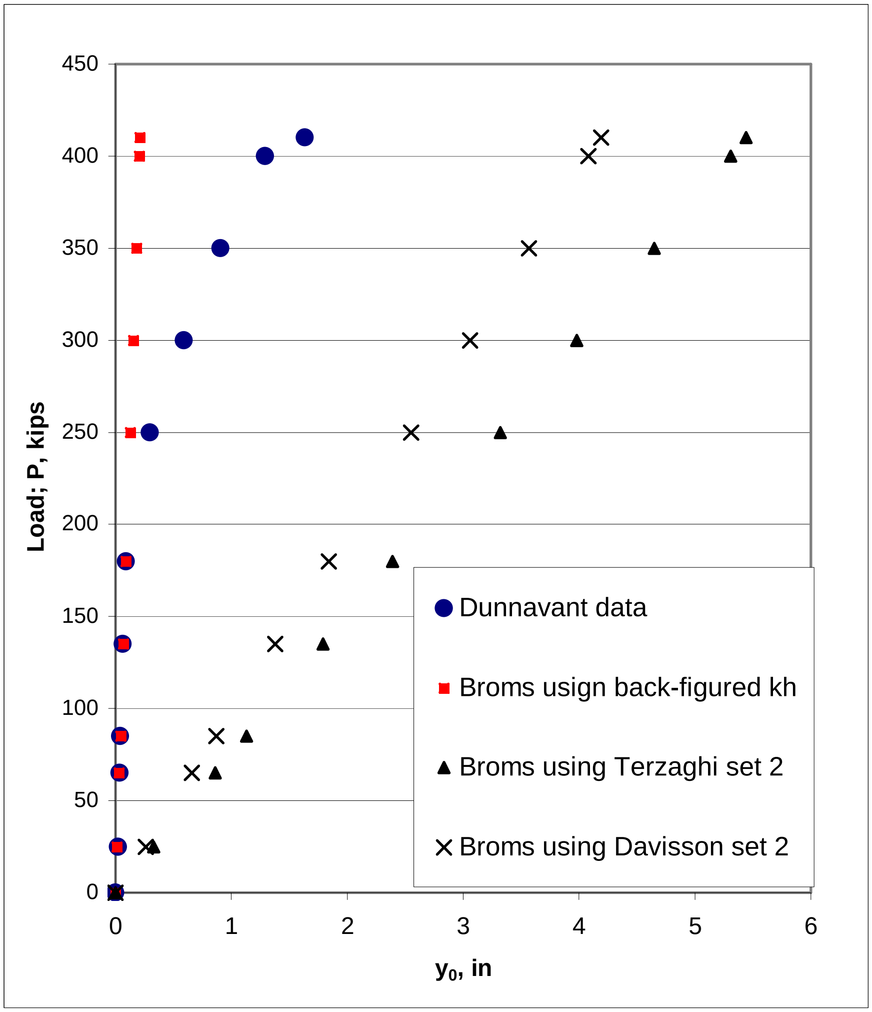


Figure 7-32 Ground-line Deflections by Broms Using Parameter Set 2 for Dunnavant Data

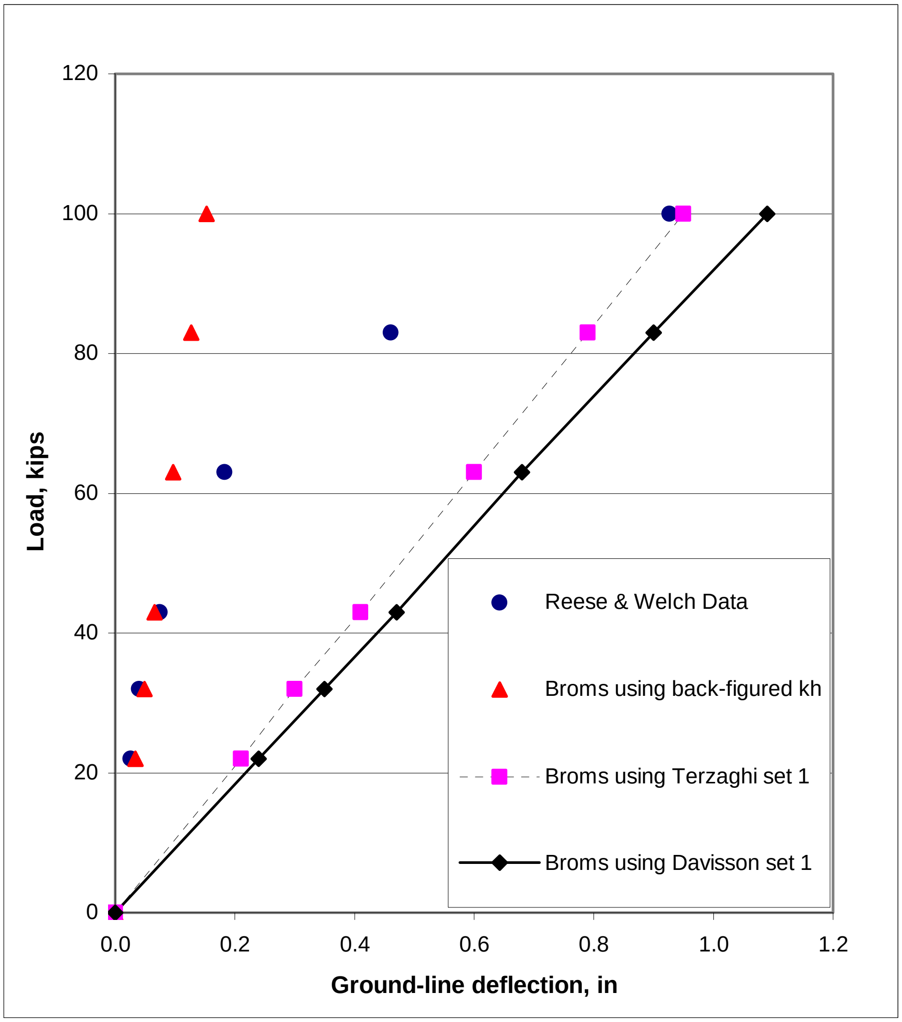


Figure 7-33 Ground-line Deflections by Broms Using Parameter Set 1 for Reese & Welch Data

7.3.6 Summary and Conclusions

Broms proposed the method to determine the ultimate lateral resistances (P_{ult}), lateral ground-line deflections (y_0), and the maximum bending moments (M_{max}) in piles. Broms method is simple but required a number of assumptions. Piles can be divided into two types based on their behavior and lengths. Failure modes of

Table 7-25 Ground-line Deflections by Broms using Parameter Set 1 for Reese & Welch Data

Lateral Load, P	y_0 in inches using Broms and k_h by			Reese & Welch
	Terzaghi set 1	Davisson set 1	back-figured k_h	Data
(kips)	(in)	(in)	(in)	(in)
0	0.000	0.000	0.000	0.000
22	0.210	0.240	0.034	0.026
32	0.300	0.350	0.049	0.040
43	0.410	0.470	0.066	0.075
63	0.600	0.680	0.096	0.183
83	0.790	0.900	0.127	0.461
100	0.950	1.090	0.153	0.927

both types are shown in Figure 7-20. Lateral ground-line deflections were calculated based on subgrade reaction concept. The most important soil parameter is the coefficient of horizontal subgrade reaction. Broms method can only determine the lateral deflection at working load, 1/3 to ½ of $P_{ult;}$ consequently it requires P_{ult} before calculating lateral deflections. The ultimate lateral resistance and maximum bending moment can be determined using hypothesized soil reaction distributions. To determine P_{ult} and M_{max}, the undrained shear strength (c_u) of clay is required.

Broms provided graphical solutions for lateral deflections and ultimate lateral resistances of piles. Equations for calculating these deflections and ultimate loads including maximum bending moments in the piles were also given.
Broms (1964a), Terzaghi (1955), and Davisson (1970) proposed the methods for estimating the coefficient of horizontal subgrade reaction. k_h also can be back calculated from lateral load tests on piles. Of all methods for evaluating k_h, the back calculation of k_h from pile load test is the best method. Both Terzaghi and Davission method underestimate the value of k_h. The back-figured k_h is 17 and 10 times greater than the values from Terzaghi method Dunnavant and Reese & Welch data, respectively.

Broms method gives good estimation of lateral deflections at working loads if the coefficients of horizontal subgrade reaction are estimated correctly. It is incapable of computing the ultimate ground-line lateral deflection. Broms method using Terzaghi's and Davisson's methods of calculating k_h grossly overestimates the ground-line lateral displacement.

7.4. Analyses Using LPILE Program

7.4.1 Introduction

LPILE program is developed to solve the problems of piles or drilled shafts under lateral loads. Piles of different materials and cross sections can be analyzed. It has a capability to analyze piles in different soil types, such as soft clay, saturated stiff clay, unsaturated stiff clay, saturated sand, and dry sand and can calculate deflection, shear, bending moment, and soil response along length of piles.

The constitutive relation of the soil around a pile is represented by p-y curve, where p represents soil reaction pressure per unit length of pile and has the unit of pound per square inch, and y the lateral deflection of pile at the same location. The p-y curves can model the nonlinear behavior of soils and the pile behavior is governed by a beam-column equation.

Soil, pile, boundary conditions, and loading data are required as input data. For soil data, soil properties of different layers, with or without water table, are needed. Pile type, cross section, and material properties are required. Boundary conditions at pile heads are required in terms of loads (shear and moment) and deformations (displacement and rotation of pile head). Two full-scale pile tests used in Section 7.3 were also used to calibrate the LPILE program.

7.4.2 Theoretical Background of LPILE

LPILE uses the beam-column equation (Hetenyi in 1946) and the solution mechanism, p-y method (McClelland and Focht, 1958), to solve the beam-column equation. Much effort has been devoted to improve the p-y method (Lymon C. Reese). In 1986, the first version of LPILE was released to the market. To solve the 4th order differential beam-column equation, finite difference and p-y methods are required. The beam-column equation, p-y curves, and bending characteristics of pile are explained in this section.

7.4.2.1 The Beam-Column Equation

The beam-column equation is the governing equation for a pile subjected to axial and lateral loads. Hetenyi gave the solution to this equation in 1946, which was subsequently enhanced by many researchers. Figure 7-34 shows an element cut from a beam-column bar on elastic foundation subjected to horizontal load and a pair of compressive forces P_x acting in the center of gravity of the end cross-sections of the bar. The equilibrium of moments (ignoring second-order terms) leads to EQ (7-40) as shown below:

$$(M + dM) - M + P_x dy - V_v dx = 0 \tag{7-40}$$

or

$$\frac{dM}{dx} + P_x \frac{dy}{dx} - V_v = 0 \tag{7-41}$$

Differentiating EQ (7-41) with respect to x, we get

$$\frac{d^2M}{dx^2} + P_x \frac{d^2y}{dx^2} - \frac{dV_v}{dx} = 0 \tag{7-42}$$

It is noted the following identities:

$$\frac{d^2M}{dx^2} = EI \frac{d^4y}{dx^4}$$

$$\frac{dV_v}{dx} = p$$

$$p = -E_{py} y$$

where E_{py} is soil modulus or the secant modulus of the p-y curves.

Substituting the above equations into EQ (7-42), the following equation is obtained:

$$EI \frac{d^4y}{dx^4} + P_x \frac{d^2y}{dx^2} + E_{py} y = 0 \tag{7-43}$$

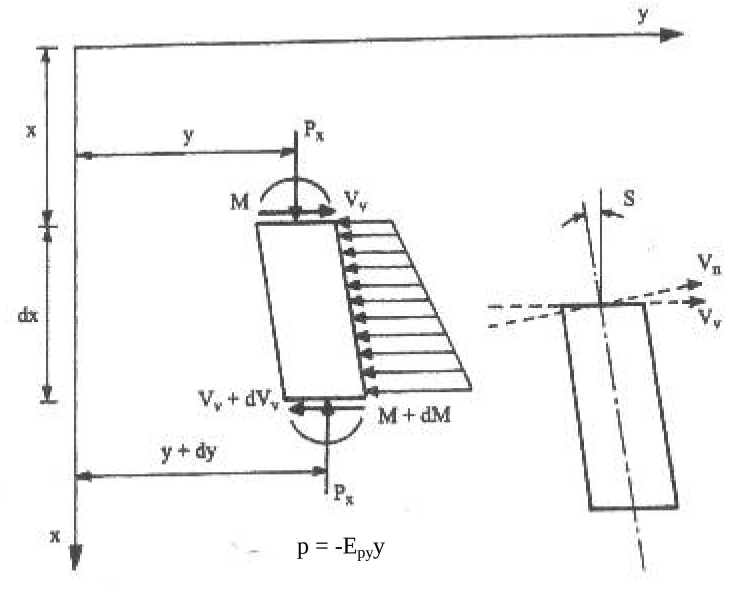


Figure 7-34 Element from Beam-Column Bar (Reese and Van Impe 2001)

The shearing force in the plane normal to the deflection line can be shown in term of V_v and P_x as shown in EQ (7-44):

$$V_n = V_v \cos S - P_x \sin S \tag{7-44}$$

where S is the angle between the element axis and the vertical axis.

For small angle of S, it is assumed that cosS = 1 and sinS = tanS = dy/dx. Then, EQ (7-44) becomes:

$$V_n = V_v - P_x \frac{dy}{dx} \tag{7-45}$$

If the distributed force W per unit length along the upper portion of a pile is applied, the complete form of the equation used in LPILE program is shown in EQ (7-46).

$$EI \frac{d^4 y}{dx^4} + P_x \frac{d^2 y}{dx^2} - p + W = 0 \tag{7-46}$$

where EI is flexural rigidity or bending stiffness of the pile
y is lateral deflection of the pile
x is the depth of soil below ground surface
P_x is axial load on the pile
p is soil reaction or response per unit length of pile
W is distributed load along the length of the pile

In addition to the beam-column equation, other auxiliary expressions for analyzing piles under lateral loads are shown in EQ (7-47) to EQ (7-50).

$$p = EI\frac{d^4y}{dx^4} = -E_{py}y \qquad (7\text{-}47)$$

$$EI\frac{d^3y}{dx^3} + P_x\frac{dy}{dx} = V \qquad (7\text{-}48)$$

$$EI\frac{d^2y}{dx^2} = M \qquad (7\text{-}49)$$

$$\frac{dy}{dx} = S \qquad (7\text{-}50)$$

where E_{py} is soil modulus or the secant modulus of p-y curve
V is shear force in the pile
M is bending moment in the pile
S is slope of the axis of the pile

In deriving the differential equation, the following assumptions are made (LPILE Technical Manual 2000):

1. The pile is straight and has a uniform cross section.
2. The pile has a longitudinal plane of symmetry; loads and reactions lie in that plane.
3. The pile material is homogeneous.
4. The proportional limit of the pile material is not exceeded.
5. The modulus of elasticity of the pile material is the same in tension and compression.
6. Transverse deflections of the pile are small.
7. The pile is not subjected to dynamic loading.
8. Deflections due to shearing stresses are small.

EQ (7-46) can be solved by numerical techniques. LPILE uses finite difference method and with the aid of p-y method.

7.4.2.2 p-y Curves

The p-y method was proposed by McClelland and Kocht in 1958. This method requires p-y curves at different depths of soil deposit. p is defined as the resultant of soil reaction or response at certain depth with a dimension of force per unit length, and y the lateral deflection of pile. The schematic diagram of p and y is shown in Figure 7-35. p-y curves are mechanism to model soil behavior. p-y curves depend on soil type, depths along the pile, and pile stiffness. Typical p-y curve and resulting soil modulus (E_{py}) is shown in Figure 7-36.

The development of p-y curves requires both analytical and experimental procedures. The analytical methods are used for determining soil reaction (p). Lateral deflections are measured from full-scale lateral load tests on piles. Inclinometers are used to determine lateral deflections of piles. Soil reactions or responses are calculated from bending moments along the pile and bending moment is computed from the product of pile curvature and the bending stiffness, obtained directly from pile tests.

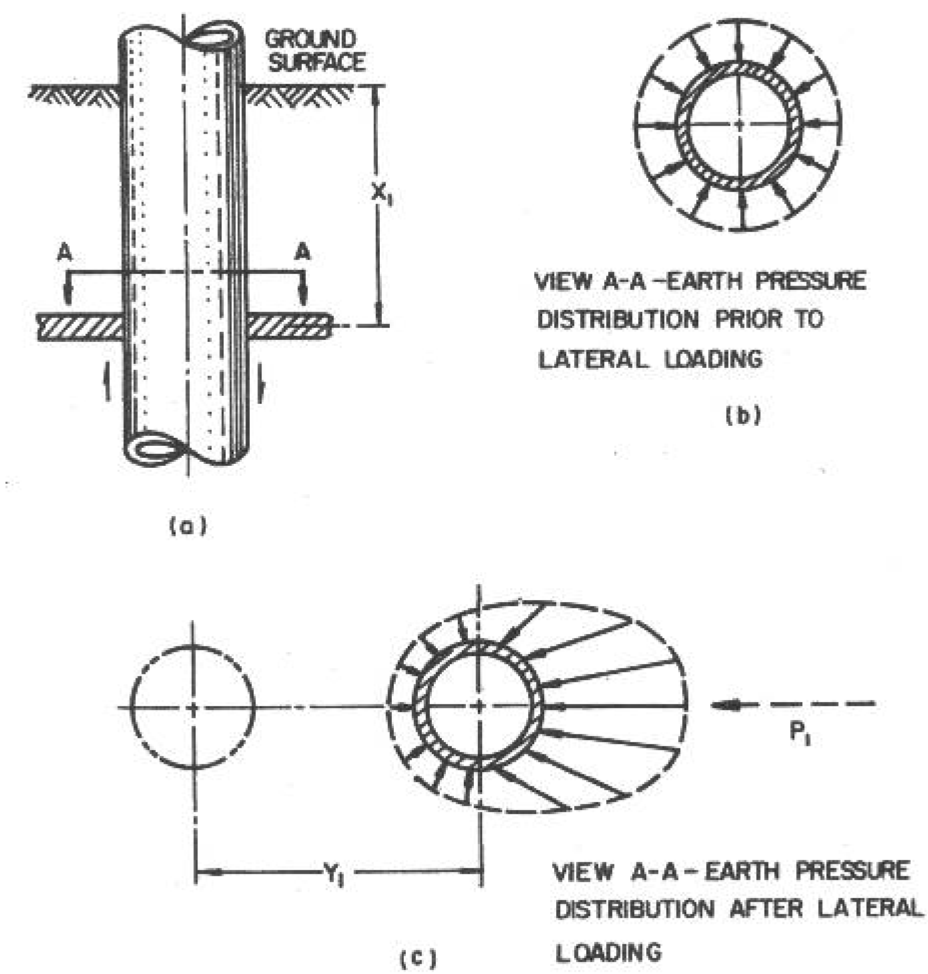


Figure 7-35 Schematic Diagram of p and y (Reese and Allen 1977)

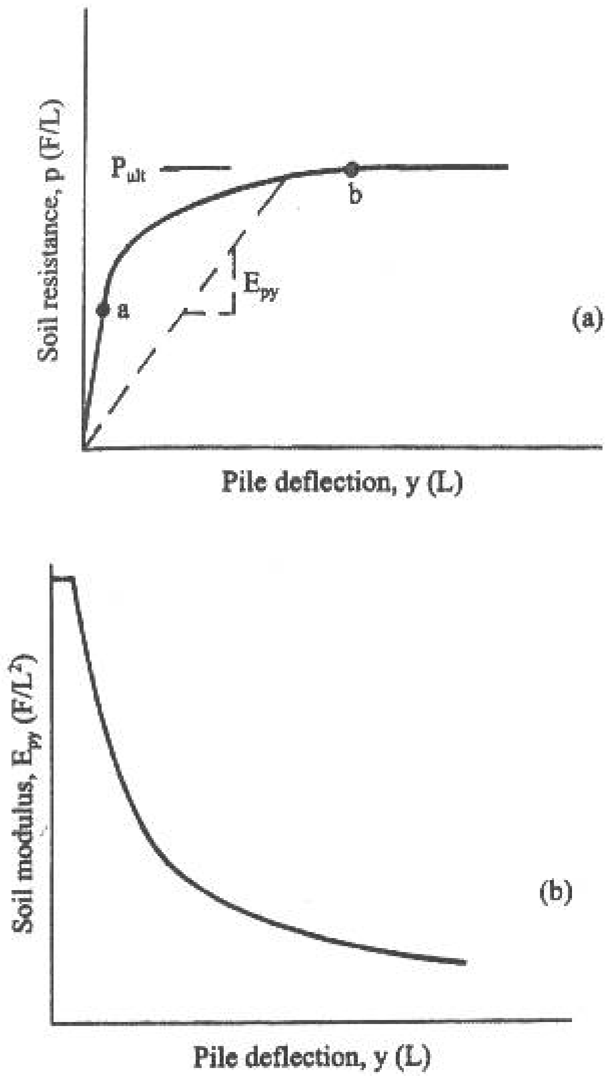


Figure 7-36 (a) Typical p-y Curve and (b) Resulting Soil Modulus (Reese and Impe 2002)

7.4.2.3 Ultimate Bending Moments (M_{ult})

In designing piles or drilled shafts, piles must satisfy both structural and geotechnical requirements. For structural purpose, drilled shaft should be design as a compression member subjected to bending and axial load. The ultimate bending moment and bending stiffness of piles are required. The ultimate bending moment of the section is the maximum bending moment to be resisted by the cross section before failure.

Three possible modes of failure in reinforced concrete piles subjected to combined bending and axial load are: tension failure, balanced failure, and compression failure. The tension failure occurs when the steel on the tension side yields, compression failure occurs at the initial crushing of the concrete at the compression side, and the balanced failure is the simultaneous failure in tension and in compression.

For pile subjected to combined compression and bending, the ultimate bending moment of the pile can be determined using LPILE, or the analytical solution, or Whitney's approximate solution. The Whitney's solution is empirical and approximate. The circular pile is transformed to idealized equivalent rectangular and circular sections for compression and tension failures, respectively, as shown in Figure 7-37. Figure 7-38 shows stresses and forces in the equivalent rectangular section as shown in Figure 7-37 (a).

The circular pile has outside diameter D, the diameter of reinforcement cage D_s, the total area of reinforcement A_{st}, and the gross area A_g.

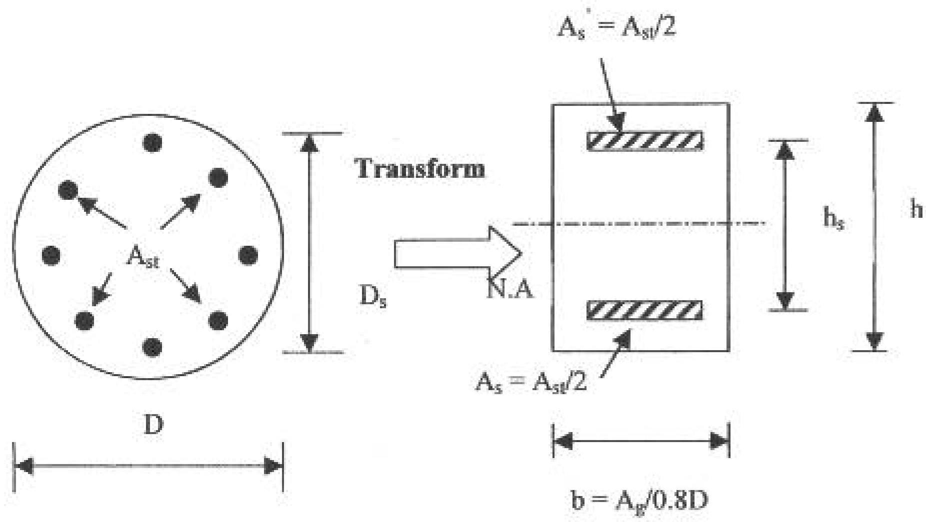
For compression failure, the equivalent rectangular has:

- $h = 0.8D$
- $b = A_g/0.8D$
- $h_s = 2D_s/3$
- $A_s' = A_s = 0.5A_{st}$

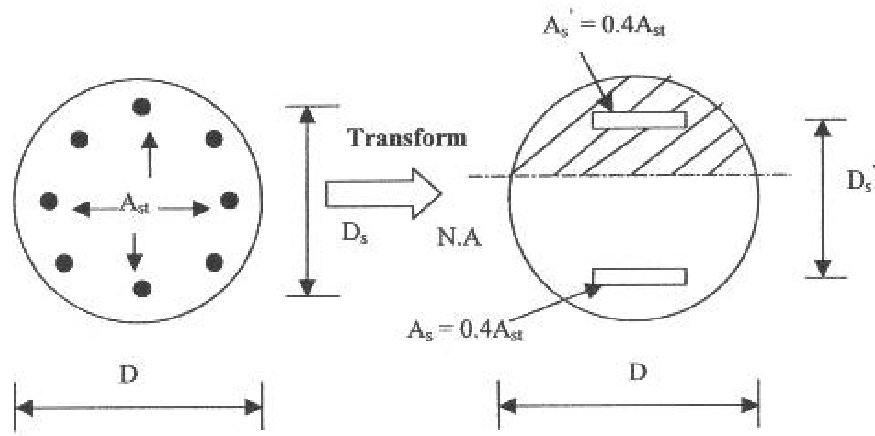
For tension failure, the equivalent circular section has:

- $D_s' = 0.75D_s$
- $A_s' = A_s = 0.4A_{st}$

For balanced failure, circular pile is transformed to the equivalent rectangular as shown in Figure 7-37 (a). The ultimate bending moment for a circular pile subjected to eccentric axial load can be determined using the transformed rectangular section and expressed in EQ (7-51).



(a) Equivalent Rectangular Section for Compression and Balanced Failures



(b) Equivalent Circular Section for Tension Failure

Figure 7-37 Transformed Sections for (a) Compression and Balanced Failures and (b) Tension Failure

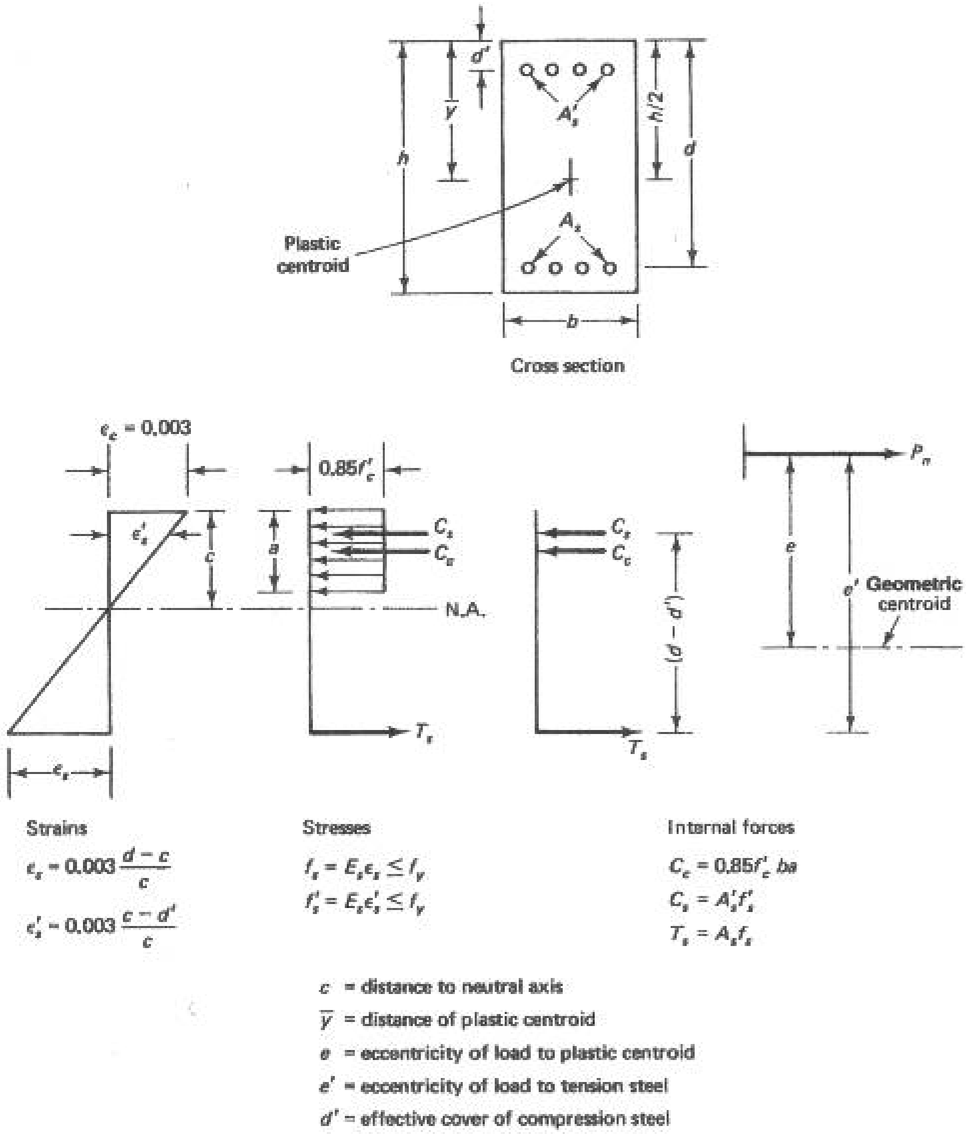


Figure 7-38 Stresses and Forces in Pile Section (Navy 2000)

$$M_{ult} = Q_{nb}e_b = 0.85 f_c^{'} b a_b (\bar{y} - \frac{a_b}{2}) + A_s^{'} f_s^{'} (\bar{y} - d') + A_s f_y (d - \bar{y}) \qquad (7\text{-}51)$$

a_b is the depth of the equivalent stress block and expressed in EQ (7-52).

$$a_b = \beta_1 c_b = \beta_1 \frac{0.0003d}{0.003 + \frac{f_y}{E_s}} \qquad (7\text{-}52)$$

The value of the stress block depth factor β_1 can be determined from:

$$\beta_1 = 0.85 \qquad 0 < f_c^{'} \leq 4000\,psi$$

$$\beta_1 = 0.85 - 0.05\left(\frac{f_c^{'} - 4000}{1000}\right) \qquad 4000\,psi < f_c^{'} \leq 8000\,psi$$

$$\beta_1 = 0.65 \qquad f_c^{'} > 8000\,psi$$

where Q_{nb} is the axial load corresponding to the balanced condition
e_b is the horizontal eccentricity of Qnb from the centroid of pile section
$f_c^{'}$ is the ultimate compressive strength of concrete
b is the width of the pile section
c_b is the distance to neutral axis for the balanced condition
$\bar{y}$ is the distance to the centroid of the section

The ultimate bending moment for pile under combined axial load and bending moment can be designed using interaction diagram. The interaction diagram represents the relationship between axial loads and the ultimate bending moments of pile section as shown in Figure 7-39. The procedure for determining interaction diagrams can be found in reinforced concrete design textbooks. However, LPILE also provide the interaction diagram for a given pile section.

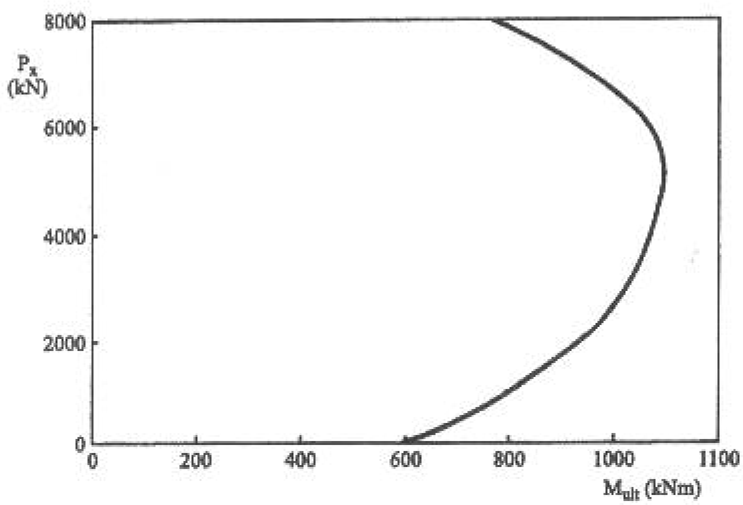


Figure 7-39 Typical Interaction Diagram of Pile (Reese and Impe 2002)

7.4.3 Input Data

LPILE program requires different input data grouped into three categories: pile data, soil data, and boundary conditions and loading data. The detail of how to input these data can be found in LPILE user's manual.

7.4.3.1 Pile Data

In LPILE, the required basic pile data consist of pile dimension, material properties, and cross-sectional details. A general description of the data is shown below:

1) Types of Pile Cross Sections: The following nine different types of cross sections or piles (Figure 7-40) can be analyzed using LPILE:

- Rectangular or square section: this section can be reinforced concrete pile,
- Circular section for reinforced concrete bored pile or drilled shaft,
- Circular with shell but without core: it can be the composite pile constructed using steel pipe and filled with reinforced concrete,
- Circular with shell and core: this section can be the composite section constructed using steel pipe filling with hollow reinforced concrete,
- Circular steel pipe,
- Circular prestressed concrete pile,
- Hollow circular prestressed concrete pile,
- Solid square prestressed pile with chamfered corners,
- Hollow square prestressed pile with chamfered corners.

2) Diameter: this data corresponds to the outside diameter (D) of the pile section.
3) Total Pile Length: this number represents the total length of pile.
4) Pile Length above the Ground surface: this data is required when the pile with the eccentricity load will be analyzed.
5) Moment of Inertia: the effective moment of inertia of pile section is specified.
6) Area: the cross-sectional area of pile section is entered.
7) Modulus of Elasticity: this data corresponds to the Young's modulus of elasticity of the pile section.

If reinforced concrete pile section is analyzed, the material properties and the reinforcement details will be required. The material property data consists of the ultimate compressive strength of concrete (f_c'), the yield strength of reinforced steel (f_y), the modulus of elasticity of reinforced steel (E_{steel}). The rebar number, the number of rebars, concrete coverage are used for defining the reinforcing details.

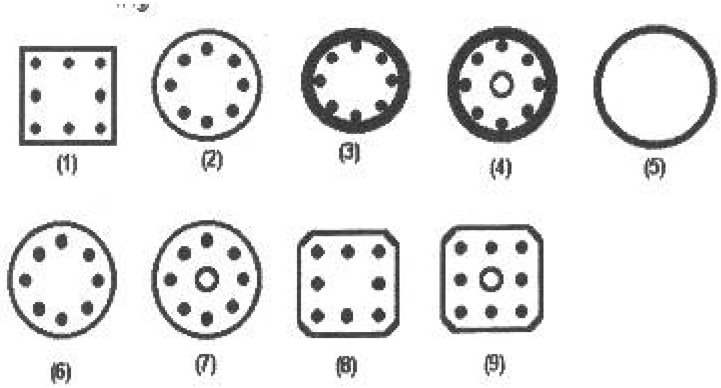


Figure 7-40 Pile Cross Sections

7.4.3.2 Soil Data

LPILE divides a soil deposit into a number of layers with the layer information:

1) Soil Types: There are five general types of soils and one type of rock that users may specify: soft clay, stiff clay with water table at layer top, stiff clay without water table at layer top, sand, silt, and weak rock
2) Unit Weight: Effective unit weight for soil at each depth is needed. LPILE will linearly interpolate values of unit weight for soil between two specified soil depths.
3) Soil-Modulus Parameter k: This is the constant k used in the equation $E_{py} = k_x$. E_{py} is soil modulus and x is depth below the ground surface. The parameter k has a unit of force per cubic length and depends on the type of soil. LPILE user's manual provides the numerical values of k as shown in Table 7-26.
4) Undrained Shear Strength: Undrained shear strength (c_u) values are required for clays and silts at each depth. c_u is not needed for sands.
5) Angle of Internal Friction of Soil: Values of the angle for sands or silts are entered at each depth in degree. This input data is not required for clays and rocks.
6) Soil Strain: Values of strain at 50% of the maximum stress (ε_{50}) are for clays and silts and not for sand. The values are assigned at each soil depth. The values of ε_{50} are given in Table 7-27 and provided by LPILE user's manual.

Other rock properties and parameters are required for piles in weak rocks. They are modulus of elasticity, unconfined compressive strength, rock quality designation, and parameter k_{rm}. The parameter k_{rm} ranges between 0.0005 and 0.00005 (LPILE user's manual 2000).

Table 7-26 Soil-Modulus Parameter **k** for Clays (LPILE User's Manual 2000)

Clay Type	**Average Undrained Shear Strength (c_u), psi**	**Parameter k, pci**
Soft	1.74 – 3.47	30
Medium	3.47 – 6.94	100
Stiff	6.94 – 13.9	500
Very Stiff	13.9 – 27.8	1000
Hard	27.8 – 55.6	2000

Table 7-27 Strain at 50% of The Maximum Stress (ε_{50}) for Clays (LPILE User's Manual 2000)

Clay Type	**Average Undrained Shear Strength (c_u), psi**	**ε_{50}**
Soft	1.74 – 3.47	0.020
Medium	3.47 – 6.94	0.010
Stiff	6.94 – 13.9	0.007
Very Stiff	13.9 – 27.8	0.005
Hard	27.8 – 55.6	0.004

7.4.3.3 Boundary Conditions and Loading Data

The following five boundary conditions can be assigned:

1) Shear and Moment: Values of applied lateral load (P_t) and applied moment (M_t) at the pile head can be specified. This condition indicates that the pile head is free to rotate and move in lateral direction (free-head condition). The positive values of lateral load are defined when applied from left-to-right. The positive values of moments are defined when applied clockwise.

2) Shear and Slope: Applied lateral load (P_t) and the pile head rotation (θ_t) can be specified. The pile head rotation can be assigned in term of the angle between the pile axis and the vertical axis. This angle has a unit of radian. The rotation is positive when the pile head rotates counterclockwise. The lateral load is positive when applied from left-to-right. This boundary condition implies that the pile head is fixed (fixed-head condition). This means the lateral movement of the pile head is allowed but no pile head rotation is permitted.

3) Shear and Rotational Stiffness: Applied lateral load (P_t) and a value of rotational stiffness are assigned. The rotational stiffness at the pile head is defined as a ratio of applied moment (M_t) at the pile head to the pile head rotation (θ_t). The values for rotational stiffness are always positive. In this boundary condition, a fixed-head condition will be specified if a large value of rotational stiffness is assigned. If the user intends to simulate an elastically restrained type of pile-head connection, this boundary condition can be served the user's intension.

4) Displacement and Moment: The lateral pile head displacement (y_t) or deflection and the pile head rotation (θ_t) are specified. The displacement is positive when applied from left-to-right. The positive moment is defined when applied clockwise. This boundary condition represents the free-head condition.

5) Displacement and Pile Head Rotation: Value of the lateral pile head displacement (y_t) and the pile head rotation (θ_t) are assigned in this boundary condition. The positive lateral pile head displacement is considered positive when applied from left-to-right. The pile head rotation is positive when the pile head rotates counterclockwise. This boundary condition implies the free-head condition.

Values of axial loads applied at the pile head may be input after specifying the boundary conditions and corresponding loads. Axial loads are only utilized to account for secondary moments produced when the pile is laterally deflected (P-Δ effects).

7.4.4 Evaluation of LPILE Program using Field Test Results

Two full-scale lateral load tests on piles were used for calibrating LPILE program. These full-scale test results were also used previously to evaluate Broms method. The first test was conducted on 6-foot diameter pier by Dunnavant in 1986. The second test was performed on 2.5-foot diameter pile by Reese and Welch in 1975.

7.4.4.1 Dunnavant's Pier Test

Two soil systems were investigated. The first soil system is uniform soil layer. The second system is two-layered soil. For two-layered soil, the upper and lower layers are 18-ft and 19.5-ft thick, respectively. The ground-line deflections using LPILE for pier in uniform soil and two-layered soil are shown in Table 7-28. The ground-line deflections for pier in both soil systems using LPILE are plotted with Dunnavant's measured results as shown in Figure 7-41.

Table 7-28 Ground-line Deflections (y_0) for Pier in Uniform Soil and Two-layered Soil using LPILE with Dunnavant Data

Lateral load (kips)	y_0 from LPILE for pier in 2-layered soil (in)	y_0 from LPILE for pier in uniform soil (in)	y_0 from Dunnavant (in)
0	0.000	0.000	0
25	0.011	0.008	0.022
65	0.044	0.025	0.037
85	0.065	0.036	0.040
135	0.130	0.069	0.066
180	0.202	0.104	0.093
250	0.344	0.170	0.296
300	0.470	0.226	0.589
350	0.621	0.290	0.909
400	0.803	0.366	1.291
410	0.843	0.383	1.634

From Figure 7-41, ground-line deflections for pier in uniform soil fit the initial curve better than those for pier in two-layered soil. However, two-layered soil model tends to predict the nonlinear behavior of pier better than uniform soil model.

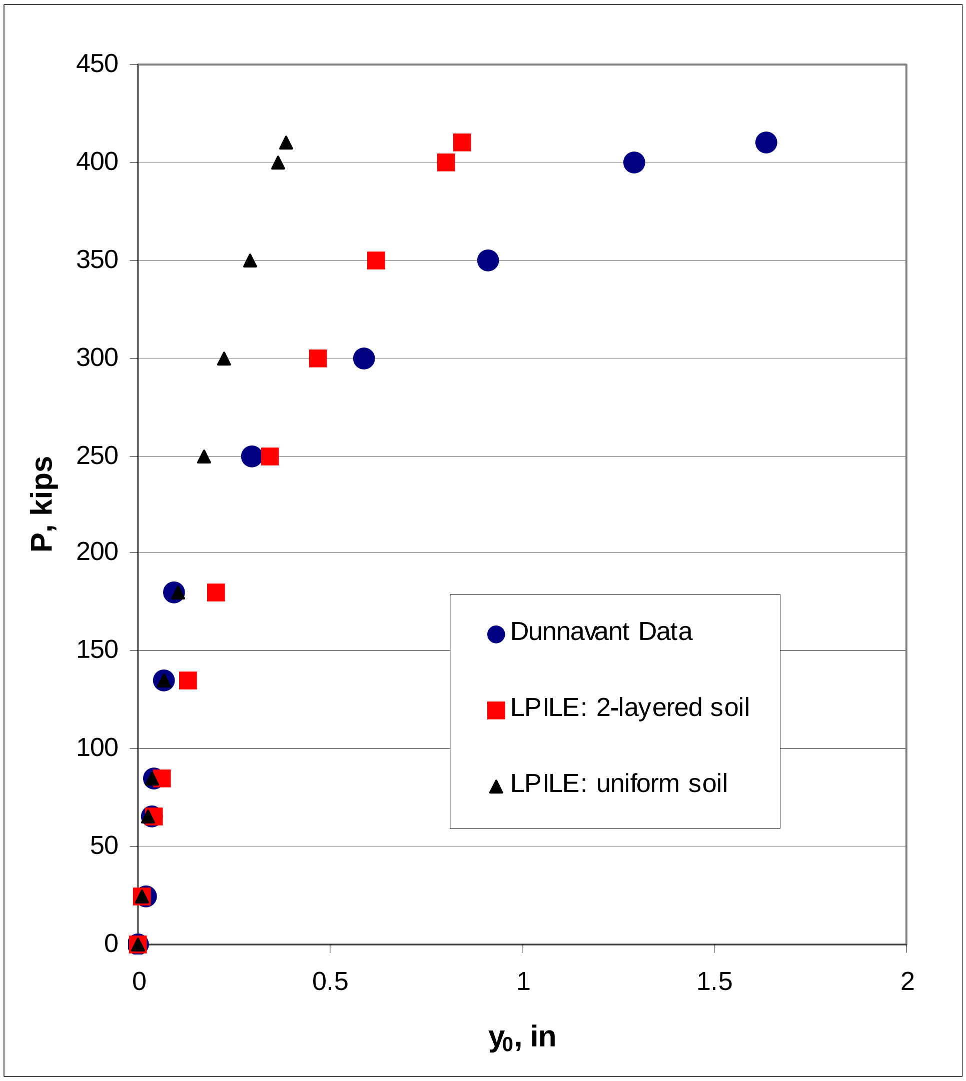


Figure 7-41 Ground-line Deflections using LPILE for Dunnavant's Test Results

7.4.4.2 Reese & Welch's Pile Test

For Reese & Welch case, the two-layered soil was used in the analysis. The upper layer is unsaturated stiff clay with the thickness of 20 feet and the bottom layer is saturated stiff clay with thickness of 22 feet. Ground-line deflections using LPILE for Reese & Welch's pile are shown in Table 7-29. These deflections were plotted along with Reese & Welch's results as shown in Figure 7-42.

As shown in Figure 7-42, the ground-line deflections from LPILE show good agreement with results from Reese & Welch data; however, the analysis underestimated the ultimate ground-line deflection.

Table 7-29 Ground-line Deflections (y_0) for Pile using LPILE with Reese & Welch Data

Lateral Load (kips)	**y_0 from LPILE (in)**	**y_0 from Reese & Welch Data (in)**
0	0.000	0.000
22	0.024	0.020
32	0.054	0.040
43	0.102	0.065
63	0.231	0.187
83	0.415	0.452
100	0.615	0.942

7.4.5 Summary and Conclusions

LPILE program was developed on the basis of the beam-column equation and p-y curves. The beam-column equation can be solved by both analytical and numerical procedures for simple cases. In LPILE, finite difference method was used in solving the beam-column equation. The concept of p-y curves was also employed in analyzing piles under lateral loads. The p-y curve concept was initially developed by McClelland and Focht. p-y curves were used to model the nonlinear soil and pile behavior. The nonlinear behavior of pile under lateral load depends partly on both soil and pile stiffness. The nonlinear behavior of soil is represented by an unconnected series of nonlinear springs. The pile stiffness depends on cross-sectional area and material properties of pile. The pile stiffness indicates the capacity of pile in resisting bending moment. The ultimate bending moment of pile section can be calculated for a given cross section and material property of pile.

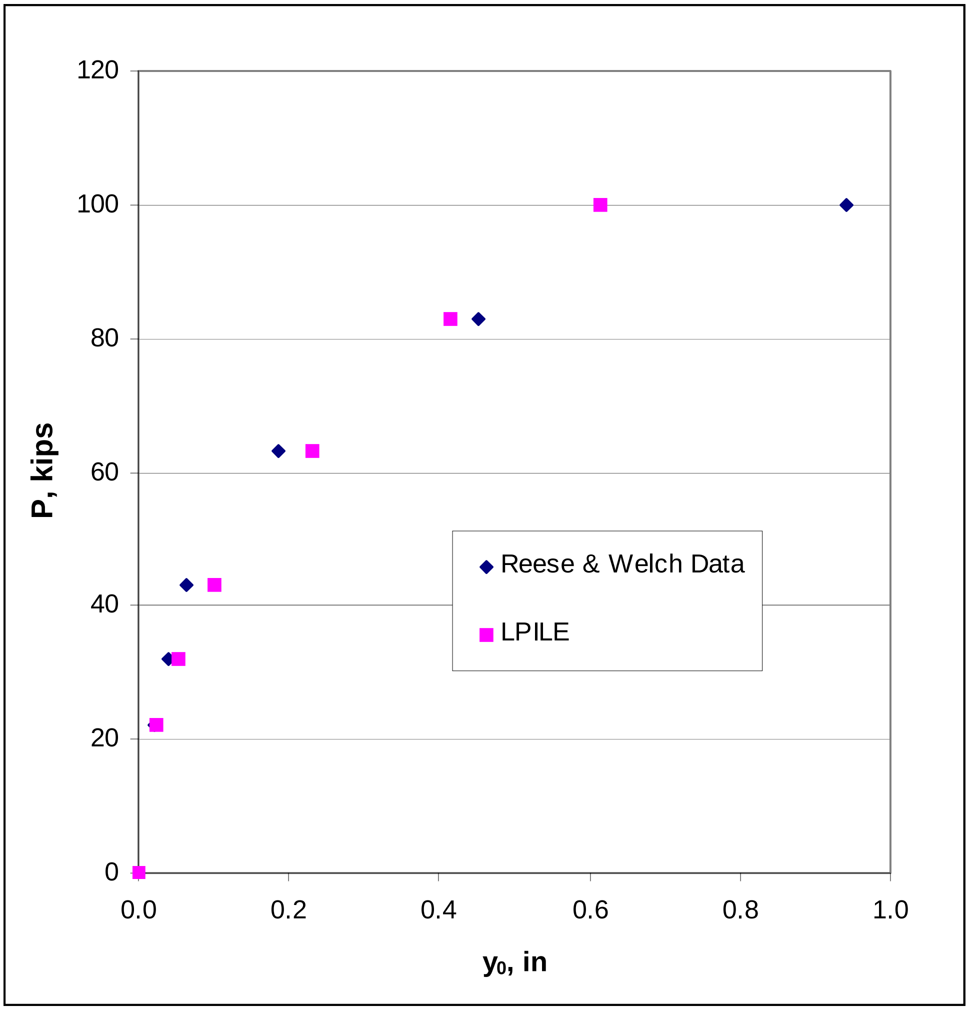


Figure 7-42 Ground-line Deflections using LPILE for Reese & Welch Data

Input data for LPILE can be grouped into three categories i.e. pile data, soil data, and boundary conditions Loading data. LPILE allow users to analyze nine different cross-sectional shapes of piles. Five soil conditions and one rock are available in LPILE program. Five boundary conditions and loading data can be represented the conditions and loads at the top of piles.

Two full-scale lateral load tests on pier (Dunnavant, 1986) and pile (Reese & Welch, 1075) were used for evaluating LPILE performance of piles under lateral loads. The ground-line deflections are in good agreement with the value at working load, and were underestimated at the ultimate load. Soil parameters used in LPILE can be estimated correctly using undrained shear strength (c_u) from unconsolidated undrained (UU) triaxial test.

8. SUMMARY, DISCUSSIONS, CONCLUSIONS, AND RECOMMENDATIONS FOR FURTHER STUDY

Summary

In 1986 a group of five state bridge engineers including **Colorado** met in Denver and drafted a letter to the Subcommittee on Bridges and Structures including their concerns that the AASHTO Specification was falling behind the times. In August of the same year, the NCHRP Project 20-7/31 on "Development of Comprehensive Bridge Specification and Commentary" was initiated. The effort resulted in the current AASHTO LRFD Bridge Design Specifications. One might say, "Colorado is one of the five pioneer states in initiating the study on probabilistic and reliability design specifications for bridges that resulted in the AASHTO LRFD Bridge Design Specifications." CDOT needs to carry this pioneer spirit forward with the complete implementation of the LRFD code. While its implementation for superstructure design is already in place, the implementation for substructures is not so straightforward. It requires the evaluation of the Colorado-specific resistance factors. The task requires systematic laboratory and field testing of strength and deformability of soils and rocks, and field full-scale load tests on some selected foundations. The implementation of the LRFD code in geotechnical investigation design and investigation will need major investment of time and money and it will take a few years depending on the amount of investment. The national survey shows that over 70% of foundation for bridge support is deep foundation with drilled shafts (21%) and driven piles (51%). This statistic does not differ much from the CDOT practice.

Discussions

Extended discussions were held between the members of the study panel and the P.I. of this research project. The essentials are presented as follows:

Questions from the Study Panel and PI's Responses

Question In the current political environment the increase in staff is highly unlikely. Therefore, in addition to the "increase staffing" option, please provide an alternate strategy that does not require increase in staff and state-owned equipment. In other words, please provide an alternative that relies on outsourcing of additional testing needed.

The implementation of the geotechnical LRFD in Colorado requires the Colorado-specific resistance factors, which, in turn, needs the creation of database for all geotechnical material parameters for each geological materials, soils and rocks. This will take:

- Choice of a design method(s).
- Evaluation of material parameters for the chosen design method.
- Establishment and maintenance of the database for all material parameters.

- Determine the probability density function for each design parameter with sufficient statistical sample size.
- Formulation of resistance factor evaluation procedures.
- Calibration of the resistance factors.
- Development of the geotechnical LRFD procedures by adopting the Colorado-specific resistance factors.
- Training of the CDOT personnel in the use of geotechnical LRFD procedure.
- Technology transfer from CDOT to the professional engineering community and local government agencies.
- Adoption of the Colorado-specific geotechnical LRFD.

If the current political environment were strictly financial, then outsourcing does not make sense because it might cost even more to the Colorado tax payers. But if CDOT were to bypass the solution of its geotechnical staffing and field testing equipment problems, then, to meet the FHWA mandate deadline, it has no choice but to outsource all its geotechnical LRFD tasks. In the long run, the CDOT Geotechnical and Soil-Rockfall programs will lose the design capability and become engineering regulatory design evaluation units.

Question: In regards to Implementation Recommendations Nos. 4 and 5: How will the purchase and use of CPT, PMT, GE, and VST equipment move us toward LRFD?

PI Response: So far the geotechnical investigation at CDOT relies almost exclusively on SPT, the roughest method of field investigation. The use of CPT, etc will improve the precision of material parameter evaluation, and, thereby, enhance the reliability of the LRFD geotechnical design process.

Question: In your report you state that we should focus on SPT and DMF, so you need to explain your strategy of expanded use of CPT, PMT, GE, and VST.

PI Response: Since SPT has been the only field investigation tool available to CDOT, all foundation designs are based on blow count, N. This method of foundation design is locally called the Denver Magic Formula (DMF) approach. The kind of general practice is not observed anywhere else in the country as shown in the national survey of state DOT's. So CDOT is so entrenched in the DMF practice, it is very difficult and unrealistic to stop the practice and switch to a more rational practice immediately. Thus, it is advised to continue the practice and transition the design procedure to the eventual geotechnical LRFD procedures. The development of the new LRFD procedure requires the use of the field test devices more precise than SPT, which include PMT, CPT, VST and GE, etc devices for more reliable assessment of the material parameters and subsurface stratification. The acquisition of field test equipment is dictated by the funding availability, and can be pursue in a financially sound time frame. When the equipment funding is not available, the job will obviously have to be outsourced to different geotechnical companies or universities.

Question: You recommend that CDOT should be moving toward a rational method; what rational method do you recommend? Why?

PI Response: Two types of design methods are available: empirical and rational. The former is based mainly on the observation and the latter is mechanistic. There are times where mechanistic and empirical methods can coexist to serve as mutual check. Using the empirical method without serious verification can be either un- or over-conservative. In the design of deep foundation, the DMF method is empirical based on experience and observation, while the Terzaghi's bearing capacity, for instance, is a rational mechanistic approach. In the former the allowable bearing capacity is given as N/2 ksf (kips per square foot) and in the latter the bearing capacity of a deep foundation includes two terms: side friction and end bearing, both are functions of angle of internal friction and cohesion of soils, fundamental Mohr-Coulomb strength parameters of soils and rocks that can be evaluated in laboratory. The blowcount in the Denver Magic Formula is an all-inclusive parameter and not a fundamental engineering parameter. Therefore the design approach based on the fundamental engineering parameters is more rational. It is good to move toward a rational design approach to enable the communication among all design engineers, in/out of state.

Question: Absent a load test at each location, how will soil parameters and boring logs be used to establish failure probability curves?

PI Response: Two separate issues are raised: probability density function (pdf), and nominal strength of soils or rocks. The nominal strength of a foundation is evaluated based on a selected design method (or equation) for computing the nominal strength, which involves the material parameters and a boring log aids in the selection of material parameters by identifying the material type. The former is obtained in a laboratory/ field test and the latter revealed in a subsurface exploration program. If an engineer is equipped with the boring log and soil parameters for the specific location then the design is straight forward by using the parameters provided and the equation selected.

The stratification, soil/rock types, and associated material parameters vary in a random fashion and probabilistic approaches are needed to rational describe their variation, i.e. a probability density function (pdf) is needed for each parameter. Probabilistic approaches also require calibration against field load tests, which are usually expensive and can be performed at all sites. Thus, the database of field load test results is extremely valuable in serving as the calibrator for the rational approach and also the LRFD procedures when a sufficient statistical sample size is achieved. Thus, in the Colorado geotechnical LRFD (G-LRFD) effort, more field load tests should be performed to enhance the database to be used in the calibration of G-LRFD procedures.

Additionally even a theoretical formulation involves some uncertainty because of the simplifying assumptions to make the problem mathematically manageable. Like in the deep foundation design, there are several formulations for bearing capacity computation with Meyerhoff's solution giving the highest capacity, while the Terzaghi's, the lowest or most conservative and the load tests can provide precision check. A load test must be physically sound and conducted accurately to be useful in serving as a calibrator. For instance, questions for the Osterberg Cell load test (O-C load test) include: Can it provide an accurate top–down load capacity? How can it serve reliable settlement assessment for deep foundations? The questions need to be investigated before the extensive use of the O-C load test for the top-down behavior prediction.

Question: Your recommendation for a database is probably a good one, but we need to know what kind of analysis will be performed at some future date to make sure we are collecting and storing the right data.

PI Response: You are exactly right. This is why I recommend the formation of the G-LRFD Committee to delineate the approach to the problems of identification of analysis method, parameters needed, and the method of testing, etc.

Question: Also database maintenance is costly and we would need to demonstrate that we would be getting something of sufficient value to justify the investment. Is our current standard drilling practice sufficient for a viable database or do we need to perform additional tests at each hole?

PI Response: CDOT uses nearly exclusively SPT in its drilling practice. The blowcount reflects approximate strength through it correlation with the field load tests, and provides samples, which are, in principle, good only for examination of soil types and index property tests. Because of the shortage of geotechnical staff, CDOT has been unable to utilize its excellent laboratory strength test facility to aid in the evaluation of strength parameters needed in the foundation design. In sum, the current CDOT geotechnical investigation practice is held to the minimum and needs improvement for the effective implementation of the geotechnical LRFD design procedures. The geotechnical investigation should be extended from SPT to cover laboratory and field testing, whenever possible. In the CDOT quest for the implementation of G-LRFD procedures the establishment and maintenance of material property databases are optional, but are needed.

It is recommended to establish and maintain the databases on a pool-fund basis among different states because all states face the same problem and the cost will reduce as more states participate in the G-LRFD effort. Besides, a financially-sound implementation time frame can be established to avoid unbearable financial burden.

Question: You recommended that we use CPT, PMT, GE, and VST tests. When should we perform these tests? On what type of soils and rocks? How often?

PI Response: The following tests are routinely required: SPT, CPT (soils only) and PMT. VST is recommended for the investigation of soft and medium clay deposits and GE in a big project. In a big project, a series of GE tests will delineate subsurface stratification on a large-scale basis. This can then be followed by SPT and CPT tests for general exploration at selected locations. Pros and cons of SPT and CPT are outlined as follow:

	SPT	CPT
Pros	1. Easy to conduct. 2. Equipment is readily available 3. While somewhat disturbed, it does provide samples, at least, for examination and maybe for testing	1. Easy to conduct. 2. Excellent delineation of strength and stratification
Cons	1. Results do not reflect the static strength of soil or rock 2. Detail delineation of layers is not possible.	1. No sample for properties. examination or testing.

Thus, it is best to perform SPT and CPT in parallel side by side to provide subsurface information for the selection of other laboratory/field tests needed in further investigations to provide accurate material parameters for the critical subsoils and rocks.

PMT (pressure meter test) and **VST** (vane shear test) are usually requested by a design engineer to provide more reliable properties and/or strength data at the locations considered critical to the design project. PMT can be performed on both rock and soils to provide stress-strain-strength parameters, coefficient of earth pressure at rest, Young's modulus, compressibility, etc. VST provides the undrained shear strength of clayey soils. Situations do occur when VST is performed to provide the in-situ strength of clay, particularly in a stability investigation. Not all tests are required at all sites.

Question: There are several foundation design models available. Which one do you recommend we use? The answer may not only depend on accuracy but also on the cost and ability to collect and test rock samples to derive the parameters for the model.

PI Response: The answer to this question involves the following factors: effectiveness and reliability of each design model, effort needed to provide the design material parameters, designer’s preference, and cost, etc. A design engineer will have to select one or more theories for comparison. In the deep foundation design, among many methods, one may pick DMF, Terzaghi method, and/or Meyerhoff method, etc. Each method comes with some deficiencies. For instance, DMF lacks the backing of a serious study for its effectiveness for Colorado soils and rocks and the blowcount reflects only the bulk or approximate strength of soils or rocks and not the basic engineering property. Because of the tremendous experience in using DMF approach, it might be good for CDOT to critically examine its effectiveness during its transition to the general G-LRFD procedures. Additionally the Terzaghi and/or Meyerhoff methods ought to be investigated for their effectiveness in Colorado geological environment. It is a very difficult problem to answer and the choice is very personal. To avoid the answer being tainted by the PI’s personal opinion, it is recommended to allow the G-LRFD Committee to jointly make the unbiased selection of the design method.

Question: How will soil parameters and boring logs be used to establish failure probability curves? Will the data tell us how variable our rocks are?

PI Response: The boring logs are used to identify the stratification of subsoils and rocks. Each soil and rock can be tested for its design material parameters. When sufficient statistical sample size is achieved for a parameter, its probability density function (pdf) can be formulated. The pdf's of all design parameters are entered into a selected design method to compute the pdf for the nominal strength. The nominal strength probability density curve can then be compared to the load pdf to determined the failure probability as outlined in Chapter 2 on "Fundamental of LRFD." The data will show the variability of soils and rocks as demonstrated by the histograms, pdf curves, and the mean, standard deviation and coefficient of variation of a parameter.

Question: Your recommendation of an LRFD foundation committee is a good one, but they need a starting point. Visualize CDOT in five year after we have been collecting data on every hole we drill. What analyses should we perform on the database? Toward what objectives?

PI Response: The establishment of geotechnical design criteria, procedures, and theory should be objective and not biased by the decision of a single individual. Many methods (or theories) are available for the deep foundation design. To avoid a biased choice, the decision and selection should be made by the collective wisdom of the G-LRFD Committee through serious open deliberation. Thus, this report has not attempted to make the choice for the State of Colorado. Instead, the committee will assume the task. This report serves as a good starting point for the G-LRFD Committee to begin its work and deliberation.

If serious about the implementation of the G-LRFD design procedures, CDOT must invest, whether in-house or outsourcing. The tentative tasks for the committee as proposed in this study, when it convenes include the following:

- Decide the core committee with the membership from FHWA representative, CDOT Bridge, Geotechnical Program and Research and UCD representative.
- Convene the core committee meeting to choose the LRFD Committee membership from structural and geotechnical consulting engineering industries and other collaborative government agencies.
- Convene the first big committee meeting to establish agenda and progress schedule, and assign responsibility, etc.
- Select a foundation type (s) for first implementation of the Geotechnical LRFD (G-LRFD) procedures, design theories (or models, methods), design parameters, types of tests and test procedures, and the Database Center (DC).
- I recommend the deep foundation as the first foundation type for the G-LRFD implementation. Once the process progresses to an extent that the committee is ready to move onto another foundation type or geotechnical problem, then the implementation effort for the new foundation type can be gradually phased in.

- DC will be responsible for updating and analyses of database, formulation of probability density function (pdf), evaluation of nominal strength, and evaluation of resistance factors based on the reliability theory. The database center will report to the LRFD Committee on a quarterly basis of the progress made.
- DC will inform the LRFD Committee to call a meeting(s) to discuss the status of data collection and analysis and timing for the initiation of the effort for the evaluation of resistance factors and eventual implementation of G-LRFD procedure for the selected foundation type(s).
- LRFD Committee will initiate another geotechnical problem(s) for G-LRFD implementation and process continue until the full implementation is achieved.
- CDOT can initiate a pool fund study among willing participating states, like the neighboring states and a separate committee can be established to carry out the pool fund study effort. DC can expand its effort to cover the data collection and analysis for the data from the out-of-state participants.

CDOT is still leading the G-LRFD implementation effort among the neighboring states, and can continue to take a leadership role if it initiates the effort soon before others begin to catch up.

Question: Just because there is a specific testing device available, doesn't mean we buy one and use it on every borehole sample. We need to decide if it is cost-effective. The data collection plan must be based on some vision of how we are going to use it.

PI Response: You are right again. The purchase choice of field test equipment must be based on its applicability to the Colorado geological materials, the CDOT fiscal strength, and the chosen design method. Not all equipment is good for projects, soils and rocks. Funding dictates any project activity and its extent. Under the Colorado budget constraint as pointed out by the CDOT Research, the purchase choice must be prioritized and the actual purchase is priority based. The equipment most appropriate to the Colorado geological and budget environments is first purchased and so on. The sole objective of performing different types of tests is to enhance the data reliability, and better and more reliable design of a foundation structure. Structural LRFD is much more straight-forward, because of much less material uncertainty than the material uncertainty in the geotechnical design. Thus, to implement geotechnical LRFD, we need to have a much better understanding of material properties by establishing the statistical database of material parameters obtained from more reliable laboratory and/or field tests for all relevant materials. One needs to bear in mind that the implementation of G-LRFD is not be free. It is a major financial and budget commitment to the CDOT Administration and the Colorado tax payers. While out-sourcing might be an alternative, somehow the CDOT needs to keep some in-house expertise to be cost effective and to lead the engineering industry in Colorado.

There are three different levels of G-LRFD procedure calibration, from simplest to most sophisticated, as follows:

- Level I is referred to as a mean value first order and second moment (MVFOSM) procedure and the value of the reliability so evaluated is least precise.
- Level II is advanced first order and second moment (AFOSM) method. It provides more precise reliability.
- Level III, a fully probabilistic and most complex method, requires the knowledge of pdf of each random material parameter and the correlation between all parameters. It is not used in the general calibration of the G-LRFD procedure.

CDOT's G-LRFD implementation can begins at the Level I for now and advance to a more advanced procedure in the future. This would allow the G-LRFD implementation effort to proceed and progress.

Conclusions

1. The Geotechnical and Soil-Rockfall Programs are severely understaffed. The national statistics give an average geotechnical staff size of 32.27; the average for the Colorado neighboring states is 34.3. This manpower deficiency can hamper the delivery of quality service of the programs and staff morale. Most importantly, the safety of the traveling public could be affected in an event of sinkholes, bridge foundation collapses, rockfalls and landslides, etc.
2. The CDOT foundation design is nearly exclusively based on the result of the standard penetration test (SPT) and so called Denver Magic Formula. The survey indicated that no other states in the Union practice the DMF-type design procedures. Instead, they all based their foundation designs on a rational design approach more in line with what AASHTO requires.
3. The survey shows that CDOT has laboratory testing facility comparable in quality to those of other states. However, it is severely deficient in the field investigation equipment for geophysical exploration (GE), cone penetration test (CPT), Menard pressure meter test (PMT), and vane shear test (VST), which are routinely used in other states in geotechnical investigation.
4. CDOT does not have any staff responsible for foundation design. This results in significant operational chaos in the bridge substructure design. In a reasonable operation, the geotechnical programs provide subsurface investigation results and design soil parameters, and the bridge staff provides design requirements to foundation engineers, who are responsible for the foundation design. It might be a good idea to create a foundation engineer position.
5. CDOT does not have any in-house capability for full-scale foundation load tests. This can be economical when a large number of tests are recommended for both design calibration and evaluation of resistance factors for Colorado-specific soils and rocks.

Recommendation for further study

1. Establish a G-LRFD Committee, a think-tank group, to formulate the strategy and plan for an effective G-LRFD implementation. The committee membership

should include some responsible persons from the CDOT Central Administration, Bridge Branch, Geotechnical Program, and Soil-RockFall Program; one representative from each of the structural and geotechnical consulting industries; and an academic representative knowledgeable in LRFD, probability and reliability.

2. Hire additional geotechnical engineering and investigation staff required for the laboratory and field testing, quality service delivery, and implementation of G-LRFD procedures.
3. Create a foundation engineer position(s) responsible for all foundation designs. The person(s) should have the overlapping capability of structural, geotechnical and foundation design to effectively communicate with both geotechnical and structural staff.
4. Upgrade the field investigation facility by purchasing PMT, CPT, GE, and VST equipment whenever the budget permits.
5. Activate the excellent laboratory testing apparatuses and place them in a production line to generate urgently needed laboratory data of soils and rocks for the formulation of resistance factors for Colorado-specific geological materials.
6. Select the deep foundation as the first foundation type for G-LRFD implementation and formulate detailed procedures for the evaluation of resistance factors for all Colorado-specific geological materials. The practice can be extended and gradually phased in to other areas of geotechnical investigations and designs.
7. Continue to practice and calibrate DMF, while G-LRFD implementation effort progresses. The possibility of enhancing DMF for Colorado geotechnical design and investigation should be explored, while the implementation of a G-LRFD design procedure more in line with the AASHTO procedures progresses.
8. Develop in-house full-scale load test capability to check the design recommendation and calibrate the G-LRFD recommendation.

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APPENDIX A SURVEY RESULTS

Table A.1 LRFD Survey Questionnaires

1) How many people are there in your respective geotechnical and structural engineering divisions? Are the divisions combined? Is the size of your staff sufficient?
2) What kind of projects does your division do?
   - ☐ Bridge
   - ☐ Slope Stability
   - ☐ Road/Highway Construction
   - ☐ Other
3) What are the predominant soil and rock types in your state?
   - ☐ Clay
   - ☐ Sand
   - ☐ Clay shale
   - ☐ Sandstone
   - ☐ Igneous/Metamorphic rock
   - ☐ Loess
4) What codes do you currently use in designing geotechnical structures?
   - ☐ ASD
   - ☐ LFD
   - ☐ LRFD
5) What codes do you currently use in designing substructures or superstructures?
   - ☐ ASD
   - ☐ LFD
   - ☐ LRFD
6) Do all projects require subsurface investigation?
   - ☐ Yes, what type? ____
   - ☐ No
7) What lab tests are normally required for a geotechnical project?
   - ☐ Swell/Consolidation
   - ☐ Unconfined Compression Test
   - ☐ Triaxial
   - ☐ Direct or Simple Shear
   - ☐ Standard and Modified Proctor
   - ☐ Atterberg Limits
   - ☐ Grain Size Analysis
   - ☐ Other ____

   Is your facility sufficient? If not why?
8) What types of insitu tests are required for geotechnical projects?
   - ☐ Conventional static load test. If yes, what type?
   - ☐ Osterberg Load Cell Test
   - ☐ Statnamic Load Test
   - ☐ Dynamic Load Test; Type of test _________________________________
   - ☐ Others ____

   Is your facility sufficient? If not why?
9) Do you contract laboratory or insitu testing to local firms?
   - ☐ Yes _____________% contracted?
   - ☐ No
10) What are your standard procedures for subsurface investigation?

11) What procedures do you follow to decide the types of tests required for a project?
12) Do you contract any design projects to local firms?
    - ☐ Yes ____________% contracted?
    - ☐ No
13) What is the supporting relationship between the structural and geotechnical divisions?
14) Do you use any "magic formulas" used in geotechnical design? For instance, in Colorado, the end-bearing pressure for drilled piers is usually estimated as N/2 ksf.
15) Has your DOT begun to implement or perform research on LRFD design concept?
    - ☐ Yes
    - ☐ No
    - ☐ Structural
    - ☐ Geotechnical
16) If you have not implemented LRFD in geotechnical design, what is your implementation schedule?
17) If you have not implemented LRFD in structural design, what is your implementation schedule?
18) In your opinion, is the LRFD method a better method for geotechnical design? If yes, why? If no, why?
19) If LRFD is used for geotechnical design, are your load and resistance factors state specific? If not, do you have plans to generate state specific factors?
20) If LRFD is used in geotechnical design at your DOT, then how long has it been in place?
21) If LRFD is used in geotechnical design, do you use it for all projects?
22) The main objective of our research is to develop a geotechnical database required for evaluation of resistance factors for eventual implementation of LRFD in geotechnical engineering. What do you think would help us in achieving this research objective?
23) If you have implemented LRFD in geotechnical design, have you experienced resistance from the geotechnical consulting industry? If yes, how did you address the issue?

Table A.2 Survey results on status of implementation of geotechnical LRFD

Primary ID	State	Department	Respondent	Title
1	North Dakota	Geotechnical Section	Jon Ketterling	Geotechnical Engineer
2	Oregon	Geo/Hydro	Tim Potter	HQ Unit Manager
3	Georgia	Bridge Design	Paul V. Liles Jr.	State Bridge Engineer
4	Ohio	Geotechnical Engineering	Steven Sommers	Geotechnical Program Coordinator
5	Oklahoma	Bridge Division	Greg Allen	Assistant Bridge Engineer
6	Utah	Geotechnical Division	Darin Sjoblom	Geotechnical Engineer
7	Indiana	Geotechnical Section	Steve Morris	Geotechnical Engineering Group Leader
8	South Carolina	Bridge Design, Geotech Section	Tim Adams for Jeff Sizemore	Former Bridge Design Geotechnical Engineer
9	Pennsylvania	Transportation	Scott Christie	Chief Bridge Engineer
10	Texas	Bridge Division	Mark McClelland	Geotechnical Branch Manager
11	Minnesota	Bridge Office	David Dahlberg	Bridge Design Unit Leader
12	Wyoming	Geology Program	Mark Falk	Project Geologist
13	Connecticut	Soils and Foundations	Leo Fontaine	Trans. Principal Engineer
14	Missouri	Geotechnical Section/Bridge Design Unit	Kevin W McLain & Bryan Harnagel	Geotechnial Engineer, Structural Special Assignments Engr.
15	Louisiana	Bridge Design & Pvmt & Geotech Design	Kim Martindale, Kelly Kemp	Pvmt & Geotech Des. Engr Adminstrator, Bridge Design
16	South Dakota	Geotechnical Engineering Activity	Kevin Griese	Geotechnical Engineer
17	Mississippi	Materials Division, Bridge Division	James Williams, Mitch Carr	Materials, Bridge Engineer
18	Arkansas	Bridge, Geotechnical	Stewart Linz, Johnathan Annable	Staff Bridge Design Engr, Staff Geotech Engr
19	Tennessee	Structures Division	Edward P Wasserman	Civil Engineering Director Structures Div.
20	Alabama	Transportation	Fred Conway	Bridge Engineer
21	Maryland	Geotechnical Explorations Division	Mark Wolcott	Chief
22	Nevada	Transportation	Parviz Noori	Assistant Materials Engineer
23	Hawaii	Transportation	Herbert Chu	Engineer V
24	Rhode Island	Bridge Design	Anthony Palombo	Sr. Civil Engineer
25	N.Y.	Transportation	D. Walton	Assoc. Soils Engineer
26	N.J.	Transportation	Harry A. Capers	Manager, Structural Engineering
27	Kansas	Bridge/Geology	Loren Risch	Bridge Design Engineer
28	VT	Transportation	Chris Benda	Soils and Foundation Engineer

# people in geotech	Geotech div independent?	Size of geotech staff enough?	Engineers	Geologists	Field Techs	Lab Techs	Drill Crew	Drafters
6	yes	no	3	0	0	0	3	0
75	yes	no	24	19	0	0	6	4
36	yes	no	6	1	2	2	12	1
21	yes	no	8	1	0	3	7	2
0	yes	no	3	0	3	5	8	0
10	yes	no	4	1	1	1	3	0
21	yes	no	9	4	0	3	4	1
19	no	no	8	1	0	6	4	0
20	no	yes	20	0	0	0	0	0
125	no	n/a	5	0	0	0	0	0
25	yes	yes	6	3	0	1	12	0
26	yes	yes	1	14	0	2	8	1
20	yes	yes	7	0	0	0	2	0
47	no	no	6	2	2	2	21	0
32	no	not answered	16	0	1	8	6	1
12	yes	yes	4	1	0	2	5	0
20	yes	yes	6	2	0	2	10	0
16	yes	yes	3	1	1	1	10	0
28	yes	no	6	7	0	3	10	2
60	yes	no	6	1	18	2	16	1
42	yes	no	11	0	4	7	20	0
20	yes	no	7	0	0	2	3	0
8	yes	no	3	0	2	3	0	0
2	no	no	2	0	0	0	0	0
138	yes	yes	55	12	21	17	30	3
22	no	no	8	4	0	0	11	0
41	yes	no	6	12	4	4	10	3
12	yes	no	3	1	0	2	6	0

Soil and Rock Types	Major geotech problems in the state
clay	settlement, low bearing capacity, frost heave, landslides
clay, sand, clay shale, sandstone, igneous/metamorphic rock, peat	earthquake, ladslides, rockfall
clay, sand, clay shale, sandstone, igneous/metamorphic rock	landslides, karst topography, river scour
clay, clay shale, sandstone, a-4, a-6, glacial till	soft, weak pavement subgrades- landslide- mines (surface and underground)- embankment support/settlement- water bearing sandstone- weatherable shales, mudstones and siltstones
clay, sand, clay shale, sandstone, igneous/metamorphic rock, loess, limestone	blank
clay, sand, clay shale, sandstone, igneous/metamorphic rock, lake bottom sediments	settlement, potentially liquefiable soils, landslide, collapsible/expansive soils, rockfall, highly organic soils
clay, sand, clay shale, limestone, glacial till	soft and wet subgrade, peat and marl deposits, landslides
clay, sand, clay shale, igneous/metamorphic rock, limestone, marl	setlement and liquefaction of soft/loose sedimentary deposits in the coastal plan, adequate pile penetration in limestone areas due to lateral loads, unsupported length and scour, rock socket penetrat. for drilled shafts, length of socket vs. rock quality
clay, sand, clay shale, sandstone, limestone	limestone, solution cavities
clay, sand, clay shale, limestone	variable materials, construction errors
clay, sand , clay shale, sandstone, igneous/metamorphic rock, loess, peat deposits, till, carbonate deposits	peat deposits (settlement/slope stability), high groundwater
Clay, Clay Shale, igneous/metamorphic rock	swelling soils, slope stability, collapsing soils, rockfall
west & east uplands are glacial tills. central valley has deposits of sand & silt over varved clay deposits. Bedrock types are sedimentary (arkose) & igneous (basalt) rock in the central valley, & metamorphic (schist and gneiss) in the uplands	The upper half to 2/3's of the central valley have significant deposits of compressible soils (varved clays). The New Haven harbor area has significant deposits of organic silt.
clay, sand, clay shale, loess, limestone, dolomite	landslides, settlement at bridge ends, karst topography
clay, sand, loess	soft soils, lack of bearing layers, variability of profiles, setup issues
Clay shale, sandstone, igneous/metamorphic rock, glacial till	pierre shale for highway subgrades
clay, sand, loess	high volume change clays, soft soils, landslides
clay, sand, sandstone, shale, limestone, dolomite	slope failues in clay and weathered shale slopes, subgrade stability for grass root (lowfill) widening projects
clay, sand, sandstone, limestone	sinkholes, colluvium, high plasticity clays, water sensitive silts, poor quality control during construction
clay, sand, clay shale, sandstone, igneous/metamorphic rock, loess	blank
clya, sand, clay shale, sandstone, igneous/metamorphic rock	sink holes, landslide, slough
clay, sand, igneous/metamorphic rock, gravel and cobbles	boulders/cobbles, kaliche, expansive soils, hydrocollapsible soils
clay, sand, and basalt	rockfall, shallow embankment slipage
Clay shale, sandstone, igneous/metamorphic rock, glacial deposits with sand, silt, gravel, cobble,s and boulders.	no major geotechnical problems, challenging site conditions: high ground water tablesoft organic soils posing negative skin friction for piles and settlement for embankments. Liquefaction may be a problem.
clay, sand , clay shale, sandstone, igneous/metamorphic rock, and glacial soils.	galcial lake beds with deepsilt and clay deposits with organics, highly variable rock elevations within a short distance, boulders obstructing deep foundation construction, staged construction required for structure replacement.
clay, sand, clay shale, snadstone,Ingeous/Metamorphic rock, limestone.	settlement, rock slope (cut), large driven piles.
clay, sand, clay shale, sandstone, loess	swelling soils, collapsing solils, very soft alluvial soils.
Soil and Rock Types	slope instability (slides)

Tests for subsurface exploration	Lab tests req'd for geotech project
spt, shelby tubes	some swell/consol, unconfined, triaxial, some direct or simple shear, standard and modified proctor, atterberg limits, grain size, specific gravity, some permeability
cpt, spt, trenching, very little geophysical	swell/consol, unconfined, triaxial, direct or simple shear, standard and modified proctor, atterberg limits, grain size analysis, specific gravity, permeability
spt, geophysical, trenching	swell/consol, unconfined compression, triaxial, std. And modified proctor, atterberg limits, grain sz analysis, specific gravity, permeability
cpt (very little), spt, geophysical (very little)	swell/consol, unconfined, triaxial, direct/simple shear, std and modified proctor, atterberg limitis, grain sz analysis, permeability
cpt, spt, geophysical	unconfined, triaxial, direct/simple shear, standard/modified proctor, atterberg limitis, grain size analysis, specific gravityl, resilient modulus
CPT, SPT, Geotphysical, Trenching/test pits	swell/consol, unconfined, triaxial, direct or simple shear, standard and modified proctor, atterberg limits, grain size analysis, specific gravity, permeability
SPT	Unconfined, atterberg, grain size analysis, moisture content
CPT, SPT, PDA	consolidation, triaxial, atterberg, grain size, specific gravity
cpt, spt	unconfined, triaxial, atterberg, grain size analysis, specific gravity, permeability
Texas CPT	swell/consol, unconfined, triaxial, direct/simple shear, std and modified proctor, atterberg, grain size, specific grav, permeability
cpt, spt, geophysical (occasionally)	swell/consol, unconfined, triaxial, direct/simple shear, specific gravity
cpt, spt, geophysical, trenching/test pits	swell/consol, unconfined, some triaxial, direct/simple shear, atterberg limits, grain size analysis, some specific gravity and permeability
CPT-rarely, SPT-predominantly, Geophysical-occasionally, Trenching/test pits-occasionally	Atterberg Limits,Grain Size Analysis
cpt, spt, geophysical	swell/consol, unconfined, direct or simple shear, atterberg,
cpt, spt	swell/consol, unconfined, triax, direct/simple shear, standard and modifed proctor, atterberg, grain size
california retractable plug tube sampler - 2 7/8" pk rod driven with a 490 lb hammer	swell/consol unconfined compression test, direct or simple shear, atterberg limits, grain size analysis
spt, shelby tube	swell/consolidation, unconfined compression, standard and modified proctor, atterberg, grain size, specific gravity
spt	unconfined, triaxial, atterberg, grain size
spt, geophysical, shelby	swell/consol, unconfined, direct/simple shear, standard and modified proctor, atterberg, grain size, specific gravity
cpt, spt	swell/consol, unconfined, triaxial, direct or simple shear, standard and modified proctor, atterberg, grain size, specific gravity
cpt, spt, geophysical, trenching/test pits, DMT	swell/consolidation, unconfined compression test, triaxial, direct or simple shear, standard and modified proctor, atterberg, grain size analysis, specific gravity, permeability, resistivity
spt	swell/consolidation, direct or simple shear, atterberg limits, grain size analysis, specific gravity
spt, trenching and test pits	swell/consolidatioin, unconfined compression, direct shear, compaction (standard or modified Proctor), Atterberg's limits, grain size analysis, and specific gravity test.
spt, cpt, field vane shear tests, trenching/test pits and occassional geophysical exploration. undisturbed sampling, ground water observatoin, lab testing, liquefaction analysis when needed.	consolidation, unconfined compression, triaxial, Proctor compaction, Atterberg's limit tests, grain size,
SPT, geophysical exploration and undisturbed sampling.	consolidation, unconfined compression, triaxial, Proctor compaction, Atterberg's limit tests, grain size, specific gravity, pH and resistivity on backfill for MSE walls.
spt, cpt, pmt, and undisturbed sampling.	Consolidation, unconfined compression, triaxial, Atterberg's limits, Grainsize analysis, specific gravity.
spt, cpt, geophysical and continuous sampling	swell/consolidation, unconfined compression, triaxial, direct/simple shear, Proctor (standard and modified), Atterbderg's limits, grain size analysis, specific gravity, and permeability
survey, boring, inclinometers, monitoring wells, modeling	spt, vane shear, Menard pressure meter test.

Lab facility sufficient?	Contract lab/insitu test? (%)	Types of insitu tests?
yes	0	not answered
yes	10	osterberg load cell, dnamic pile load test (PDA, CAPWAP)
yes	30	osterberg load cell test, vertical pile load test
yes - AMRL certification	80	osterberg load cell, vertical pile load, lateral pile load, dynamic pile load (ASTM D4945, CAPWAP), Menard Pressuremeter
yes		plate load test, dynamic plie load, menard pressuremeter, dilatometer, texas cone penetrometer
	0	
not enough technicians	40	pile driving analyzer
yes	35	osterberg load cell, vertical pile load, lateral pile load test, dynamic pile load, PDA, Statnamic, dilatometer
	50	blank
n/a	50	osterberg load cell, vertical pile load test
yes	50	osterberg, vertical , lateral, dynamic (PDA), menard
yes	0	none
We do not employ a lab technician. We have a fully equipped lab, but have found it more efficient to outsource these activities.	70	Osterberg Load Cell Test, Vertical Pile Load Test, Lateral Pile Load Test, Dynamic Pile Load Test
no - don't have own load fram for unconfined, need new resilient modulus software and hardware	7.5	Osterberg load cell, dynamic pile load test, statnamic, SCPTU
not answered	85	osterberg, vertical pile, lateral pile, dynamic pile - PDA in house
yes	0	none
yes	10	osterberg, vertical pile, dynamic pile (PDA)
yes	0	menard pressuremeter (special projects only)
no - no triaxial equipment, too few staff, staff not formally trained in testing	60	blank
could use more help	50	osterberg load celll, vertical pile load, dynamic pile load
no, need modern computerized testing equipment	25	plate load, osterberg load cell, lateral pile, dilatometer
yes	0	dynamic pile load - PDA
no	90	plate load test, Osterberg's load test, vertical pile load test, dynamic pile load test (PDA)
no. depend heavily on contracted laboratory testing service	100	Osterberg's load test, vertical pile load test, lateral pile load test, dynamic pile driving analyzer (PDA)
yes, worry about losing it for financial reasons.	0	Osterberg's laod tests, vertical pile load test, dynamic pile driving (PDA), parallel seismic fro determining unknown pile length.
no. The lab personnel may not receive adequate training.	95	Osterberg's load test, vertical pile load test, lateral pile load test, dynamic pile driving analyzer (PDA), Menard pressure meter test.
yes	30	Osterberg's load test, vertical pile load test, Menard pressure meter test, vane shear test, Iowa MHS
unconfined compression, triaxial, Atterberg's limits, grrain size analysis, specific gravity.	0	Osterberg's load test, vertical pile load test, PDA, Menard pressure meter test.

Types of retaining walls?	Cantiliver (%)	Gravity (%)	MSE w/ block facing (%)	MSE w/ full ht panel (%)	Miscellaneous (%)
cantilever, MSE w/ block facing, MSE w/ full height panel	10	0	80	10	
cantilever, gravity, mse w/block facing, mse w/ full height panel	3	5	40	50	0
cantilever, gravity, mse w/ block facing, mse w/ full height panel	20	15	0	5	5
cantilever, gravity, mse w/ block facing, mse w/ full height panel, miscellaneous	15	1	70	1	13
cantiliver, gravity, mse w/ block facing, mse w/ full height panel	0	0	0	0	0
	0	0	0	0	0
cantilever, MSE w/ 5' x 6' R.C. panels	30	0	0	70	0
cantilever (temporary shoring sheet pile), MSE (temporary for staged construction), misc. (gabion),	80	0	0	15	5
cantilever, MSE with full height panel	50	0	0	50	0
MSE with block facing, MSE w/ full height panel, misc	0	0	5	75	20
cantilever, gravity, MSE w/ block facing, MSE w/ full ht, miscellaneous	85	2	10	2	1
cantilever, gravity, MSE w/ block facing	10	30	60	0	0
Cantilever, gravity (incl. Semi-gravity cantilever, prefab modular), MSE w/ block facing, MSE w/ full height panel (including precase partial height panels), miscellaneous	5	35	20	30	10
cantilever, MSE w/ block facing, gabions, cribwalls	9	0	90	0	1
cantilever, MSE w/ block facing	10		90		
MSE w/ block facing, MSE w/ full height panel			20	80	
cantilever, gravity, MSE w/ block facing	90	5	5		
cantilever, MSE w/ block facing	25		75		
cantilever, gravity, MSE w/ block facing, MSE w/ full height panel, bins, cribs and tie-back walls	30	20	10	30	10
cantilever, gravity, MSE w/ block facing					
cantilever, gravity, MSE w/ block facing, MSE w/ full height panel, misc.	65	5	5	20	5
cantiliver, MSE w/ block facing	35		65		
cantilever, gravity, MSE with block facings	0				
cantilever, gravity, MSE w/ block facings, MSE w/ full height rigid panel, and misc.	50	15	10	15	10
cantilever, MSE w/ block facings, soldier pile and laggings, precast walls	15	0	20	0	65
cantilever (including 35% T walls), MSE walls w/ full-height rigid panels, sheet pile/soldier piles, tieback, soil nail	50			35	15 (sheet piles,etc)
cantilever, MSE w/ block facings, MSE w/ full-height rigid panels	10	1	3	4	0
cantilever, gravity, MSE w/ block facing, MSE w/ full-height rigid panels, Misc.	90	2	1	3	4

Types of foundations for bridges	Drilled shaft (%)	Driven pile (%)	Helical Pier (%)	Shallow (%)
driven pile		100		
0	5	75	0	20
drilled shaft, driven pile, shallow	20	70	0	10
drilled shaft, driven pile, shallow	15	80	0	5
drilled shaft, driven pile	95	5	0	0
	0	0	0	0
drilled shaft, driven pile, shallow	5	80	0	15
drilled shaft, driven pile, pile footings - depends on the location - Coastal plains, shafts - 20%, driven pile - 80%. Piedmont, shafts - 80%, piles 15%, pile footings 5%	0	0	0	0
drilled shaft, driven pile, shallow	5	45	0	50
drilled shaft, driven pile	70	30	0	0
drilled shaft, driven pile, shallow	5	85	0	10
drilled shaft, driven pile, shallow	15	30	0	55
drilled shaft, driven pile, shallow, micropiles (5%)	10	25	0	65
drilled shaft, driven pile, shallow	10	65	0	25
drilled shaft, driven pile	20	80		
drilled shaft, driven pile	20	80		
drilled shaft, driven pile	5	95		
drilled shaft, driven pile, shallow	2	75		23
drilled shaft, driven pile, shallow	10	50		40
drilled shaft, driven pile, shallow	35	40		25
drilled shaft, driven pile, shallow	10	50		40
drlled shaft, driven pile, shallow	50	10		40
drilled shaft, driven piles, and shallow foundation	60	20		20
drilled shaft, driven piles, shallow	20	50	0	30
drilled shaft, driven piles, shallow, micropiles	10	40	0	40
drilled shaft, driven piles, shallow	20	40	0	40
drilled shaft, driven piles, and shallow foundation	45	45	0	10
drilled shaft, driven piles, and shallow foundation	10	40	0	50

Slope stabilization	Micropile (%)	Geosynthetics (%)	Grouting (%)	Scaling (%)	Benching (%)	Anchors (%)	Retaining Wall (%)
geosynthetics, benching, retaining wall		70			20		10
scaling, anchors, retaining wa	0	0	0	80	0	15	5
geosyn, bench., anchors, ret w	0	5	0	0	5	10	80
geosynth, benching, retain wall	0	15	0	0	60	0	25
geosynthetics, benching, retaining wall	0	0	0	0	0	0	0
	0	0	0	0	0	0	0
retaining wall	0	0	0	0	0	0	100
geosynthetics, retaining wall	0	80	0	0	0	0	20
geosynthetics, benching, retaining wall	0	0	0	0	0	0	0
most often by removal and replacement, occasionally w. geosynthetics, anchors, walls	0	0	0	0	0	0	0
geosynthetic, benching, other - retaining wall, flatten slopes, lower grade, remove and replace, drainage - 75% is these methods	0	10	0	0	10	0	5
micropile, scaling, anchors, dewatering (20%), reconstruction of fill and toe berms (50%)	10	0	0	10	0	10	0
not a significant problem	0	0	0	0	0	0	0
geosynthetics, benching, removal and replacement	0	5	0	0	95	0	
geosynthetics, benching, anchors, retaining wall							
geosynthetics, scaling							
geosynthetics, benching		20			80		
excavate and replace w/ upgraded material		0					
benching, retaining wall		0			70		30
geosynthetics, benching, retaining wall		0					
micropiles, geosynthetics, benching, retaining wall	2	38			20		40
scaling, retaining wall				50			50
geosynthetics, scaling, benching, retaining walls							
geosynthetics, benching, anchor, retaining walls	0	20	0	0	20	30	30
geosynthetics, anchors, retaining walls, soil nail walls	0	40	0	0	0	23	35
geosynthetics,retaining walls, 2:1 slope without treatment.	0	2	0	0	0	0	18
benchingl	0	0	0	0	100	0	0
geosynthetics, benching, retaining wall	0	8	0	0	90	0	2

Codes used in geotech design	Codes used in structural design	Contract design projects? (%)	Geotech and structural coordinate?	Magic formulas?
ASD	LFD	20	yes	no
ASD, LRFD	n/a	20	yes	no
ASD, LFD	ASD, LFD	40	yes	no
ASD	LFD	90	no	no
LRFD	LRFD	yes	no	empirical charts for drilled shaft design foundation using TX CPT data
n/a	n/a			
LFD	LRFD	90	yes	no
ASD, LRFD	ASD, LFD		yes	case studies used to apply typical load transfer values that would be misleading with our local experience (SPT97 not effect. For design. Piles in marl)
LRFD	LRFD	80	yes	no
ASD and TxDOT procedures	ASD, LRFD	varies	same division	specific design procedures developed by TxDOT
ASD	LFD, LRFD	15-40	yes	no
LFD	LFD	10	yes	blank
ASD, LRFD (for highway bridge des. By consultants)	ASD, LRFD (highway bridge)	75		
ASD	LFD		yes	drilled shafts in shale - ultimate EB = qu, drilled shafts in rock ultimate EB = 0.15qu
ASD	ASD, LFD, LRFD	10-15	yes - road and bridge	Su/p ratios, limiting adhesion values in drilled shafts & piles, experience based setup predictions
ASD	LFD, LRFD	10	yes	no
ASD	ASD, LFD	<10 for geotechnical services	yes	no
LFD	LFD	15	yes	no
ASD	LFD, LRFD (except fdn design)	60	yes	no
ASD, LFD	ASD	20	yes	9000 psi end bearing on piles; min drill shaft socket = 2D in soil, 1D in rock; drill shaft FS = 3; pile load test to 2P
ASD	ASD	10	yes	no
asd	asd	55	yes	no
LRFD	LRFD	80	yes	no
ASD	LFD	90	n/a (no geotechnical div)	no
ASD	LRFD (not fully)	20	We do our best. When structure by consultants and geotechnical by state the communication is much more difficult.	no
ASD	LRFD	90	yes	no
ASD	LFD	yes	yes	n/a
ASD	ASD/LFD/LRFD	30	yes	no

DOT implementing or researching LRFD?	Geotech implem schedule?	Structural implem schedule?
no	none	none
yes - both geotech & struct.	by 2008	by 2005
no	sept. 2007	sept. 2007
no	July 2007	July 2007
yes - both geotech and structural	blank	blank
yes - structural, but not geotechnical	no, maybe in the next 2 years start thinking about	n/a
No calibration, only AASHTO factors	n/a	blank
yes, geotech and structural		
yes - structural	none	20% currently, full implementation by 2007
yes, both geotech and structural	sent in their implementation schedule, see file	see file
yes - structural no - geotechnical	when we are forced to	when the feds require it - we have done a few test structures
yes, both, geotech started research with University of Missouri-Columbia	depends on results from MU	all bridge structures end of 2005
structural only	none	selected projects only, so far only 3 projects done w/ LRFD
structural only		currently some superstructure design, full implementation in October 1, 2007
no - Geotech	no	2007
no - geotech	none	when suitable design software and design aids are developed
yes - structural, no-geotech	small, can't keep up w/ workload, prefers not to retrain	start in earnest Jul 1 '03, fully implem. Oct. '06
NO	2007	2007
yes - both	2007	2007
yes - structural	none	considering - no schedule yet
yes- geotechnical and structural	implemented for two years	implemented for two years
no	n/a	none
yes in both structure and geotechnical	n/a	implemented
yes, structural	n/a	implemented
yes, both in structural and geotechnical	might be 2005	n/a
yes, structural	n/a	in transition

LRFD better method?
have not researched it thoroughly but it appears to model loads and resistances better than ASD
No - uncertain load and resistance factors. For example, correlation for driven piles, with ASD is very low, and leads to less conservative design for not much cost savings.
no - extreme unfamiliarity with the subject
Don't know enough about it yet to form an opinion. Might be helpful if over conservative designs can be reduced due to accurate load and resistance factors. If one has to use generic factors, it seems that we come up with as using FS on final values
I don't know
No, in order to know the actual strength of the soil need to take hundreds of samples and run hundreds of triaxial tests. Not practical. Better to estimates on the conservative side and use ASD than try to use LRFD
prefer ASD. However, just begun to learn how to associate a resistance factor with a factor of safety. Basically with LRFD taking the "magic" gained by years of field experience out of equation. Maybe that's a good thing….
This is a trick question. Most of LRFD was really implemented in LFD. There are the same problems in both areas. So LRFD is not the main problem. LFD is the start of the problems
not correlated against our geotech methods or experience, unable to conclude if better or worse
yes. LRFD gives an opportunity to take credit for better investigations. The risks and unknowns can be applied to the most appropriate areas through the load and resistance factors. LRFD can be calibrated to real test results
most of the geologists have taken the FHWA LRFD course. At this time, it does not appear that there are major advantages to switching to LRFD in geotechnical design
different approach, at this point in time-don't think it is better or worse that working stress. From geotech perspective much of the code based on the working stress approach. still needs time to mature and fully develop the approp. resistance fact..
not worked with it enough to have an opinion
no - very difficult to devleop resistance factors specific to state's geology. Geotech analysis involves accuracy that is far less than can be achieved in structural engr. Soil-structure interaction is crucial
Haven't had enough experience yet to answer
no, difficulty in detmining representative resistance factors for a vast array of soil types in Mississippi
blank
Geotech branch no opinion. Structural branch believes that LRFD essentially same as that in the Standard Spec. 2002
No. Not too much has been done in geotech in the current LRFD.
yes, because LRFD is based on realistic design parameters
not sure
no, the current resistance factors need to be state specific. It is still empirical in nature.
No, not yet, because the database of load and resistance factors has not been developed. Also because LFD is used for substructure and superstructure design and ASD is used for foundations. Our consultants use ASD for folundations. With LRFD, there may be a risk that engineering judgment and experience are bypassed in geotechnical designs.
Conceptually LRFD has the potential for better design, but at the current stage of development, LRFD seems more costly.
Once AASHTO introduces design specifications to address deep foundationdesigns and earth pressures for LRFD use.
no. highlight a misunderstanding of the unit of geotechnical design.
yes, it will provide more rational approach to design

Load and Resist factors state specific?	If use LRFD then how long?	If implemented LRFD any resistance from consulting?
we do not have plans at this time, however if we change over to LRFD we will need specific facotrs for ND soils. SD and MN have similar soils and we could possibly get together with them	n/a	blank
not so far, but we will develop some	just starting	n/a
not used	not used	not implemented
Don't know if we would go state specific, but I doubt we would.	n/a	n/a - getting all soils consultants on board is key because tey wil be main developers of all this data and have most of the equipment used in testing
yes, used with TX CPT tests. The rest are from LRFD spec.	five years - not used for bridges on the off-system	no
N/A	N/A	don't know
State specific factors should be determined through calibration. Don't see that I the near gurture unless a research project is funded	one year	It becomes easier when the owner is requesting the work. Details were individually checked by reviewing plans for the initial projects
yes	three years, all geotech projects	some but only because results were more conservative
n/a	n/a	n/
no plans at this time	n/a	n/a
blank	blank	blank
Not in the forseeable future.	Consultants have been using it on their projects for the past 4 years.	Globally no, however, there are situations or specific designs where there is a reluctance to use the methods/resistance factors recommended by the code.
research at MU will help generate and implement state specific factors		
will require several sets of different factors for each geologic area - marsh, prarie and upland deposits.		
haven't had enough experience yet to answer	n/a	n/a
n/a	n/a	blank
blank	blank	blank
not determined at present. Have scheduled a workshop in the fall of 2003 to discuss this.	n/a	
Will likely develop state specific facotrs and verify all designs using ASD	n/a	n/a
n/a	n/a	n/a
blank	blank	blank
no. no plan to generate the state-specific resistance factors.	two years	no
n/a	n/a	n/a
working on the state specific resistance factors where feasible.	tried on a couple of projects in the last two years. On one unfamiliarity slowed the design and was abandoned.	no
no	n/a	n/a
n/a	n/a	n/a
will probably involve in a regional basis.	n/a	n/a

APPENDIX B FINITE ELEMENT METHOD AND CALIBRATION

B.1 Introduction

This chapter is devoted to calibration of finite element models. Each finite element program must be calibrated for its ability to predict the behavior of a structure that it tries to simulate. NIKE3D was selected to analyze the behavior of drilled shafts subjected to lateral loads. Finite element mesh is generated using TrueGrid software. Griz program will be used to visualize the results of NIKE3D analyses. Microsoft Excel will be used to process and calculate results from NIKE3D. Process in using finite element program is shown in Figure B-1.

Two material models will be used for representing pile and soil in NIKE3D models. One is elastic material model for pile and soil. Another is Ramberg-Osgood (RO) material model for soil.

In finite element analyses, calibration is the most critical. Calibration requires mesh or model adjustment. Full-scale or model test results are also required to check the model's validity. Two full-scale, pier load tests were selected to calibrate the models. One is 6-ft diameter pier tested by Dunnavant in 1986. Another is 2.5-ft diameter concrete drilled shaft tested by Reese and Welch in 1975. Both test results were also used in investigating Broms method and LPILE program as discussed in chapter 4 and 5, respectively.

Comparisons between Broms, LPILE, and NIKE3D will be given in this chapter. Advantages and limitations of each method will also be discussed.

B.2 Preprocessor: TrueGrid

Finite element methods require mesh generation software to generate mesh representing the problems. TrueGrid is mesh generation software. TrueGrid can generate meshes which is compatible to NIKE3D. TrueGrid will produce input files for NIKE3D.

Physical problems are converted to mesh systems. Pile dimensions and properties, soil properties, and boundary conditions are required as input data. Loading conditions, finite element analysis options, and load curves are necessary input data. Input files for TrueGrid are stored in text-file format.

Once TrueGrid was completed, the output file called 'trugrdo' was produced. trugrdo file contains necessary information in standard format for NIKE3D analysis.

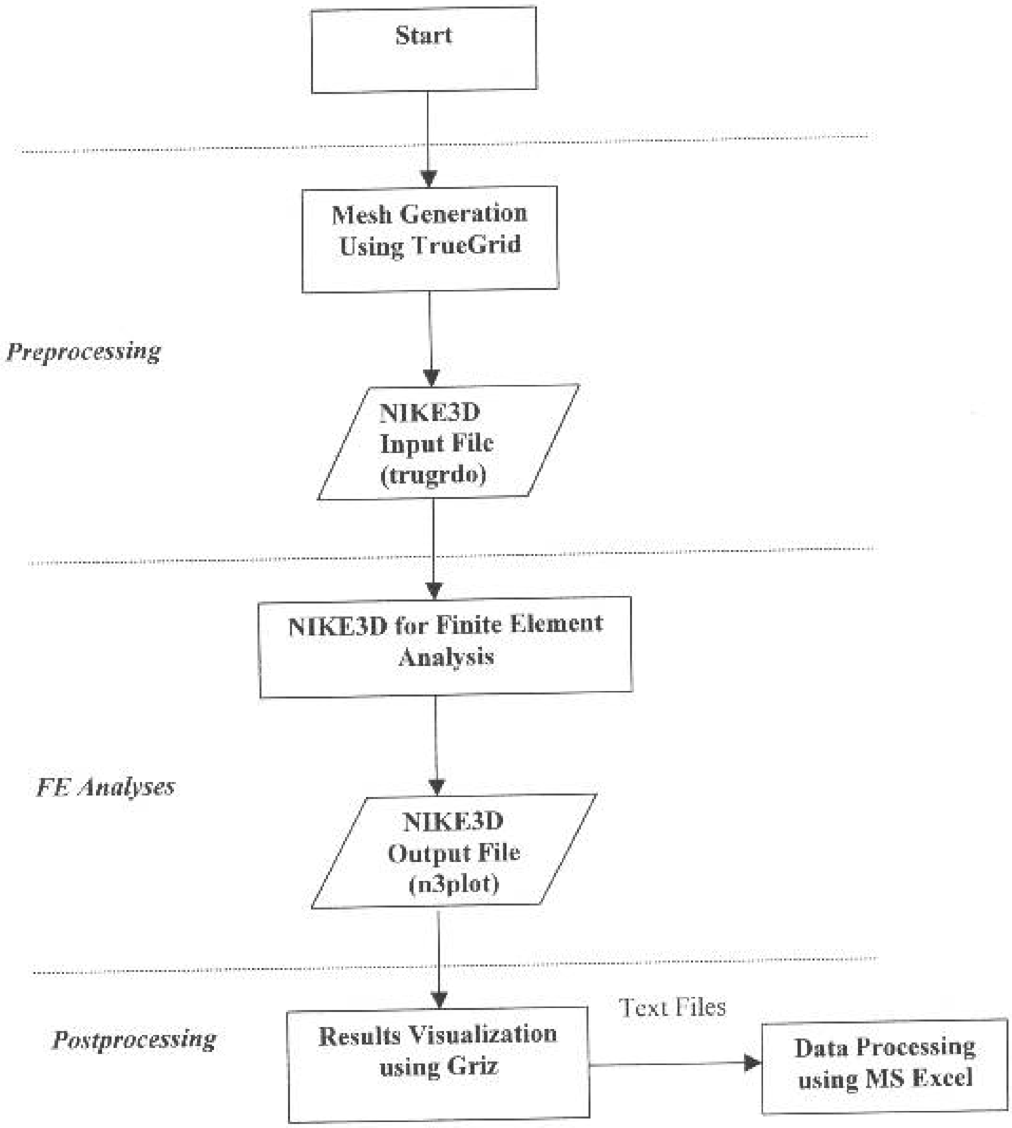


Figure B-1 Finite Element (FE) Analysis Processes

B.3 FE Software: NIKE3D

NIKE3D will be used as a finite element program in this thesis. NIKE3D is nonlinear, implicit, three-dimensional finite element software. It can be used in analyzing solid and structures of many materials. Two material models are used: elastic and Ramberg-Osgood models. Elastic material represents reinforced concrete pile and soil in the initial analyses. Ramberg-Osgood material is used to represent the nonlinear behavior of soil. Material parameters for elastic model are (1) Young's modulus of elasticity (E), (2) Poisson's ratio (ν), and (3) mass density (ρ). Material properties for Ramberg- Osgood model are (1) reference shear strain (γ_y), (2) reference shear stress (τ_y), (3) stress coefficient (α), (4) stress exponent (r), and (5) bulk modulus (K). These properties for RO models are generated using RAMBO software at given type of soil and mass density as explained in Chapter 3.

Using 'trugrdo' as an input file for NIKE3D, the result from the analysis is contained in output file called 'n3plot'.

B.4 Postprocessor: Griz

Once output from NIKE3D is obtained, software 'Griz' program is required to visualize results of NIKE3D analyses contained in 'n3plot' files. Results of NIKE3D analysis are presented in colored diagrams. Numerical results of NIKE3D analyses can be extracted and saved as text files. MS Excel then is used for processing numerical results from text files.

B.5 Calibration of NIKE3D with Available Field Test Results

Before using NIKE3D to predict the behavior of drilled shafts, it needs to be calibrated against model or full-scale test results. Two full-scale pier load tests are used in calibrating NIKE3D models.

Calibration is an iterative process. An initial, simple mesh is generated. Input data are assigned to the initial mesh. NIKE3D then analyzes the problem. Results of NIKE3D are compared with full-scale test results. If there are unacceptable differences between NIKE3D and test results, the adjustment of the mesh is required. Improvement of mesh and comparing with test results need to be done iteratively. The acceptable final mesh is the one that can produce acceptable difference between NIKE3D's and test results.

Initially pile and soil were considered elastic. Results from elastic-model run were compared with full-scale test results. Mesh adjustment can be done if required. When the elastic runs are successful, nonlinear analyses are performed using Ramberg-Osgood model for soil. Again, mesh adjustment may be necessary. This process is repeated until acceptable results from NIKE3D are obtained.

Flow chart for developing finite element model is shown in Figure B-2. Calibration process is the process shown in the dashed block.

B.5.1 Dunnavant's Pier Test

The test site was at the University of Houston. Dunnavant's pier is a reinforced concrete drilled shaft constructed in saturated stiff clay. A 6–feet diameter pier was constructed and flooded before the test was performed. Pier was reinforced with 24 #11 bars. Average bending stiffness was 3.92×10^{10} lb-ft^2. Penetration length was 37.5 ft below the ground surface. Lateral loads were applied on drilled shaft at 11 inches above the ground surface. The lateral displacement of the pier was measured by the inclinometer installed along the center of the pier. Water table was found at depth of 6 ft below the ground surface.

The soils at test site were natural soils containing two strata. The first stratum was 24-ft deep from ground surface called Beaumont clay. The upper 10-ft layer of the first stratum showed some secondary structure. The second stratum underlying the first stratum was a very thick layer of Montgomery clay.

Unconsolidated undrained (UU) and consolidated, isotropic, undrained (CIU) triaxial, vane shear (VST), and cone penetration (CPT) tests were used to determine the strength of soil. UU tests were conducted on soil samples up to the depth of 18 ft. Vane shear and CIU tests were performed in upper 5 ft. CPT were conducted along the whole length of the pile. Undrained shear strength data are shown in Figure B-3. From Figure B-3, undrained shear strength were found to increase with depth. Undrained shear strength from VST and CPT were greater than those from UU and CIU tests.

Young's modulus of elasticity in laboratory was defined as the secant modus at 20 % of the failure stress difference (E_{20}). Young's modulus of elasticity data provided by Dunnavant were UU triaxial and Cross-Hole tests (CHT) in the upper 18 ft. In addition, elasticity data provided by Marhar and O'Neill (1983) were included. Marhar and O'Neill used UU triaxial, presuremeters (PMT), and Cross-Hole tests. Elasticity from UU and PMT were reported up to depth of 40 ft below the ground surface. For CHT, data were presented up to depth of 60 ft below the ground surface. In general, elasticity increased with depth as shown in Figure B-4. From Figure B-4, CHT gave the highest values of elasticity compared to Young's modulus of elasticity from other tests. Elasticity from UU tests gave the lowest values. Unit weight data were also provided as shown in Figure B-5. Piles will be modeled using elastic material. Soil will be modeled using either elastic or Ramberg-Osgood materials.

Ground-line deflection data from Dunnavant's thesis is shown in Figure B-6. The curve can be divided into two portions: linear and nonlinear portions. Linear portion of the curve begins from origin up to the load of 180 kips. Loads greater than 180 kips show non-linearity of pile behavior. Dunnavant reported that this could be caused by tensile crack in the pile probably occurred at the depth about 20 ft below the ground surface. Load was applied up to 425 kips until it stopped because of the load frame buckling.

Deformed shapes of the pier at different loads are shown in Figure B-7. Guided by the deformed shapes of the pier, Dunnavant revealed: (Dunnavant 1986)

1. At large pile-head displacements the tip of the pile did not undergo large displacements (did not "kick out").
2. Based upon inclinometer measurements, pile curvatures at the 300- and 400-kip loads were large enough to produce compressive concrete strains up to 0.0013 and 0.0020, respectively, assuming flexure about the centroidal axis. Significant tensile cracking and nonlinear compressive stress-strain behavior probably occurred at about 20-ft depth below ground surface. This hypothesis is supported by the observation that the slope of the unloading curve is lower than the slope of the initial loading curve.

Dunnavant noted that there was translation of deflection shapes at 400 kips relative to shape at 300 kips. This translation can occur only in restrained head pile under lateral load. It has been observed that extensive concrete cracking and crushing probably occurred at 400 –kips load at about 20-ft depth below the ground surface (Dunnavant 1986). In Dunnavant' s thesis, he believed that the pile at this depth acted as a partial hinge. Consequently, the capacity of the pile to transmit moment on that section was reduced. Then, this hinge condition would resist pile tip to kick out as shown in Figure B-7.

From Dunnavant' s results, it is concluded that pile failure occurred at the load of 250 kips.

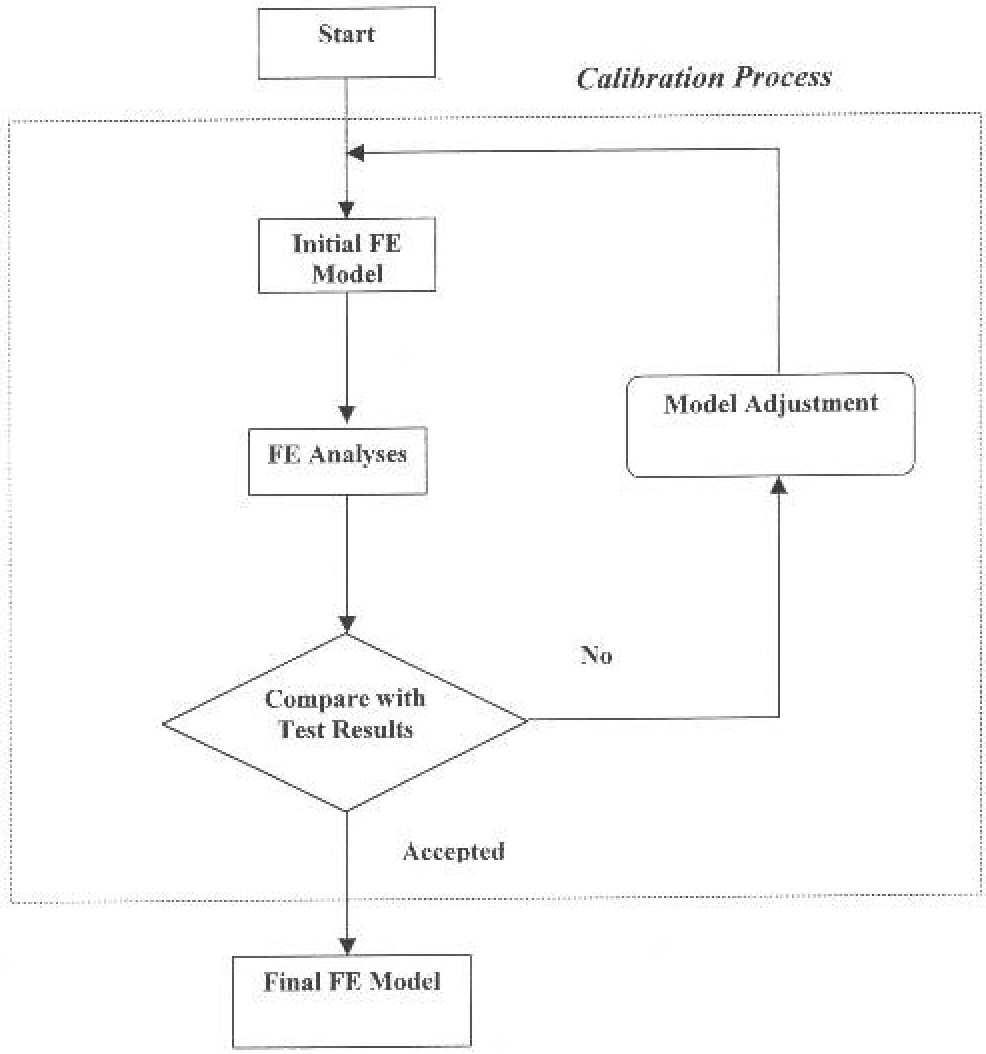
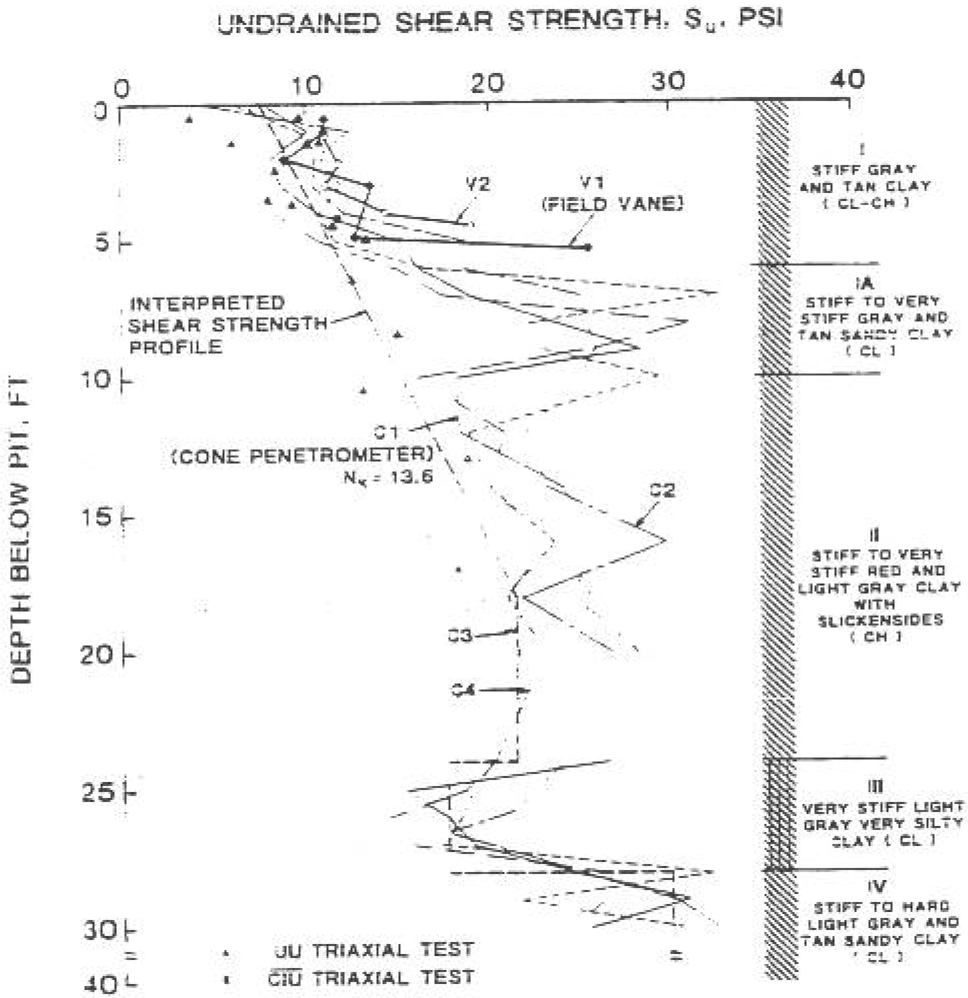


Figure B-2 The Development of Finite Element (FE) Model



FigureB-3 Undrained Shear Strength from Dunnavant's Test Site (Dunnavant 1986)

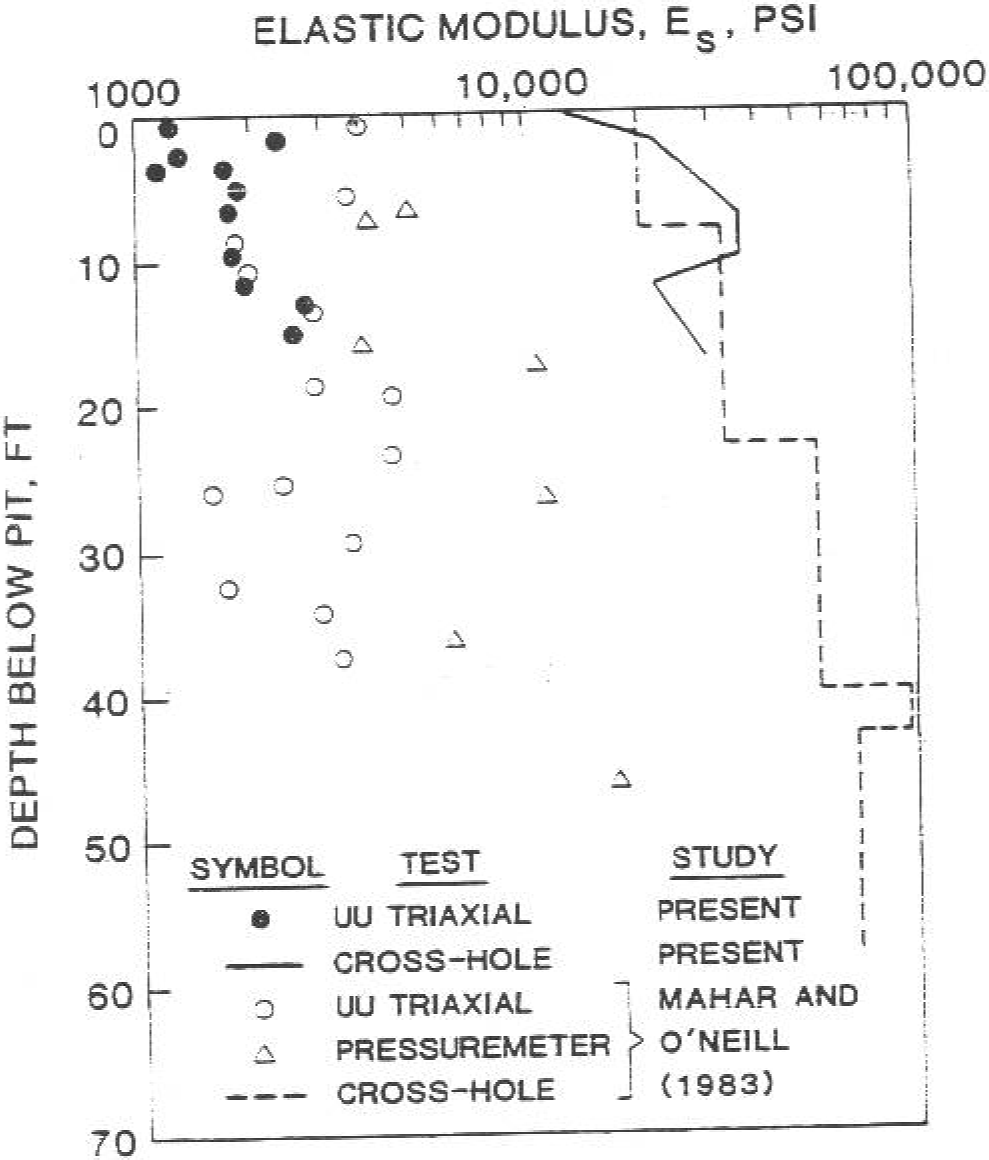


Figure B-4 Young's Modulus of Elasticity by Dunnavant (Dunnavant 1986)

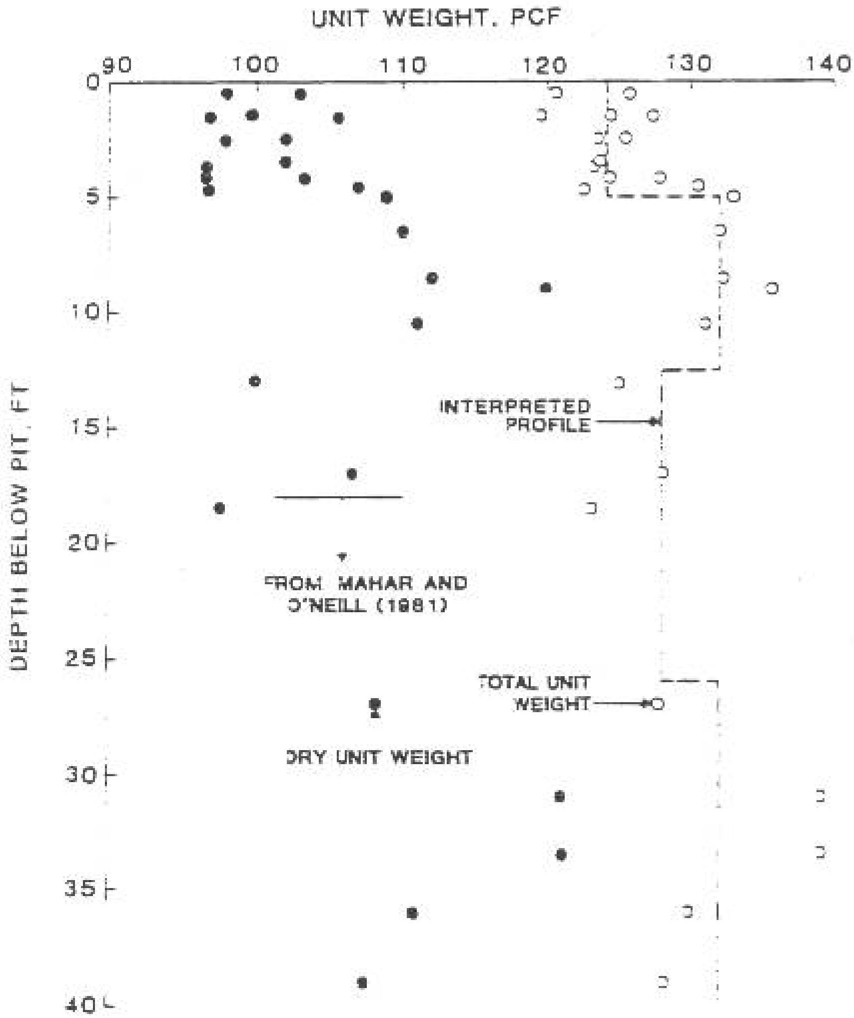


Figure B-5 Total and Dry Unit Weight by Dunnavant (Dunnavant 1986)

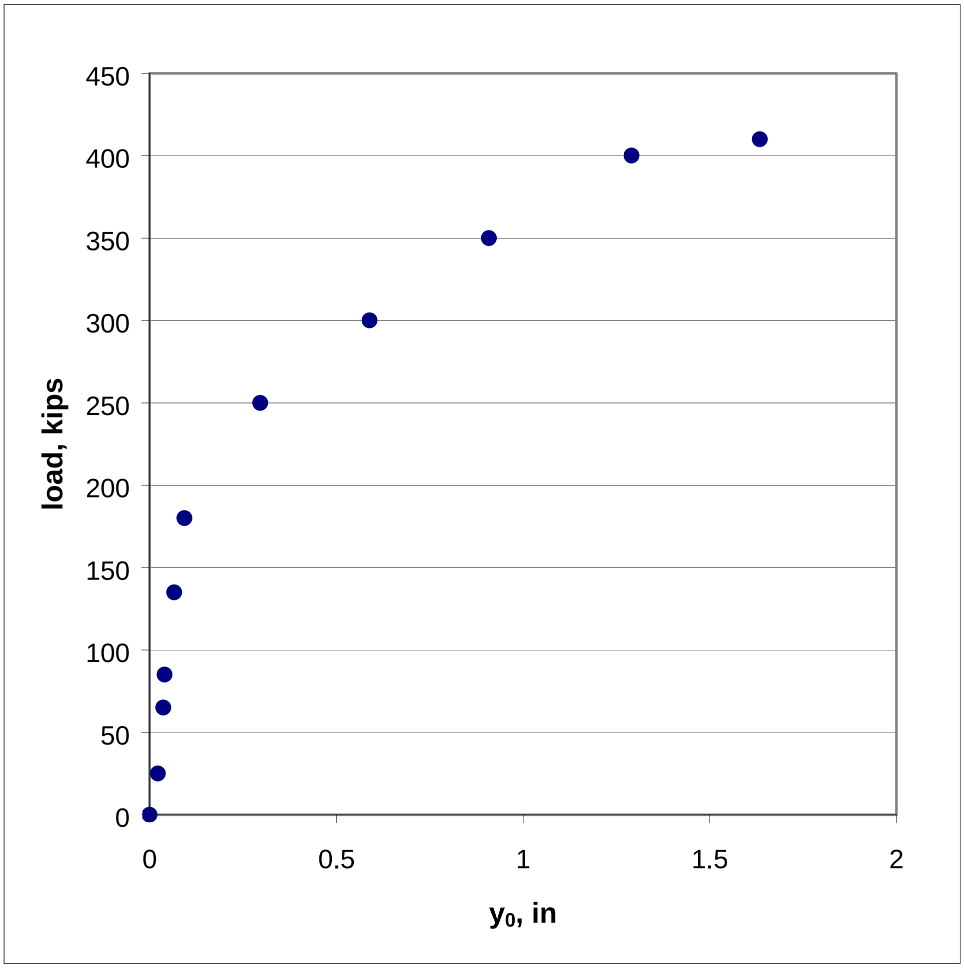


Figure B-6 Ground-line Deflections from Dunnavant Test

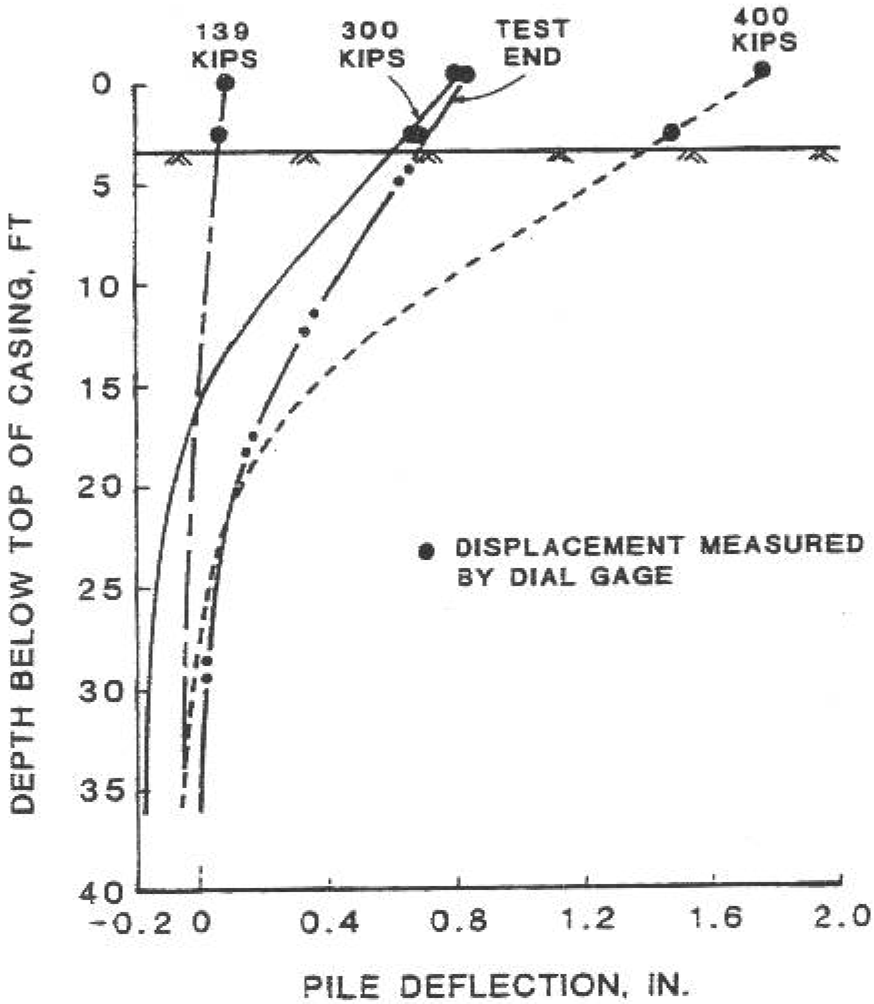


Figure B-7 Deformed Shapes of Dunnavant's Pier at Different Loads (Dunnavants 1986)

B.5.1.1 Elastic Soil Model

In these models, both soil and pile were considered elastic. Elastic properties for drilled shaft can be calculated using Dunnavant data. Modulus of elasticity of reinforced concrete pile (E_p) was $6.162x10^8$ psf. Poisson's ratio (ν_p) of 0.20 for pile was used. Pile mass density (ρ_p) was 4.50 slug/ft^2.

Two two-layered and one three-layered-soil meshes were investigated. The first two-layered-soil mesh of 18-ft plus 19.5-ft layers was selected. Young's modulus of elasticity for each layer was calculated. Unit weight and mass density were computed for each layer. Elasticity by UU tests, PMT, CHT, average of UU and PMT, and average between UU and CHT were separately calculated and input into NIKE3D. Each case was run separately by NIKE3D and its result is shown in Figure 6-8. From Figure 6-8, it is clear that the elasticity from CHT gives the best prediction of ground-line deflections in the initial part of the curve. Result using the elasticity by UU tests overestimates the ground-line deflections of the pier. Result using the elasticity by PMT gives ground-line deflections in between results using elasticity by UU and CHT.

The second two-layered-soil mesh consists of 24-ft plus 13.5-ft layers. In this model, elasticity by UU, PMT, and CHT were used. Figure B-9 shows ground-line deflections from these models. From Figure B-8 and B-9, it shows that the two-layered-soil mesh with 18-ft plus 19.5-ft gives the better results than 24-and-13.5-ft mesh in general.

B.5.1.2 Ramberg-Osgood (RO) Soil Model

For Ramberg-Osgood soil models, soils were represented using RO material but piles were modeled using elastic material. Using results from elastic soil models as guideline, the primary results show that 18-and-19.5-ft mesh is better. It is believed that the first 6-ft-deep soil layer would affect more on ground-line deflections of the pile. In RO soil models, Two-layered and three-layered soil systems were investigated. Two-layered-soil mesh consists of 24-ft plus 13.5-ft layers. Three-layered system consists of 6-ft, 12-ft, and 19.5-ft layers.

Modulus of elasticity from CHT was used for determining RO parameters. Shear (G) and bulk (K) moduli were calculated from E_{20} for a given Poisson's ratio. Ratio of shear modulus to maximum shear modulus (G_{max}) of 90 % was used. Other RO parameters can be determined using G_{max}.

Figure 6-10 shows ground-line deflections using 2- and 3-layered-soil meshes. For 2-layered mesh, ground-line deflections agree well with test results up to the load of 180 kips. For 3-layered mesh, the results from NIKE3D show good agreement with both 2-layered systems and test results up to the load of 50 kips. But 3-layered system gives better results with loads greater than 230-kips load. The two-layered-soil mesh is appropriate for linear portion of load-deflection curve. On the other hand, the three-layered mesh gives the good agreement in general. Based on RO model results, division of soil into appropriate number of layers is critical for results of FE models.

Deformed shapes of the pile at different loads are shown in Figure B-11. These show behavior of relatively stiff pile.

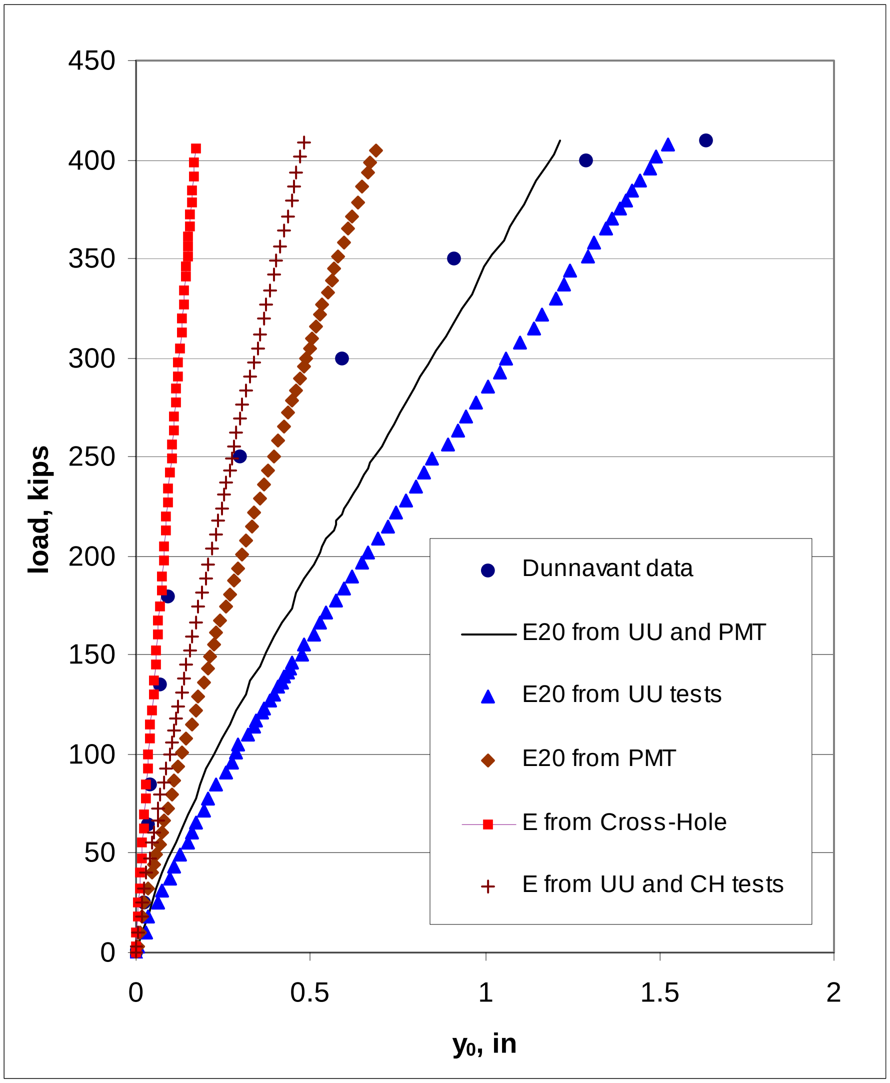


Figure B-8 NIKE3D Models of Ground-line Deflections using Different Secant Moduli with 2-layered Elastic Soil (18 and 19.5-ft layers)

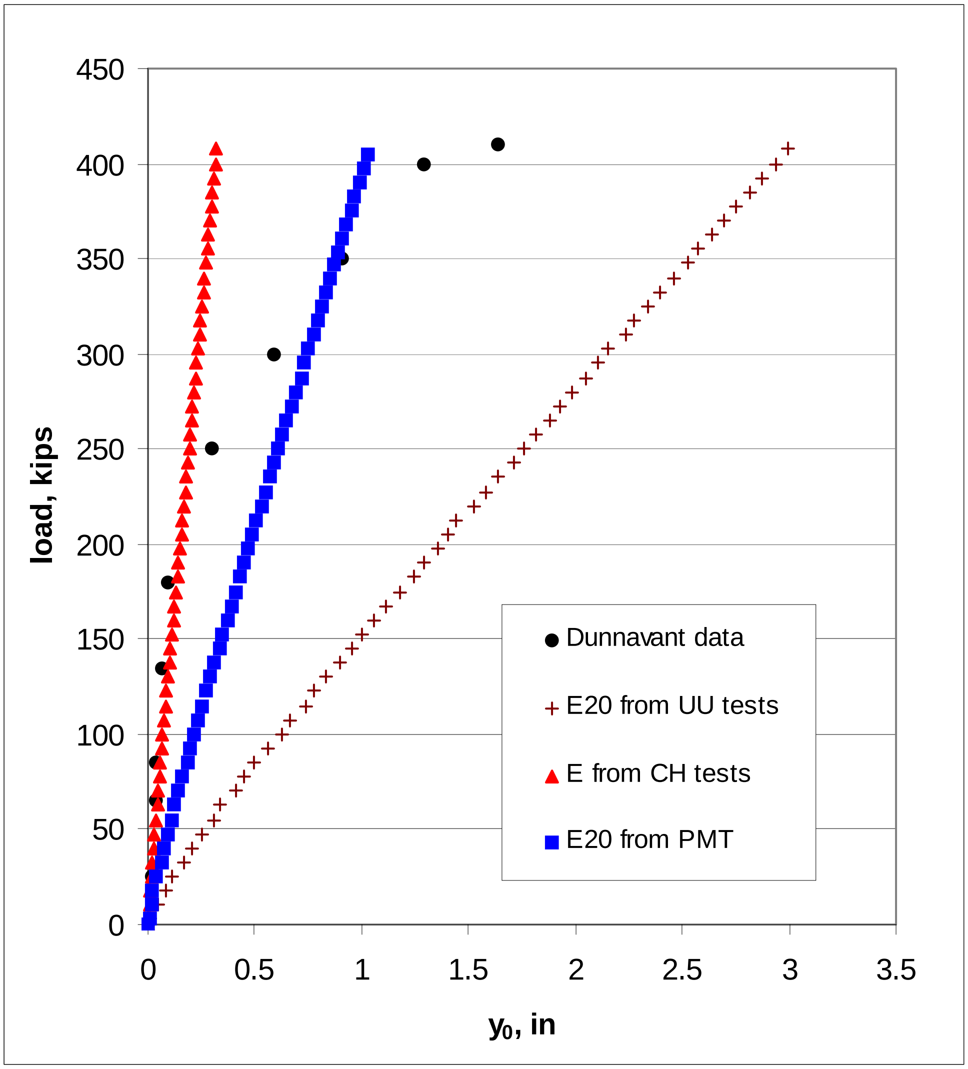


Figure B-9 NIKE3D Models of Ground-line Deflections using Different Secant Elasticity with 2-layered Elastic Soil (24 and 13.5-ft layers)

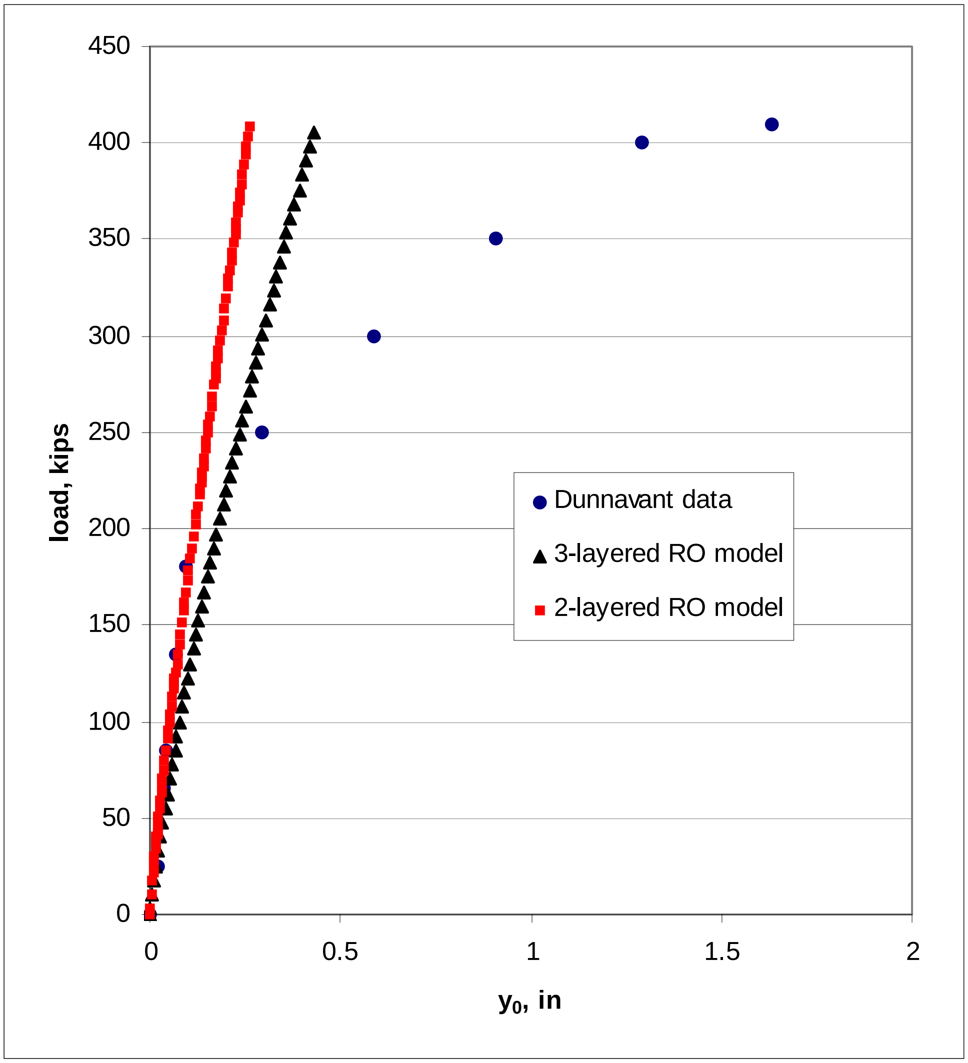


Figure B-10 NIKE3D RO Models of Ground-line Deflections with 2- and 3-layered Soil Meshes

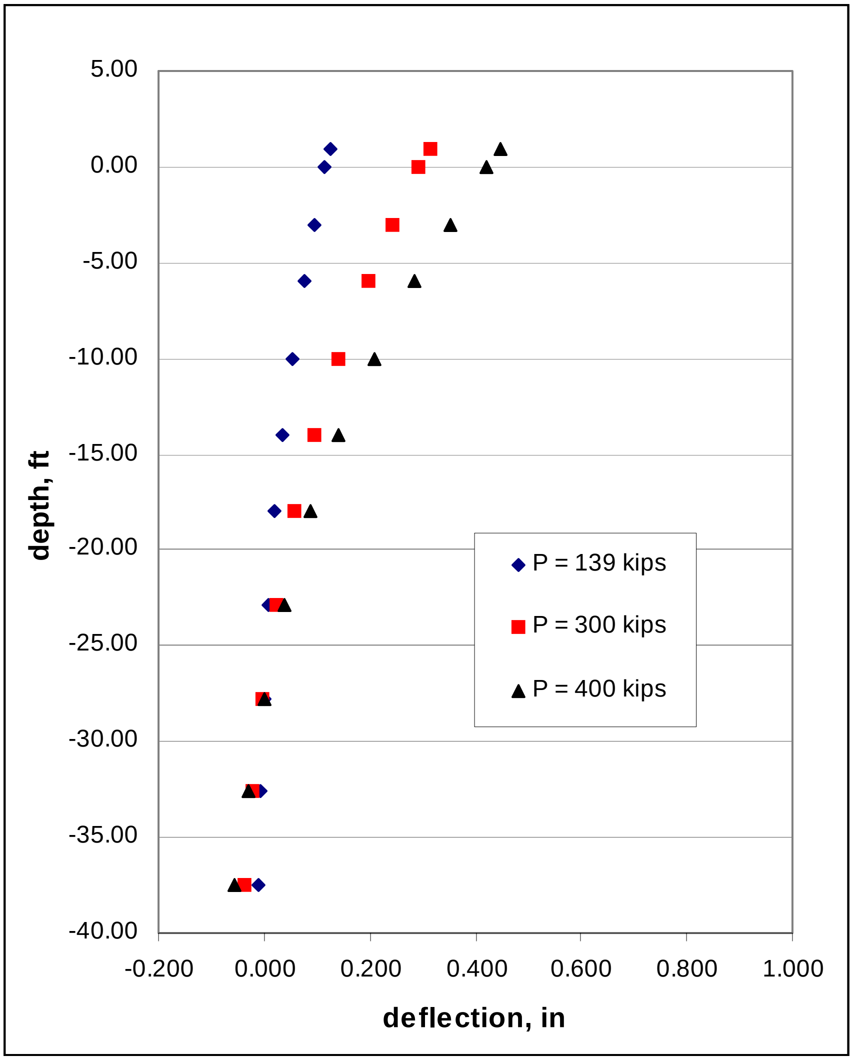


Figure B-11 NIKE3D RO Models of Deformed Shapes of the Pile at Different Loads using Two-layered Soil Mesh

B.5.1.3 Evaluation of Soil Young's Modulus

In this section, Young's modulus of soil from will be evaluated using three different ways. Elasticity was defined as the secant modulus determined at 20% of the failure stress difference. To find out the ratios between elasticity from different methods, soil profile was divided into three different meshes. These meshes are same as those used in FE models. The result is shown in Table B-1. Ratios were calculated for each sub-layer. Then the average from each sub-layer is used.

From Table B-1, average ratio of elasticity from PMT and CHT to UU tests is 2.9 and 15.5, respectively. The average ratio of CHT to PMT is 5.5. The maximum and minimum ratios of PMT to UU tests are 4.7 and 1.9, respectively. Maximum and minimum ratios for CHT to UU tests show the values of 21.5 and 10.6, respectively. The values of 7.5 and 3.7 were found for CHT to PMT.

Ratios of elasticity to undrained shear strength also were calculated as shown in Table B-2. Only results from UU tests were computed. The average, minimum, and maximum ratios of E/c_u are found to be 152, 133, and 185, respectively.

Table B-1 Relationships between Elasticity from Different Methods (Dunnavant data)

Layered Soil Mesh	Depth (ft)	Elasticity by UU tests (ksf)	Elasticity by PMT (ksf)	Elasticity by Cross-Hole Tests (CHT) (ksf)	E_{PMT}/E_{UU}	E_{CH}/E_{UU}	E_{CH}/E_{PMT}
2-layered mesh	0 to 18	338.4	738.0	4104.0	2.2	12.1	5.6
of 18-ft plus 22-ft	18 to 40	432.0	1386.0	7311.0	3.2	16.9	5.3
2-layered mesh	0 to 24	344.6	983.3	3654.0	2.9	10.6	3.7
of 24-ft plus 16-ft	24 to 40	374.4	1752.0	8064.0	4.7	21.5	4.6
3-layered mesh	0 to 6	217.5	500.0	3774.9	2.3	17.4	7.5
of 6-ft, 12-ft, and	6 to 18	335.0	635.0	4417.4	1.9	13.2	7.0
22-ft	18 to 40	440.0	1512.7	7386.5	3.4	16.8	4.9
Average					**2.9**	**15.5**	**5.5**
Max					**4.7**	**21.5**	**7.5**
Min					**1.9**	**10.6**	**3.7**

Table B-2 Ratios of Elasticity to c_u by UU Tests (Dunnavant data)

Depth (ft)	Elasticity (psf)	c_u (psf)	E/c_u
0	200000	1080	185
5	250000	1660	151
10	310000	2230	139
15	365000	2740	133
Average			**152**
Max			**185**
Min			**133**

B.5.2 Reese and Welch Test Results

The test site was in Houston. A 2.5-ft diameter drilled shaft was constructed in stiff clay. The shaft was reinforced with 20 #14 bars. Average bending stiffness was 1.39×10^9 lb-ft^2. Penetration length was 42 ft. Loads were applied at a height of 3 inches above the ground surface. Water table was found at 18 ft below the ground surface.

Soils at this site were called Beaumont clays. UU triaxial tests were conducted on undisturbed samples. Reese and Welch reported the most significant layer of soil was the upper 20 ft. The average undrained shear strength in this layer is 1.1 tsf. Between 20 and 33 ft, the average undrained shear strength was 1.0 tsf. From the depth of 33 to 42 ft, the average undrained shear strength was 1.65 tsf. Undrained shear strength is sown in Figure B-12. Undrained shear strength data was scattered but tend to increase with depth.

Soil Young’s modulus of elasticity was defined as the secant modulus at 50 % of the maximum principal stress difference (E_{50}). UU triaxial tests were used to determine soil Young’s modulus of elasticity on both vertical and horizontal samples. No significant difference in elasticity was found between vertical and horizontal samples. Elasticity data fell in a wide range from 2000 to 5000 psi between the first 20 ft. Below 20 ft, few data were shown and elasticity tended to decrease with depth. Modulus of elasticity is shown in Figure B-13.

Unit weight and water content data were provided as shown in Table B-3.

Piles will be represented with elastic material model. Both elastic and Ramberg-Osgood models were used for soil. Ground-line deflections of pile are shown in Figure B-14.

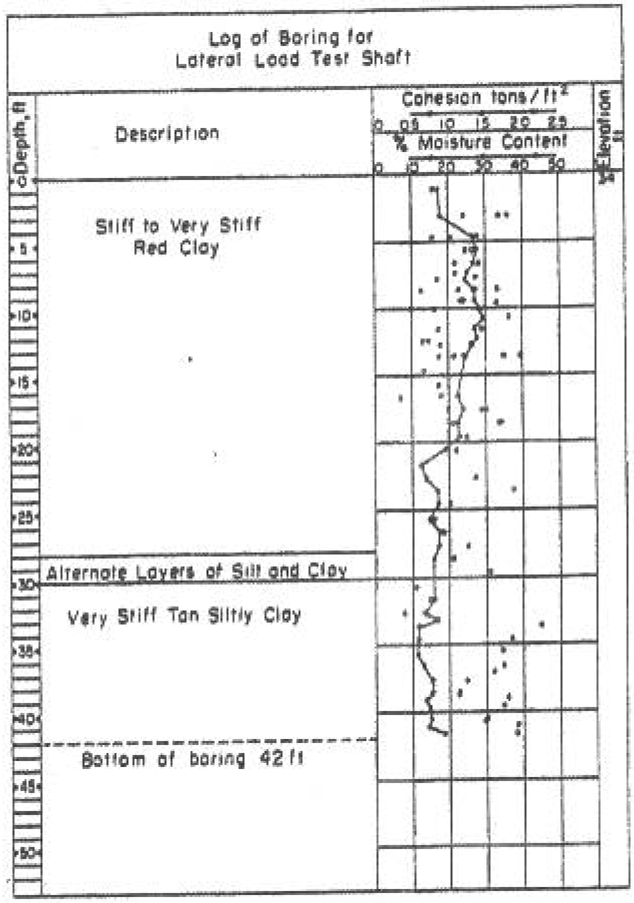


Figure B-12 Undrained Shear Strength from Reese and Welch's Test Site (Reese and Welch 1975)

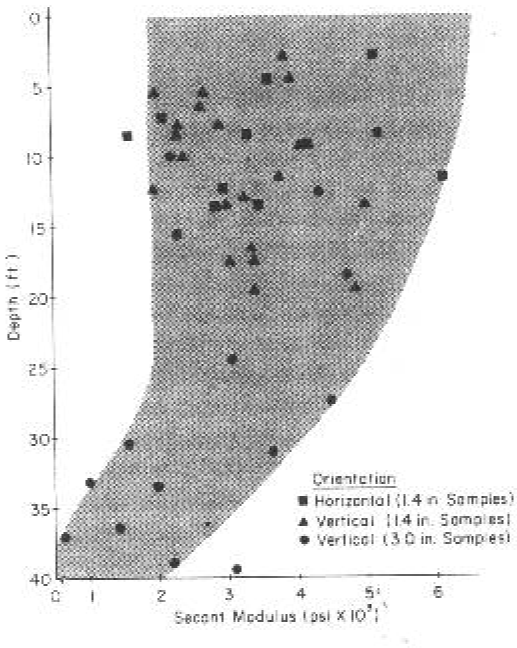


Figure B-13 Modulus of Elasticity from Reese and Welch (Reese and Welch 1975)

Table B-3 Total Unit Weight and Water Content from Reese and Welch

Depth (ft)	Total unit weight (pcf)	Water content (%)
0	123.6	18
1.3	123.6	18
3.4	119.7	22
20	121.7	20
42	126.8	15

Source: Adapted from Reese and Van Impe (2001)

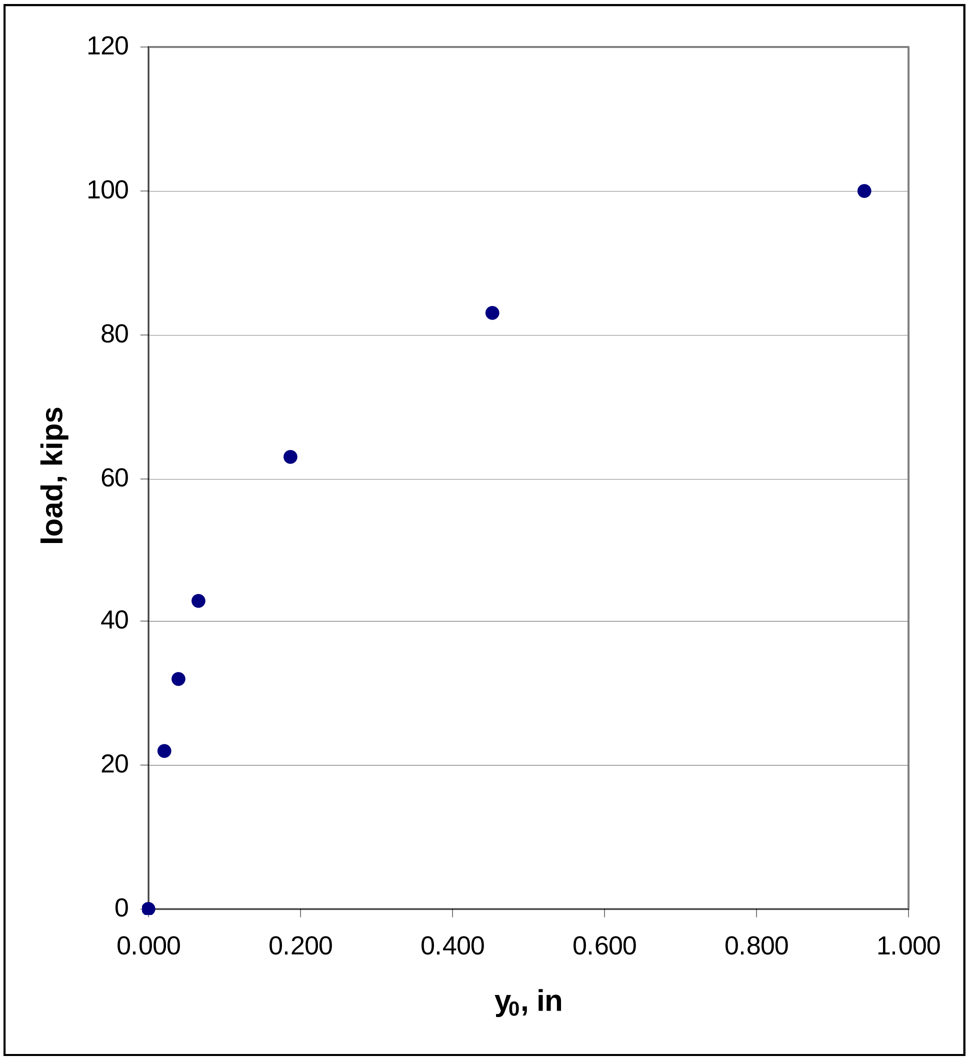


Figure B-14 Ground-line Deflections from Reese and Welch Test

B.5.2.1 Elastic Soils

As explained in Dunnavant test, soils and piles were considered elastic. From RO results in Dunnavant 's pier, results showed that the 3-layered mesh gave the better results in general. Hence, the 3-layered soil mesh will be used as an initial mesh. Three-layered soil mesh consists of 5-ft, 15-ft, and 22-ft layers. In Dunnavant results, the elasticity from UU tests (E_{UU}) gives the values much smaller than the elasticity from CHT (E_{CH}). So, the upper bound of E_{UU} was used. Elasticity of 7488 ksf was assigned to the first 5-ft layer. The elasticity of the middle layer was 7200 ksf. The elasticity for bottom layer was 5458 ksf. As mentioned in previous section, E_{UU} is much smaller than E_{CH}. It is assumed that E_{CH} is 10 times greater than E_{UU}. Then, E_{CH} or E_{10UU} was assigned to all three layers. Figure 6-15 shows ground-line deflections using both the elasticity from UU tests (E_{UU}) and the elasticity with 10 times greater than those from UU tests (E_{10UU}).

From Figure B-15, it shows that ground-line deflections from NIKE3D results using E_{10UU} agree well with test results up to 45-kip load. Then, this elasticity will be used in RO soil model.

B.5.2.2 Ramberg-Osgood (RO) Soils

The nonlinear RO model was used only to represent soils. Elastic material represented for piles. E_{10UU} was used for calculating RO parameters. These RO parameters were assigned to the three-layered mesh. Results are shown in Figure B-16. Results from elastic run are also shown in Figure B-16. It is found that both elastic and RO models gave similar results.

After reviewing undrained shear strength and elasticity data, it showed some controversy between these two data. Undrained shear strength tends to increase with depth. But elasticity shows decrease with depth. This relation is in contrary to the previous studies that show the undrained shear strength is proportional to elasticity of the soil. Then, the ratios of elasticity to c_u are used instead of the elasticity from UU tests. The ratio of elasticity to c_u for upper 20-ft layer was found to be 200. This ratio will be used for all layers.

The new mesh is composed of 20-ft, 13-ft, and 9-ft layers. Elasticity of the first 20 ft was used as a basis for elasticity and bulk moduli in other layers. The ratio of E_{50} to c_u for the first 20 ft of 200 was used. This ratio was also used for determining E_{50} of middle and bottom layers. Based on elasticity, shear modulus, maximum shear modulus, and bulk modulus can be computed. Shear modulus (G) is assumed to be 85% of maximum shear modulus.

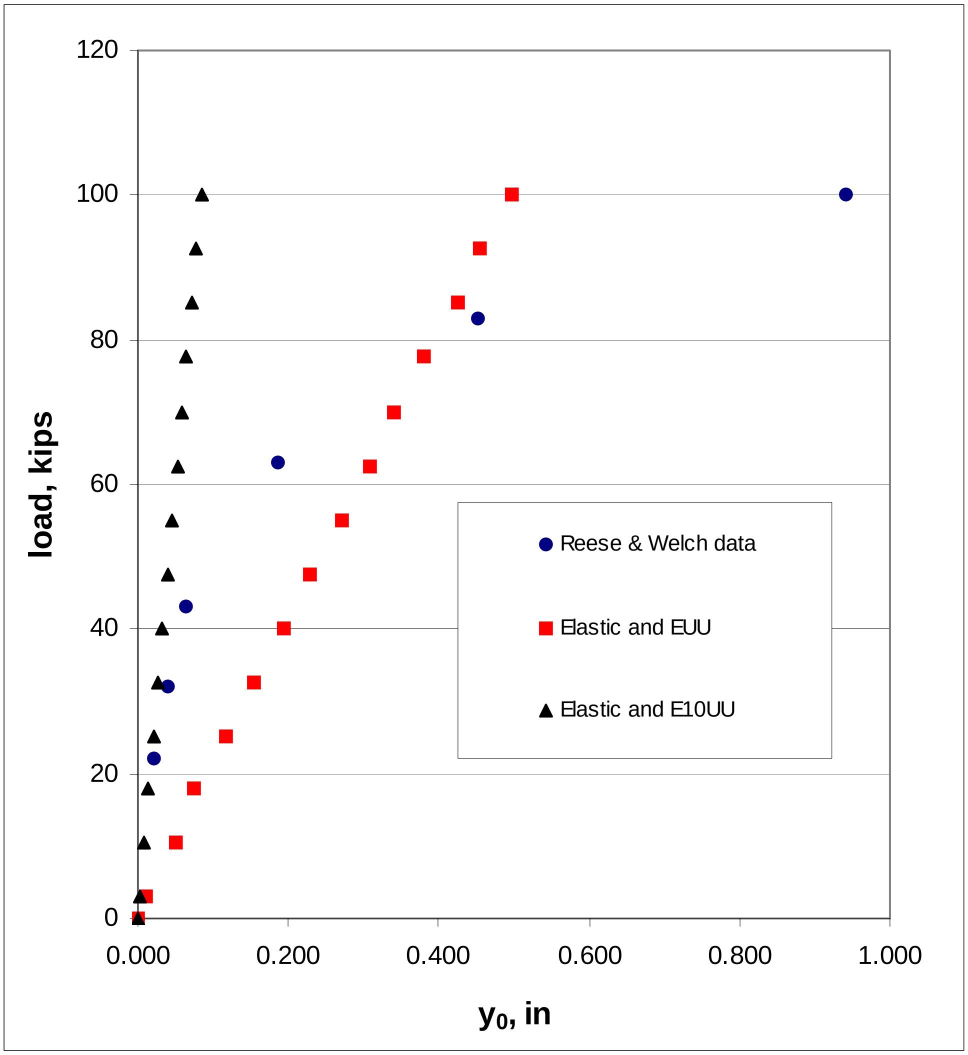


Figure B-15 NIKE3D Elastic Models of Ground-line Deflections for Different Secant Elasticity with 3-layered Soil (5-ft, 15-ft, and 22-ft layers)

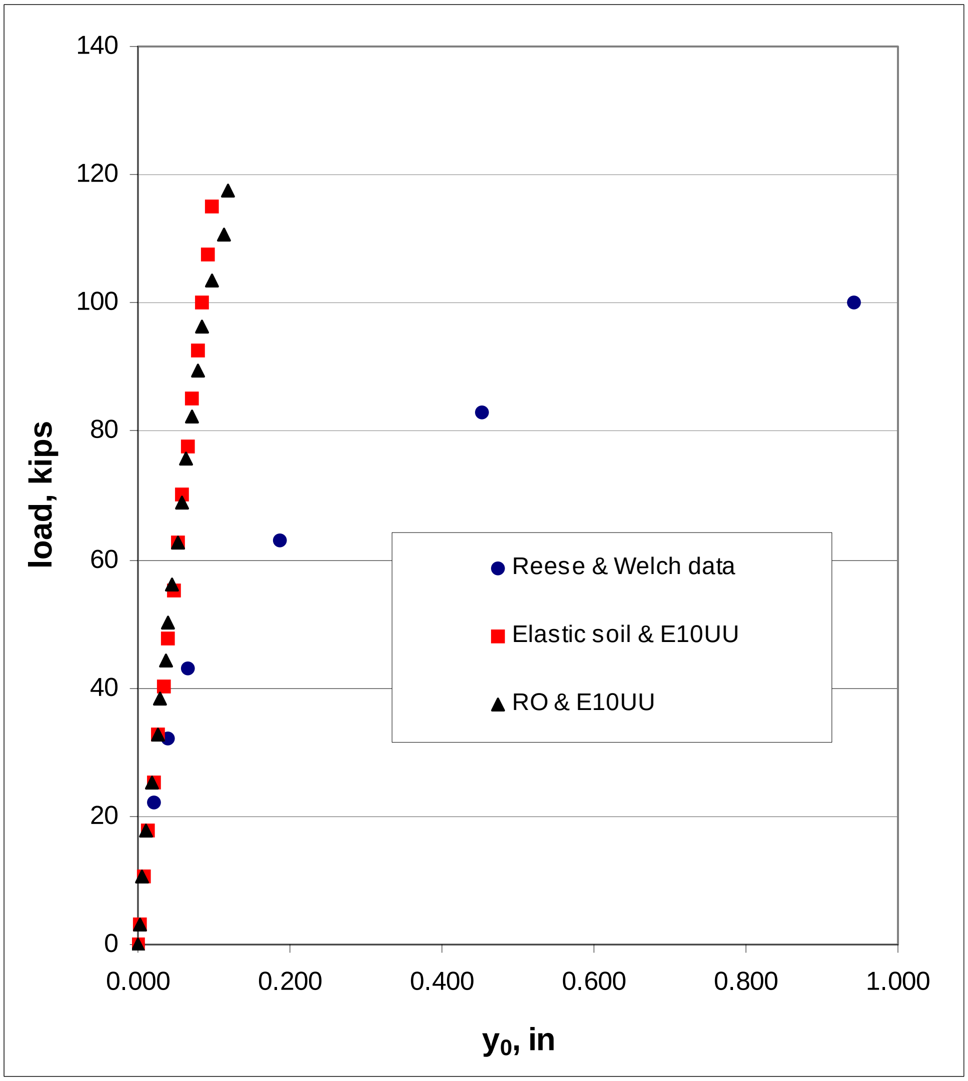


Figure B-16 NIKE3D Elastic and RO Models of Ground-line Deflections with 3-layered Soil (5-ft, 15-ft, and 22-ft layers) using 10 times Elasticity from UU Tests

The average c_u for the 20-ft layer is 1.1 tsf. c_u of 1.0 and 1.65 tsf were used in middle and bottom layers, respectively. Results from RO analysis for the three-layered mesh are shown in Figure B-17 (case I). The results show potential to give good agreement with test results.

The average values of c_u were used as in Figure B-17. It required stiffer soils to improve the agreement of the results. It is worth studying effects of soil stiffness or elasticity on load-deflection curve. Two cases were studied. First is to increase stiffness of middle layer (case II). Second is to increase stiffness of the first layer (case III). For stronger middle layer, c_u of 1.4 tsf was used. For stronger first layer, c_u of 1.3 tsf was used. These two values of c_u are still in the range between minimum and maximum values. Figure B-18 shows the results from three cases.

From Figure B-18, it shows that top layer has more effect than the middle layer. It is also clear that the agreement between RO and test results can be improved. The calibration process was continued to improve the results. Two additional cases were performed. The first case is to increase soil stiffness for all layers (case IV). Upper bound of c_u from each layer was used in this case. The second case is same as the first case but using stiffer soil in the top layer (case V). Results from case I, case IV, and case V are plotted in Figure B-19. The results from case IV and case V are similar. It shows that both cases improve the results.

Using case V's results, deformed shapes of the pile can be plotted in Figure 6-20. These show the behavior of relatively long pile.

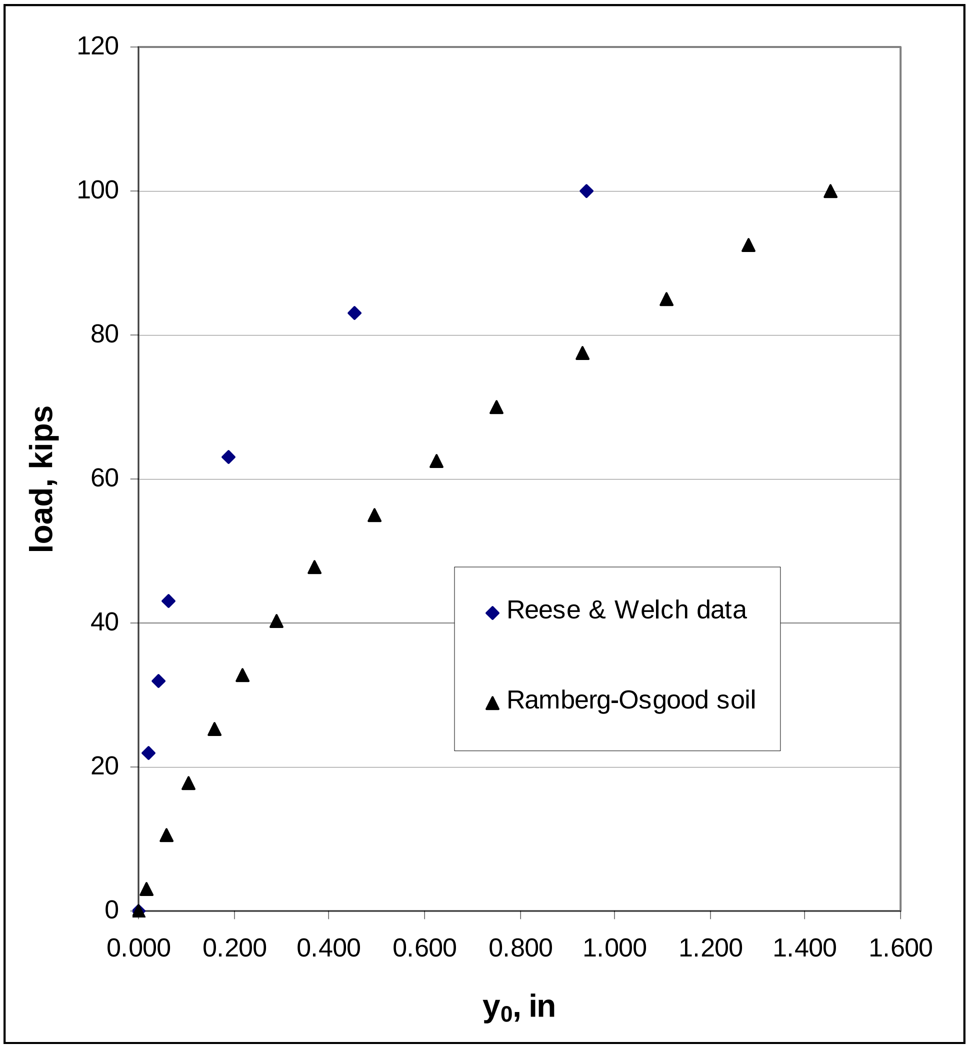


Figure B-17 NIKE3D RO Model of Ground-line Deflections with 3-layered Soil (20-ft, 13-ft, and 9-ft layers) using E_{50}/c_u = 200 (case I)

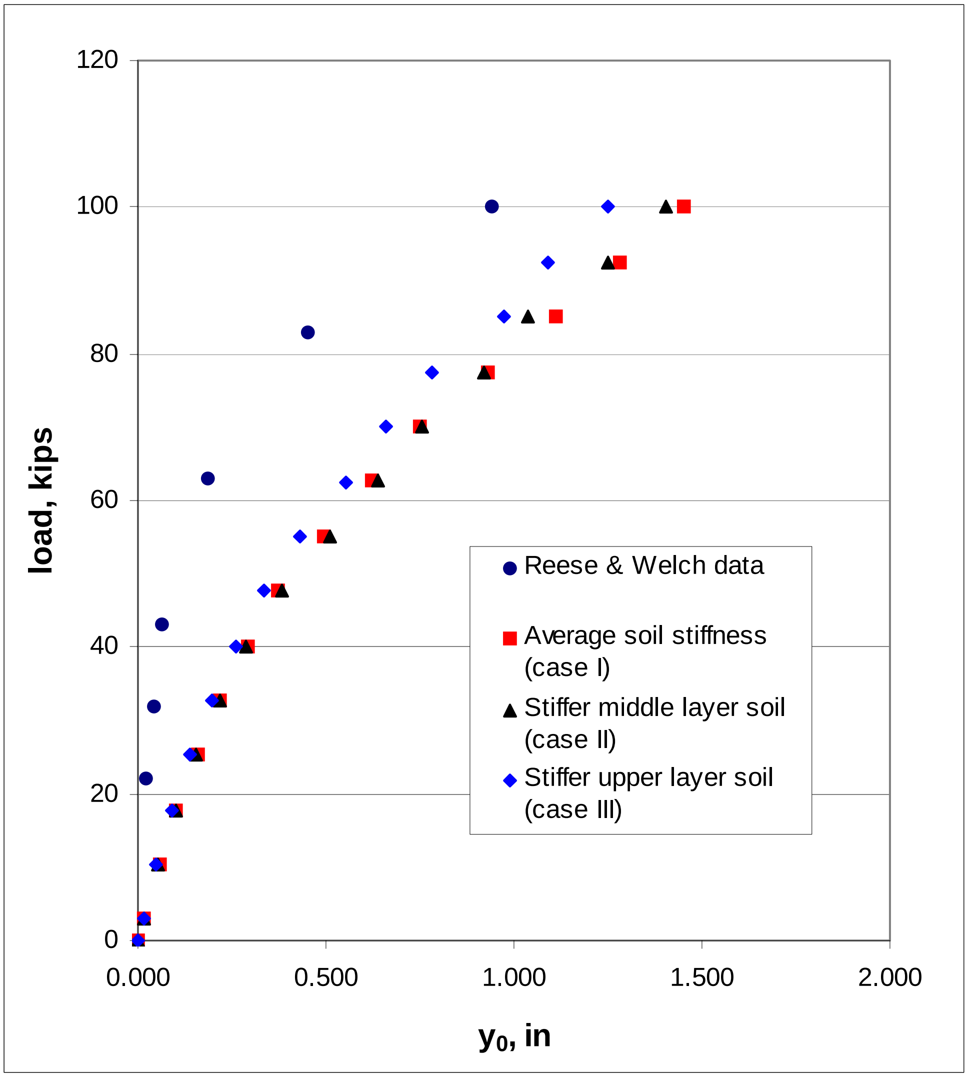


Figure B-18 Effect of Soil Stiffness (Modulus) on Load-Deflection Curve

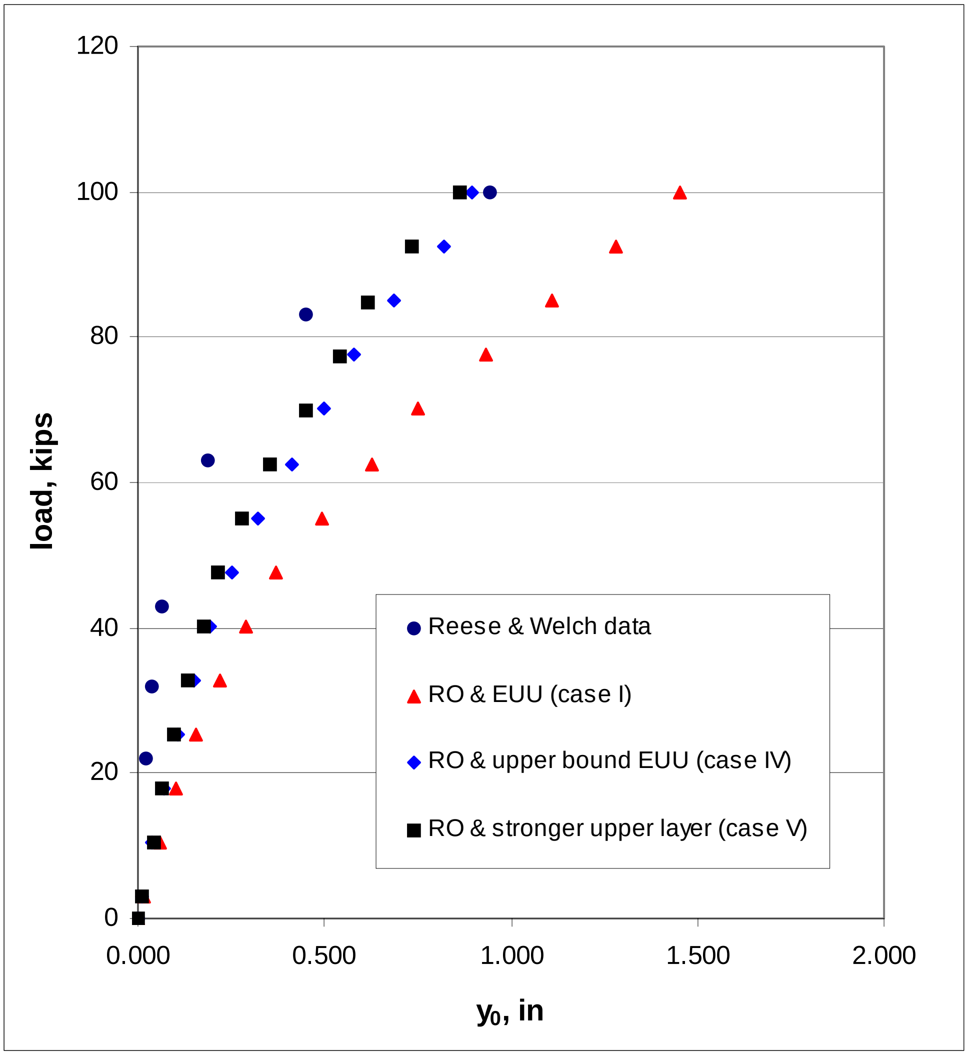


Figure B-19 NIKE3D RO Models of Ground-line Deflections for Different Soil Stiffness

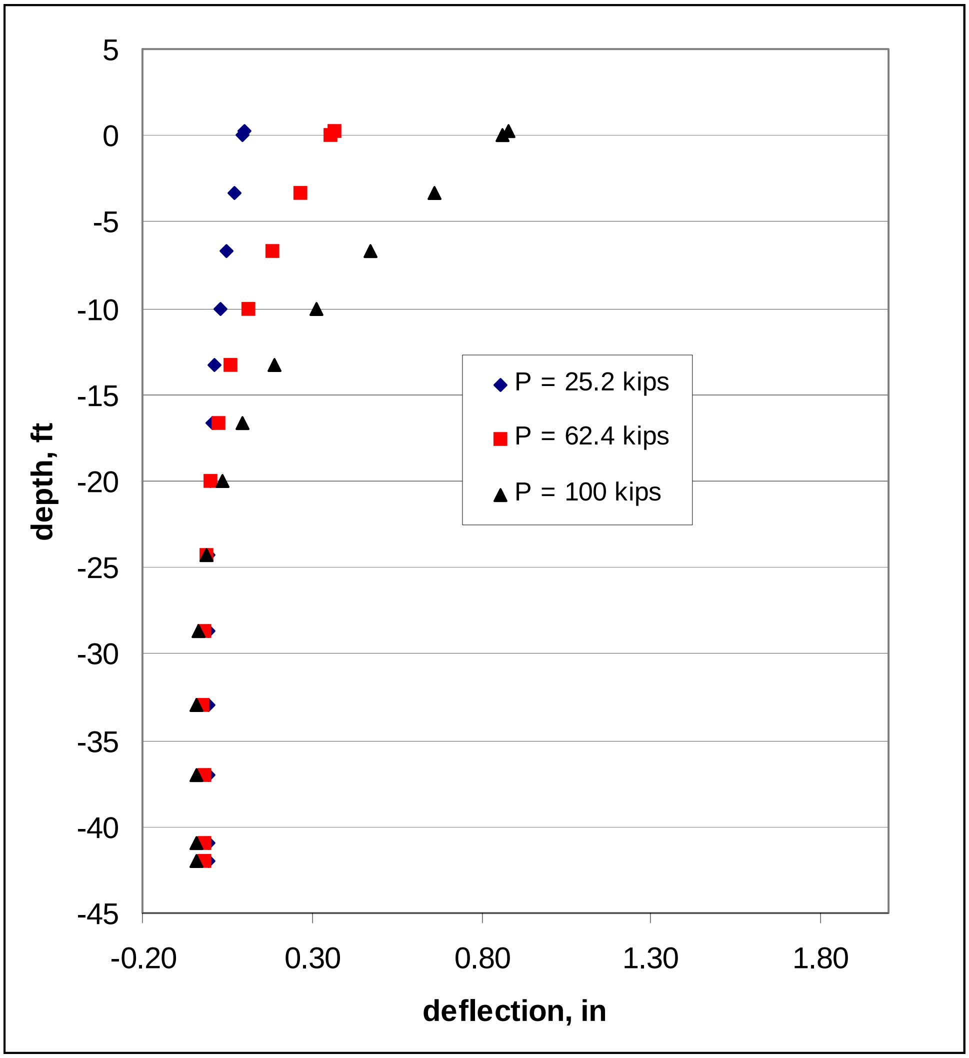


Figure B-20 NIKE3D RO Model of Deformed Shapes of the Pile at Different Loads (case V)

B.6 Comparison of Broms, LPILE, and Finite Element Analyses

In this section, results from Broms, LPILE, and NIKE3D will be compared. Both test results from Dunnavant and Reese & Welch were compared. Results from Broms and LPILE were already presented in Chapter 4 and 5, respectively. In this section, only selected results from Broms and LPILE are presented.

For Broms method, results using back-figured k_h were selected in both Dunnavant and Reese & Welch cases.

For LPILE results, the 2-layered mesh of 18-ft and 19.5-ft was chosen for Dunnavant case. The two-layered soil mesh consisted of 20-ft and 22-ft layers were selected for Reese and Welch case.

In NIKE3D, the three-layered RO model of 5-ft, 13-ft, and 19.5-ft layers was chosen for Dunnavant comparison. Three-layered RO model consisted of 20-ft, 13-ft, and 9-ft layers with stronger upper soil layer was used in comparison for Reese and Welch case.

The results of comparison are shown in Figure B-21 and B-22 for Dunnavant and Reese & Welch data, respectively. Comparisons of ground-line deflections between Broms method, LPILE, and NIKE3D for Dunnavant and Reese & Welch cases are shown in Table B-4 and B-5, respectively.

Broms method using back-figured k_h shows good agreement at loads up to 180 and 43 kips for Dunnavant and Reese & Welch results, respectively.

NIKE3D gives as good results as LPILE does in Dunnavant test. But LPILE shows better agreement with test data than NIKE3D does in Reese and Welch test. This could be because of Reese and Welch test is part of database used in developing LPILE.

B.7 Pros and Cons of Broms, LPILE, and NIKE3D

Advantages and limitations of each method will be discussed. Each method has its advantages and disadvantages. Table B-6 shows pros and cons of three methods.

Item No.1 “Material represented soil mass” refers to models used by each method. For example, linear soil in Broms method means soil is modeled with single linear spring. In LPILE, soil is represented with unconnected series of nonlinear springs. Soil is replaced by Ramberg-Osgood nonlinear material model in NIKE3D.

Item No.2 “Solving procedure” refers to procedures used in solving laterally loaded pile problems. In Broms method, solution procedure is easy and simple. Solution procedure for LPILE is a bit complicate than Broms. But it is still easy to get the results. More complicate in solution procedure is required in NIKE3D. Mesh generation and material selection make the problem more complicate.

In item No.3, soils are modeled using discrete linear and nonlinear springs for Broms and LPLIE, respectively. These methods consider soils as discrete materials. In NIKE3D, soils are represented by continuum mass.

In item No.4, soil is assumed to be uniform in Broms method. For LPILE and NIKE3D, soil can be divided into a number of layers.

Item No.5 relates to soil conditions in the problem domain. In Broms method, clay is saturated and overconsolidated. LPILE was developed using some test results. It is clear that soil test sites should have similar properties as the soil used in developing LPILE. NIKE3D can be applied to more general sites of any kind of soils.

In item No. 6, Broms method can be used to predict only the linear behavior of load-deflection relationships. LPILE and NIKE3D are able to predict the load-deflection relationships at any load.

Item No.7 considers the capacity of methods in predicting the deformed shape of the pile. Broms method is not able to predict the deformed shapes of the piles but both LPILE and NIKE3D are.

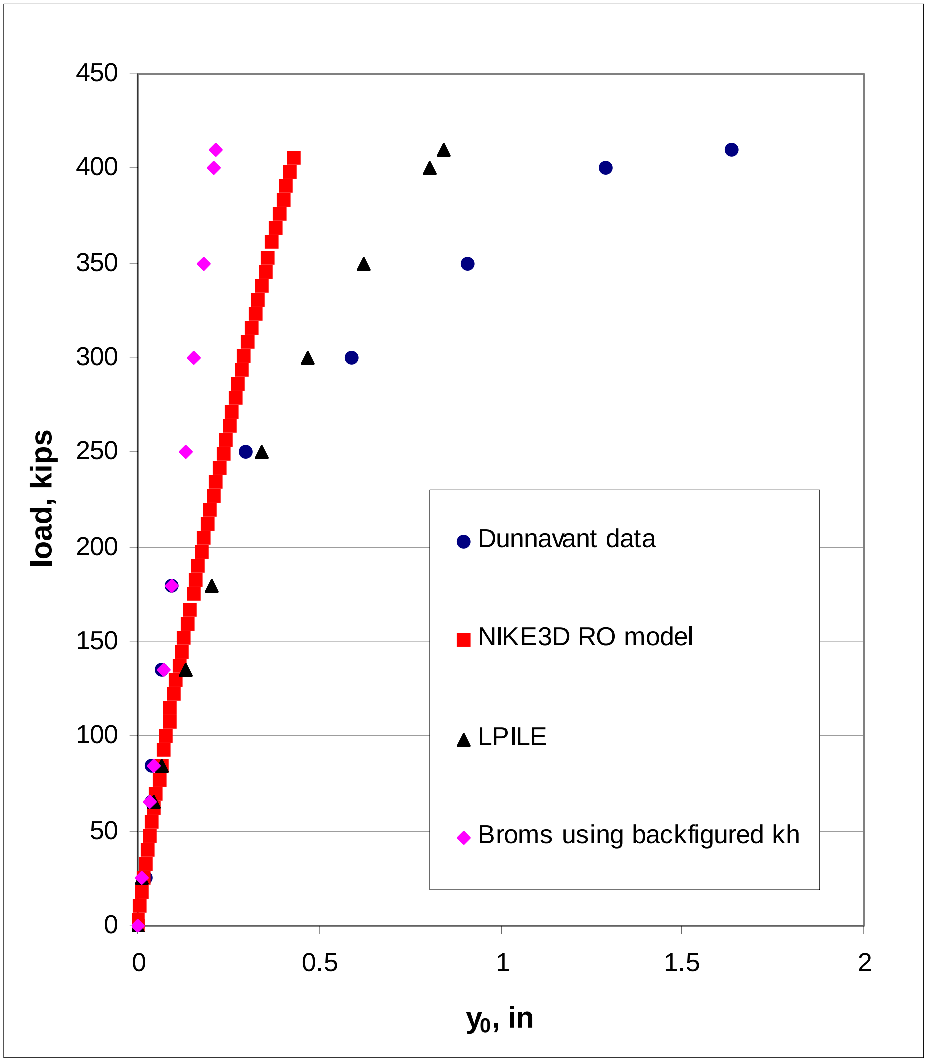


Figure B-21 Comparisons between Broms, LPILE, and NIKE3D for Dunnavant Case

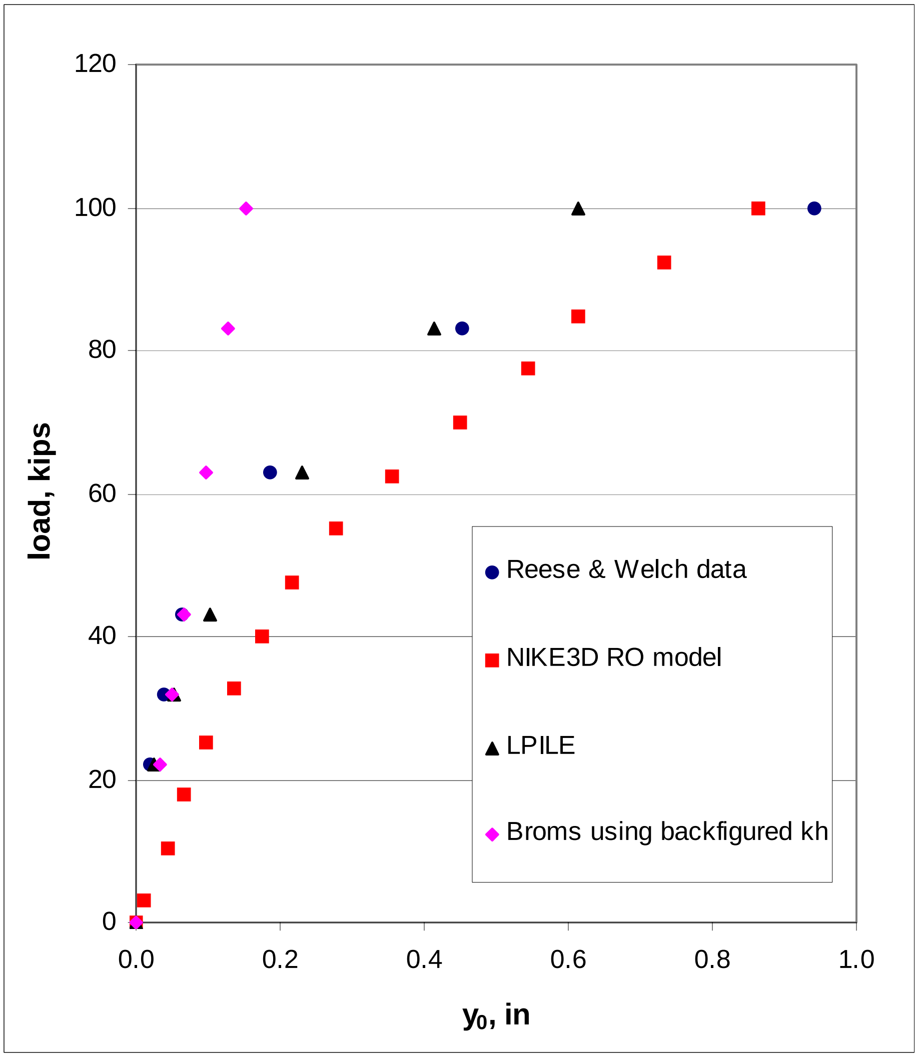


Figure B-22 Comparisons between Broms, LPILE, and NIKE3D for Reese and Welch Case

Table B-4 Comparisons of Ground-line Deflections (y_0) between Broms Method, LPILE, and NIKE3D for Dunnavant Case

Lateral Load	y_0 from Dunnavant	Broms Method		LPILE Program		NIKE3D	
(kips)	(in)	y_0, in	% Difference	y_0, in	% Difference	y_0, in	% Difference
0	0.000	0.000	0.00	0.000	0.00	0.000	0.00
25	0.022	0.013	-0.02	0.011	-50.00	0.017	-22.73
65	0.037	0.034	-0.01	0.044	+18.92	0.047	+27.03
85	0.040	0.044	+0.02	0.065	+62.50	0.066	+65.00
135	0.066	0.070	+0.03	0.130	+96.97	0.112	+69.70
180	0.093	0.093	0.00	0.202	+117.20	0.158	+69.89
250	0.296	0.130	-4.92	0.344	+16.22	0.236	-20.27
300	0.589	0.156	-25.52	0.470	-20.20	0.293	-50.25
350	0.909	0.182	-66.11	0.621	-31.68	0.356	-60.84
400	1.291	0.208	-139.86	0.803	-37.80	0.421	-67.39
410	1.634	0.213	-232.21	0.843	-48.41	0.434	-73.44

Table B-5 Comparisons of Ground-line Deflections (y_0) between Broms Method, LPILE, and NIKE3D for Reese & Welch Case

Lateral Load	y_0 from Reese & Welch	Broms Method		LPILE Program		NIKE3D	
(kips)	(in)	y_0, in	% Difference	y_0, in	% Difference	y_0, in	% Difference
0	0.000	0.000	0.00	0.000	0.00	0.000	0.00
22	0.020	0.034	+68.23	0.024	+20.00	0.085	+325.00
32	0.040	0.049	+22.35	0.054	+35.00	0.131	+227.50
43	0.065	0.066	+1.18	0.102	+56.92	0.191	+193.85
63	0.187	0.096	-48.47	0.231	+23.53	0.367	+96.26
83	0.452	0.127	-71.92	0.415	-8.19	0.597	+32.08
100	0.942	0.153	-83.76	0.615	-34.71	0.863	-8.39

Table B-6 Pros and Cons of Broms, LPILE, and NIKE3D

No.	**Items**	**Broms Method**		**LPILE Program**		**NIKE3D**	
		Pros	**Cons**	**Pros**	**Cons**	**Pros**	**Cons**
1	Material represented Soil Mass		Linear	Nonlinear		Nonlinear	
2	Solution Procedure	Simple		Simple			Complicate
3	Continuity of Soil Mass		No		No	Yes	
4	Layered Soil		No	Yes		Yes	
5	Application to Various Soil Conditions		Limited soil conditions		Limited soil conditions similar to data base	Any soil conditions	
6	Load-Deflection Curves		Only linear portion	Whole range of load		Whole range of load	
7	Deformed Shapes of Piles		No	Yes		Yes	

B.8 Conclusions

In previous sections, the development of finite element (FE) model was explained. This process consists of preprocessing of FE analysis input information, FE analyses, and post-processing of FE analysis results. Preprocessing is mesh generation using TrueGrid. FE analyses are performed by NIKE3D. Post-processing is visualization and data processing. Visualization can be done with Griz. Data processing can be obtained with MS Excel.

Calibration of NIKE3D was conducted using two pile-load-test results from Dunnavant (1986) and Reese & Welch (1975). Calibration process requires iterative procedure. Mesh or model adjustment are essential steps in FE development.

Dunnavant's test site was in Houston, Texas. On-site soils were saturated, overconsolidated clays. Soil and pile data were provided. In Dunnavant case, the pile failed at the load of 250 kips. So the results used up to 250-kip load are used in the calibration. Both elastic and RO models were investigated. Two 2-layered soil meshes using elastic soil were studied. In elastic models, elasticity from CHT gave the best fit for test data up to 180 kips for both layered systems. Both 2-layered and 3-layered soil meshes were analyzed using RO model. Results from both soil meshes with RO model gave good agreement with test results for load up to 250 kips.

Reese & Welch site was in Austin, Texas. Soils at the site were overconsolidated clays with water table at the depth of 18 ft below ground surface. Two cases were analyzed on elastic 3-layered soil systems. Five RO models (case I to case V) were investigated. It was found that the upper layer of soil had most effect on ground-line deflections of the pile.

Results from Broms, LPILE, and NIKE3D were compared. Broms method' s results were calculated using three different methods in assessing k_h of the soil. They are back-figured k_h, Terzaghi, and Davisson. Broms method using either Terzaghi or Davissons' methods in evaluating k_h fails to predict ground-line deflections of the pile in both Dunnavant and Reese & Welch tests. Results from Broms using back-figured k_h have good agreement with linear portion of load-deflection curves for both test data.

NIKE3D showed better results than LPILE did in Dunnavant test for the load up to 250 kips. But LPILE showed better results for load beyond 250 kips. As already known, pile failed after 250-kip load. So, the prediction of load-deflection curve for load after 250 kips by LPILE is in question.

LPILE showed better results in Reese and Welch test excepting the last point at load of 100 kips. This leads to the question of using LPILE to predict pile behavior after pile was loaded to a certain load. NIKE3D showed ability of prediction of load-deflection curve and gave the results closer to test data. NIKE3D also appeared to have potential in giving better results if more soil data are provided.

From the comparisons of two test results, NIKE3D showed good results for both cases. While LPILE leaves us with question about how good it can predict the behavior of pile nearing failures.

NIKE3D can predict more insight into pile-soil interface behavior than either LPILE or Broms method. NIKE3D can provide shear force at tip of the pile for static equilibrium verification but LPILE does not have this ability. Soil pressure distribution around and along the pile can also be obtained by NIKE3D. It has ability to predict pile behavior under combined loadings and other more complicated problems.

APPENDIX C. FINITE ELEMENT ANALYSES OF ROCK SOCKETED DRILLED SHAFTS UNDER LATERAL LOAD, MOMENT AND TORSION

C.1 Drilled Shafts for Sign Pole Structures

Sign pole structures are subjected to winds. Wind loads can induce lateral force, moment, and torsion imposed on sign pole foundations. Usually drilled shafts are used as foundations. This chapter will discuss the drilled shaft performance under lateral load, moment, and torsion. Finite element method was used in analyzing drilled shafts under lateral load, moment or torsion.

It is usually assumed that piles or drilled shafts supporting sign pole structures fail by lateral loads or moments. The failure criteria of piles under lateral loads or moments are either lateral deflections or ultimate moments of the piles. But the piles also are subjected to torsions. There is a possibility that torsions could control the design or the any combination of lateral loads, moments, and torsions. While this thesis would not accomplish this goal of pile performance under the above combined loads, it is important to unfold the mystery pile behavior. To design piles or drilled shafts, failure modes under loads need to be known. Failure modes of piles under lateral loads and moments are same. Failure modes for piles under torsions are not clear. It could be either axial rotation of pile or pile itself fails under torsion.

C.2 Rock Socketed Drilled Shafts

In this chapter, the rock socketed drilled shaft under different loads is analyzed. Concrete and soil properties are presented. Parameters for NIKE3D are also given.

Drilled shaft is assumed to have a diameter of 4 ft and total length of 40 ft. The bottom 10 ft of the shaft is situated in the rock as shown in Figure C-1. Top of drilled shaft is at ground surface. Failure criteria for drilled shafts under lateral loads, moments, and torsions are presented.

The 4-ft diameter drilled shaft of 40-ft length with bottom 10 ft socketed in the rock was analyzed. This drilled shaft is assumed to situate in uniform medium and hard clay layers overlain over the rock. Average undrained shear strength of 0.75 and 1.75 tsf were assigned for medium and hard clay layers, respectively. This soil-and-pile system will be used in the FE analyses.

As the rock properties, Young's modulus and Poisson's ratio are chosen as 250,000 ksf and 0.25, respectively. Soil and rock properties are summarized in Table C-1. Ultimate compressive strength ($f_c^{'}$) of concrete is 5000 psi. Concrete has Young's modulus of 4,050,000 psi. The steel reinforcement consists of 19 #14 bars with yield strength (f_y) of 60,000 psi.

Two different material models were used. Rock and concrete were assumed elastic. Clays were assumed to behave nonlinearly like Ramberg-Osgood (RO) materials. Elastic parameters of rock and concrete are shown in Table C-2. Table C-3 shows RO parameters for clays.

In NIKE3D, the coefficient of friction (μ) between clay and pier is assumed to be 0.35 for both medium and hard clays. The interface between rock and pier has the coefficient of 1.0.

Drilled shafts under lateral loads or moments can fail in the ultimate bending moment, the lateral deflections, or shaft-top rotation. The shaft used in analyses has the ultimate moment capacity of the section of 4000 kip-ft. AASTHO (1998) recommends that the allowable lateral deflections of piles should not exceed 1.5 inches. As a rule of thumb, the maximum pile-top rotations should not be greater than 1.0 degree.

The current design code has not defined the failure criteria for drilled shafts under torsions. Anyway in this thesis, shafts under torsions can fail in either the ultimate shear resistance of concrete at shaft surfaces or the axial rotation. The limiting shear stress of concrete pile caused by torsion is $2.4\sqrt{f_c^{'}}$ as recommended in the ACI Building Codes (1989). Table C-4 shows failure criteria for shafts under lateral loads, moments, and torsions.

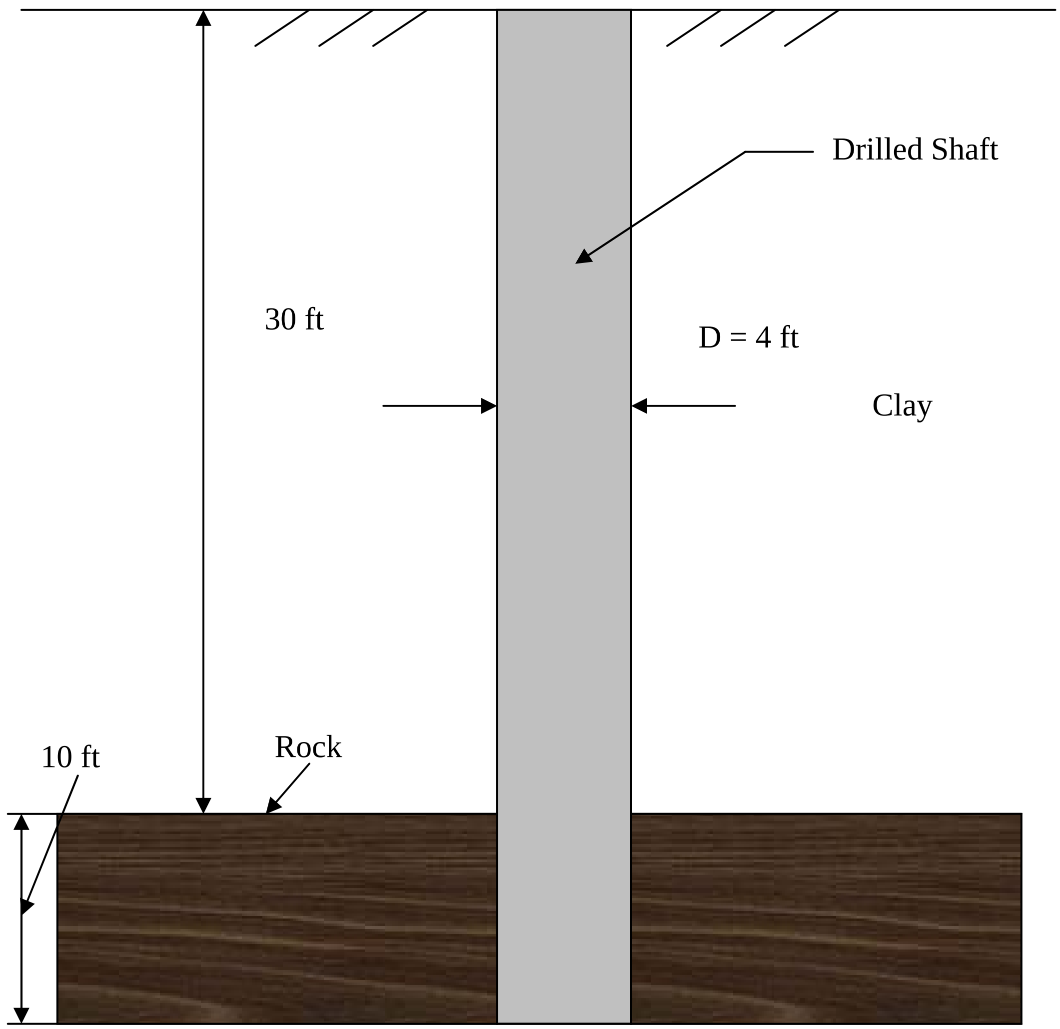


Figure C-1 Rock Socketed Drilled Shaft used in Analyses

Table C-1 Soil and Rock Properties

Material	**c_u (tsf)**	**E (ksf)**	ν	**Unit Weight (pcf)**	**E/c_u**
Medium clay	0.75	181	0.35	122	115
Hard clay	1.75	657	0.35	129	180
Rock	575	250,000	0.25	150	207

Table C-2 Elastic Parameters for Rock and Concrete

Materail	**E (ksf)**	**Density ($slug/ft^3$)**	**ν**
Rock	250,000	4.66	0.25
Concrete	670,000	4.5	0.20

Table C-3 RO Parameters for Clays

Clay	**τ_y (psf)**	**γ_y**	**α**	**r**	**K (ksf)**
Medium	28.28	0.000359	1.257	5.0	201
Hard	101.90	0.000359	1.257	5.0	723

Table C-4 Failure Criteria for Drilled Shafts

Failure Criteria	**Lateral Load**	**Moment**	**Torsion**
Ultimate Moment (kip-ft)	4000	4000	N/A
Lateral Deflection (in.)	1.5	1.5	N/A
Shaft-top Rotation (degree)	1.0	1.0	N/A
Limiting Shear Stress of Pile Section (psf)	N/A	N/A	24,400
Axial Rotation (degree)	N/A	N/A	1.0

C.3 Drilled Shaft under Lateral Load

Drilled shafts are founded in both medium and hard clays. The soil, rock, and concrete properties outlined in previous section are used. Lateral load was applied at the top of drilled shaft.

As mentioned in previous section, pier under lateral load can fail in three modes: ultimate bending moment, excessive lateral deflection, or pier-top rotation. This section will present the results from NIKE3D analyses in order to determine the failure modes of the piers under lateral loads.

Lateral loads and ground-line deflections can be plotted and shown in Figure C-2. From this graph, pier in hard clay deflects less than the one in medium clay at the same load. Load of 155 kips is required to produce the deflection criterion of 1.5 inches for pier in medium clay. For pier in hard clay, it is required 260 kips to reach at 1.5-inch lateral deflections.

Load and shaft-top rotation relationship is shown in Figure C-3. Pier in hard clay rotates less than pier in medium clay. At the loads that produce 1.5-inch deflection, shaft-top rotations of 0.35 and 0.37 are found for piers in medium and hard clays, respectively. These mean the failures of the shafts are not controlled by shaft rotations.

Deformed shapes of drilled shaft in medium clays are shown in Figure C-4. Figure C-5 shows deformed shapes of drilled shaft in hard clay. Deformed shapes have the same pattern for piers in both medium and hard clays. Points of zero deflection or fixity are at the depth of 30 ft below ground surface for both cases.

Soil pressure distributions for piers in medium and hard clays are plotted in Figure C-6 and C-7, respectively. The soil pressures have the same pattern in both medium and hard clays. The maximum soil pressures occur at the depth of 30 ft below ground surface for both cases. Rock pressures were changed to negatives values.

Shear force diagrams can be calculate from soil pressures. Figure C-8 and C-9 show shear force diagrams of pier in medium and hard clay, respectively.

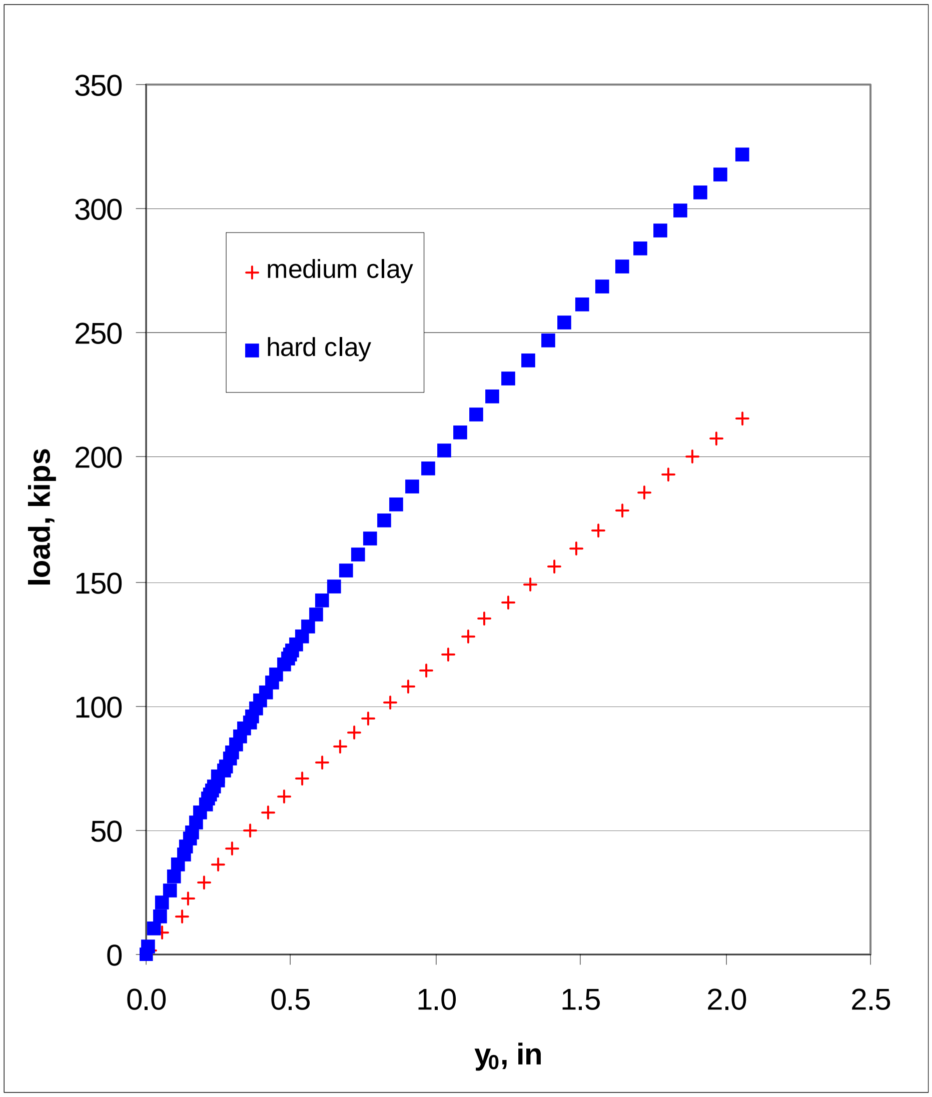


Figure C-2 Ground-line Deflections for Rock Socketed Piers under Lateral Loads

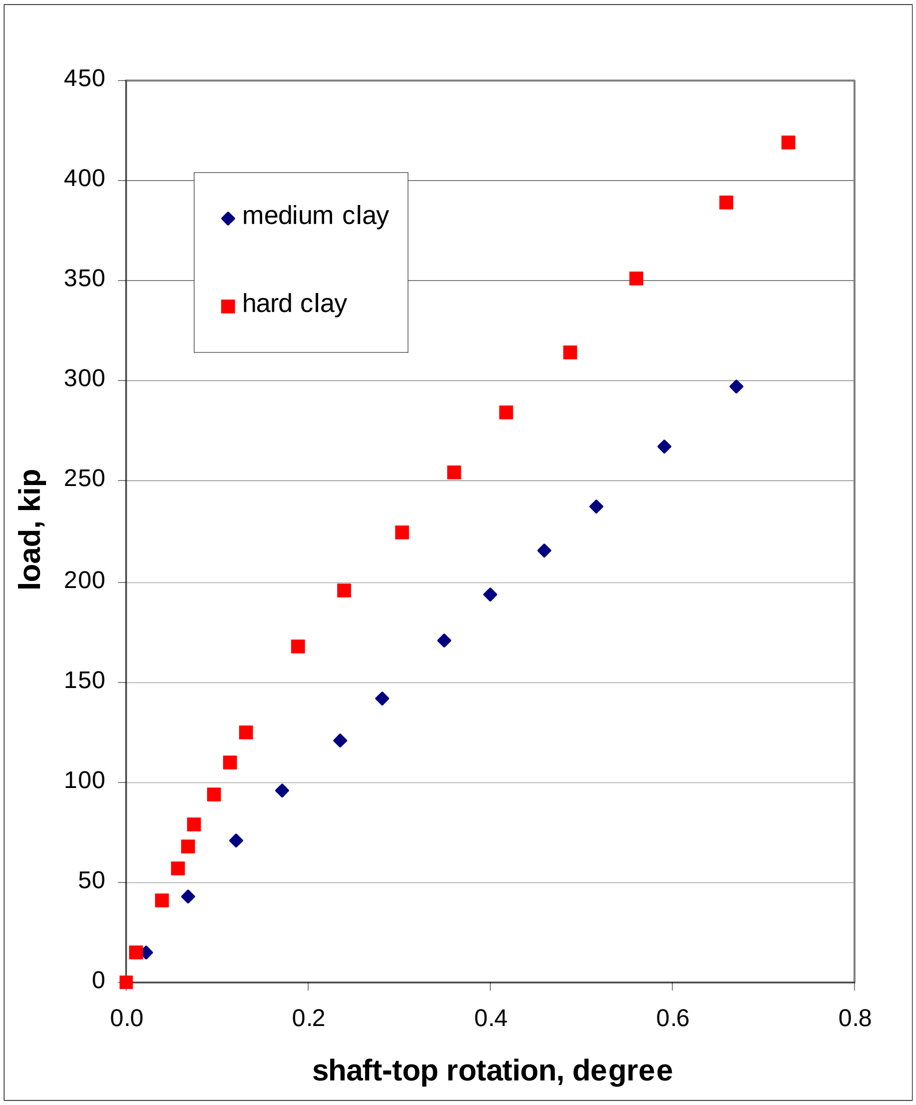


Figure C-3 Shaft-top Rotations for Rock Socketed Piers under Lateral Loads

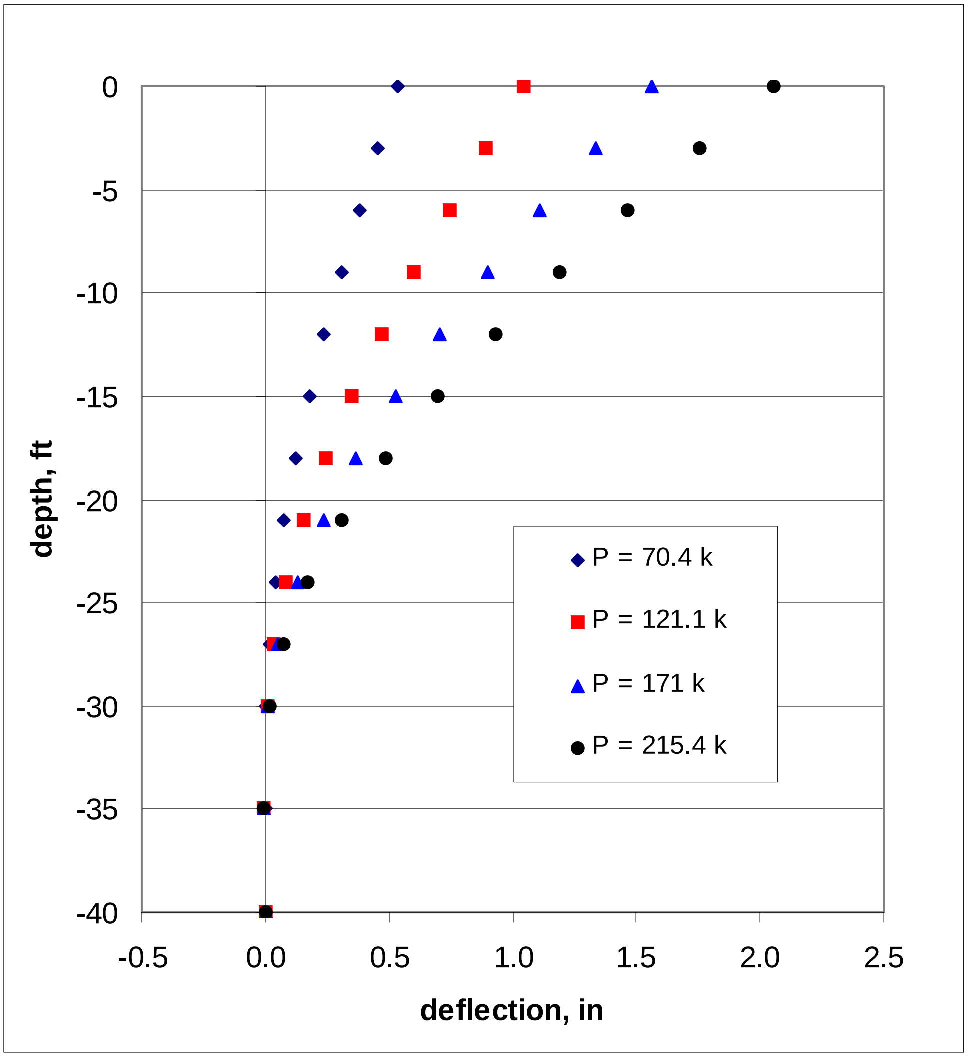


Figure C-4 Deformed Shapes of Drilled Shaft under Lateral Loads in Medium Clay

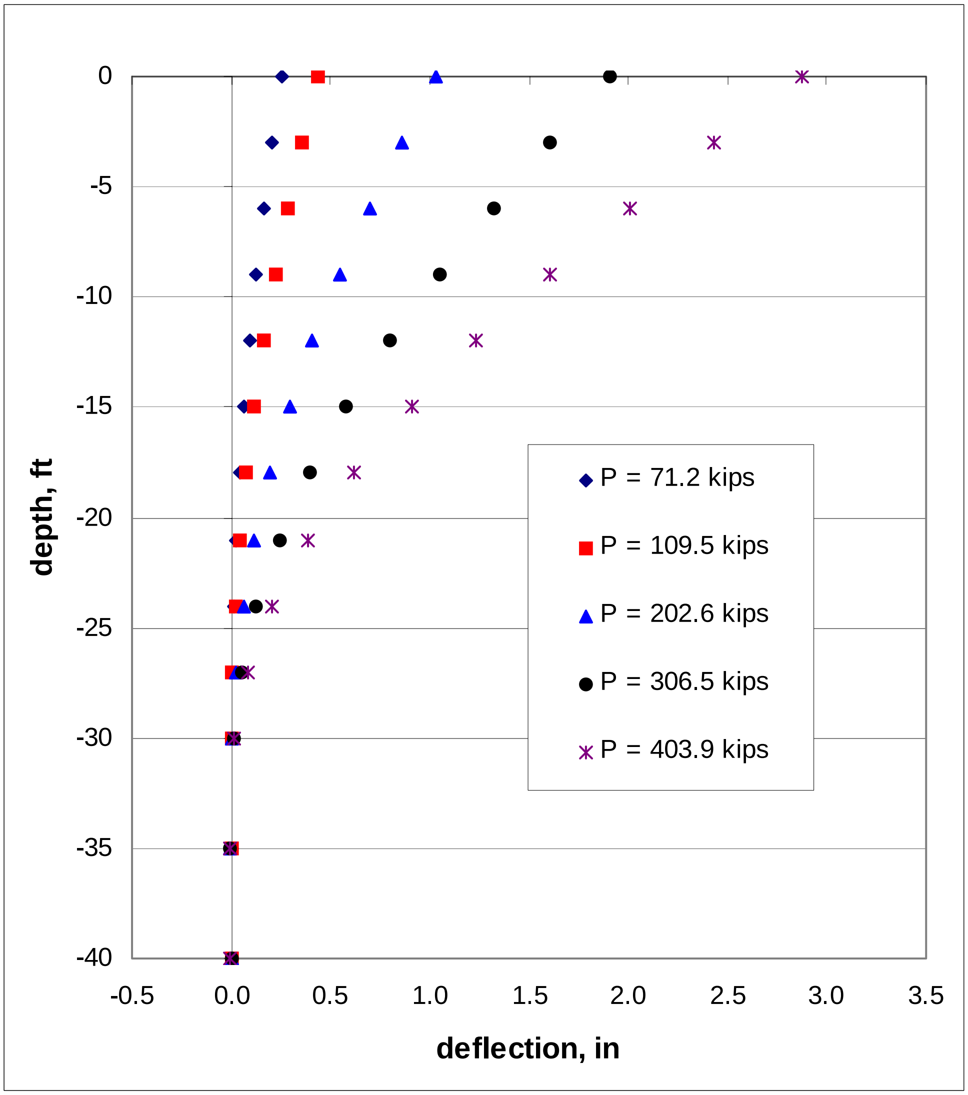


Figure C-5 Deformed Shapes of Drilled Shaft under Lateral Loads in Hard Clays

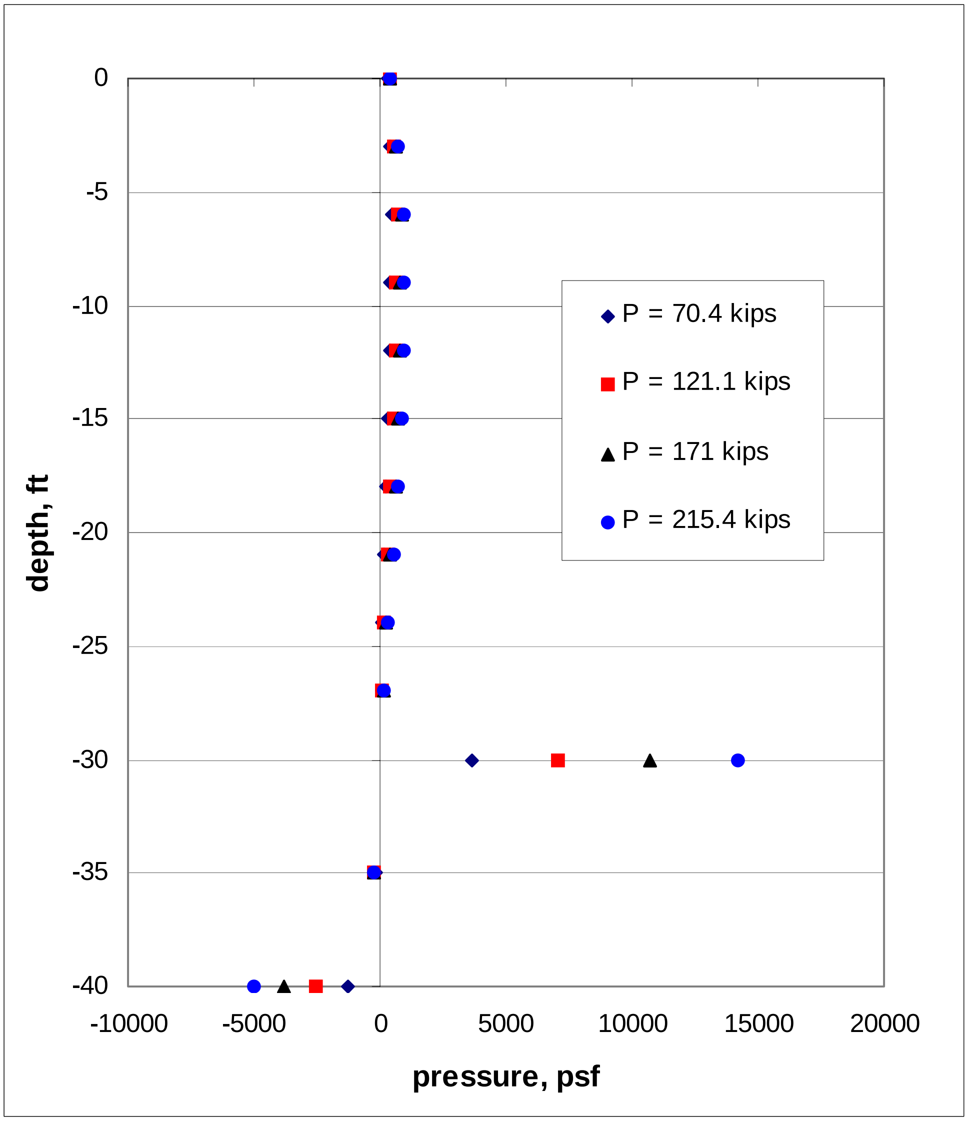


Figure C-6 Soil Pressure Distributions along the Pier under Lateral Loads in Medium Clay

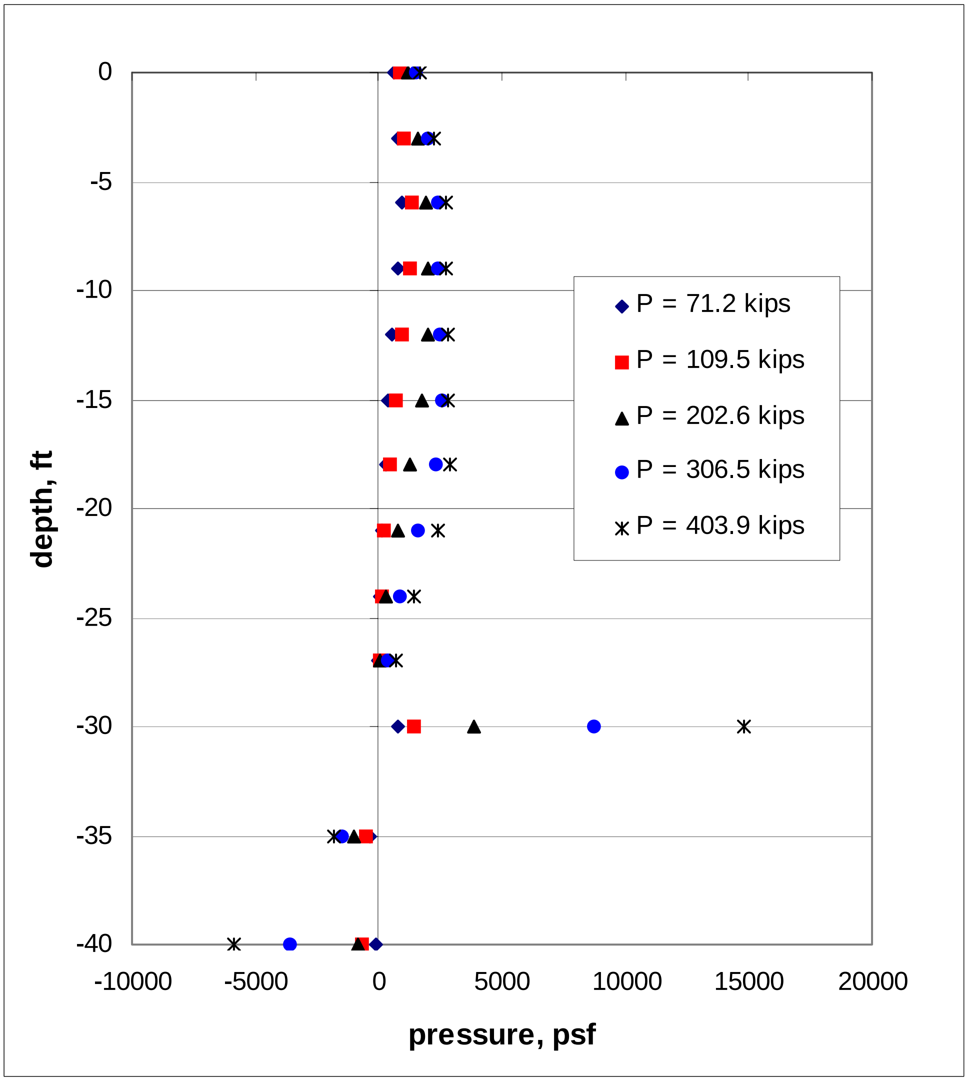


Figure C-7 Soil Pressure Distributions along the Pier under Lateral Loads in Hard Clay

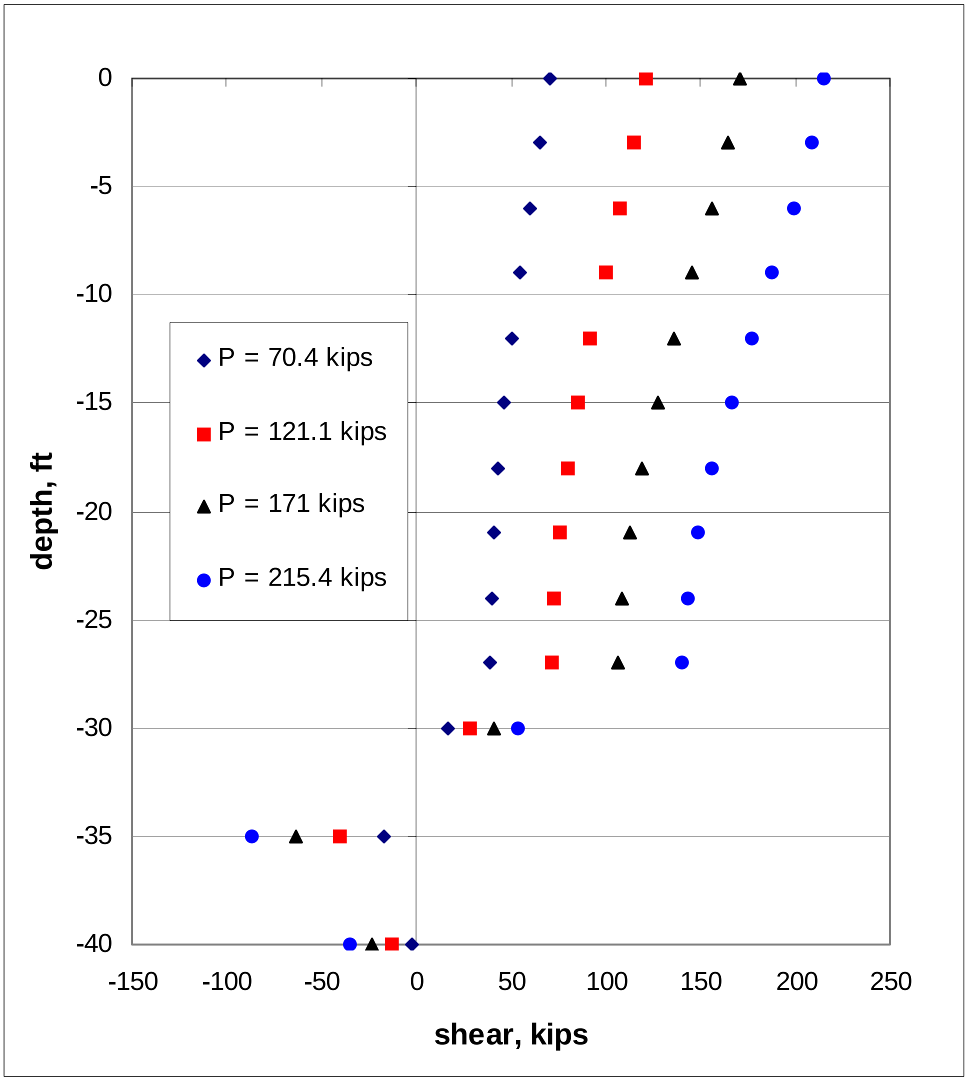


Figure C-8 Shear Force Diagrams of Pier under Lateral Loads in Medium Clay

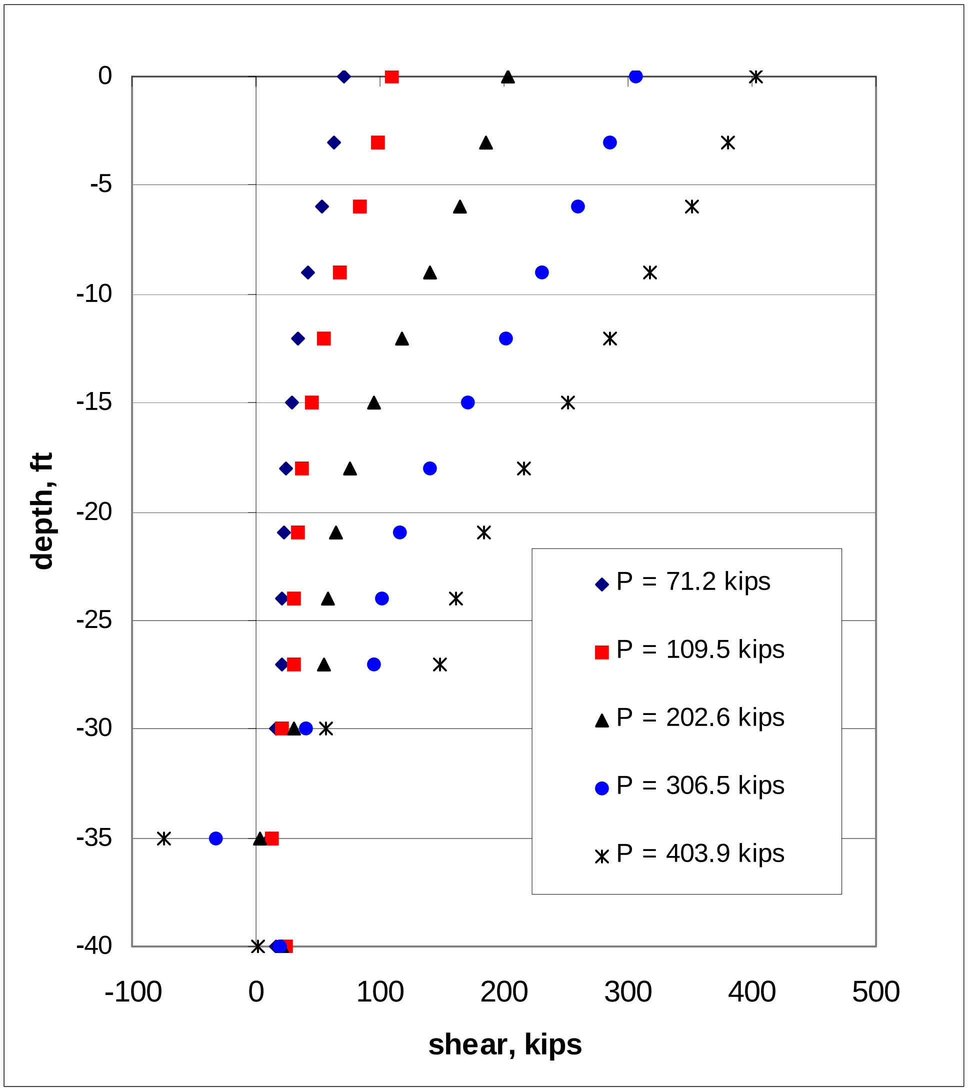


Figure C-9 Shear Force Diagrams of Pier under Lateral Loads in Hard Clay

From Figure C-8 and C-9, the maximum shear forces occur at ground surface and decrease with depth until reaching minimum values at soil/rock interfaces for both piers in medium and hard clay. Shear forces in the rock were changed to negative values.

Bending moment diagrams can be obtained from shear force diagrams as shown in Figure C-10 and C-11 for pier in medium and hard clay, respectively. Bending moments increase from zero values at ground surface to maximum values at soil/rock interfaces and then decrease with the depth of rock

Maximum bending moments occur at points of fixity at soil/rock interfaces for both cases. Loads and maximum bending moments in the piers can be plotted in Figure C-12. The ultimate moment resistance of the pier is 4000 kip-ft. From Figure C-12, loads of 180 and 230 kips produce the maximum bending moments in the piers for medium and hard clay, respectively.

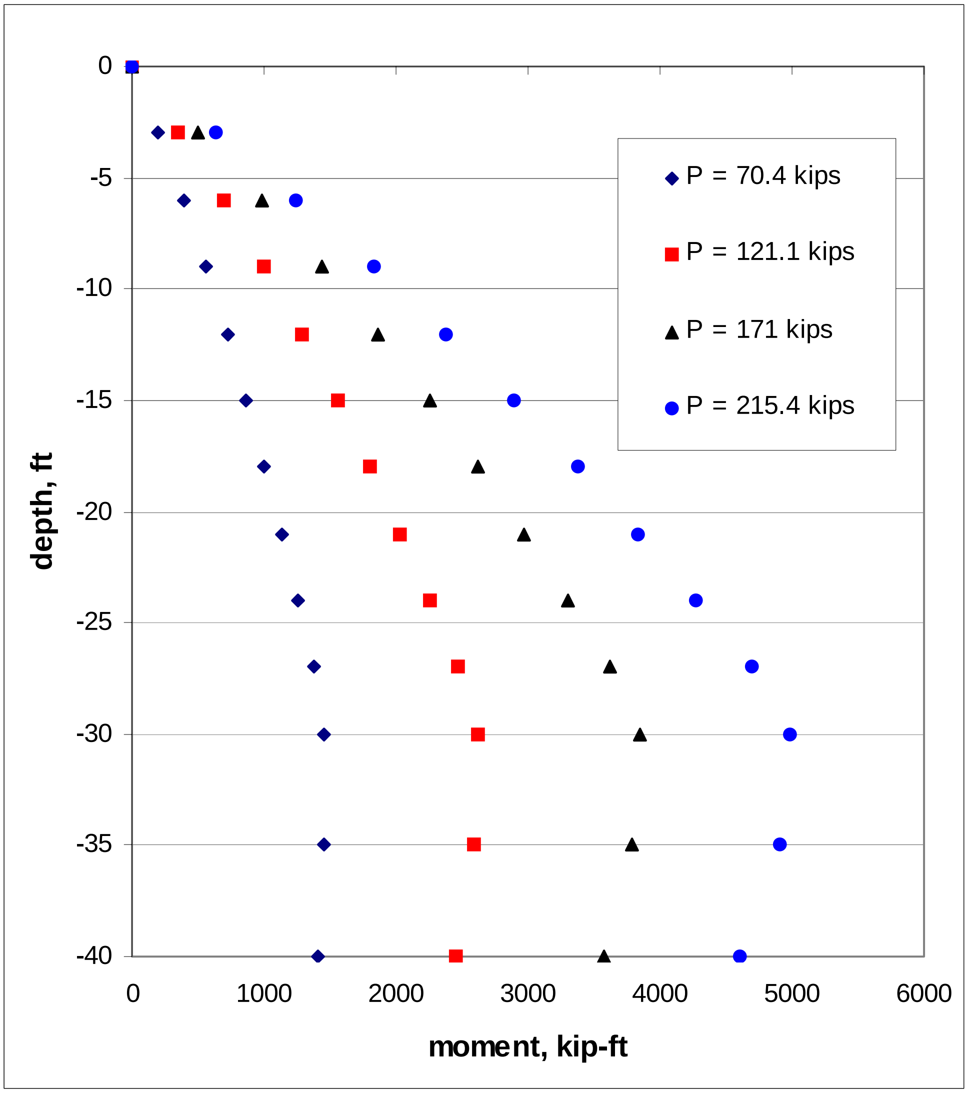


Figure C-10 Bending Moment Diagrams of Pier under Lateral Load in Medium Clay

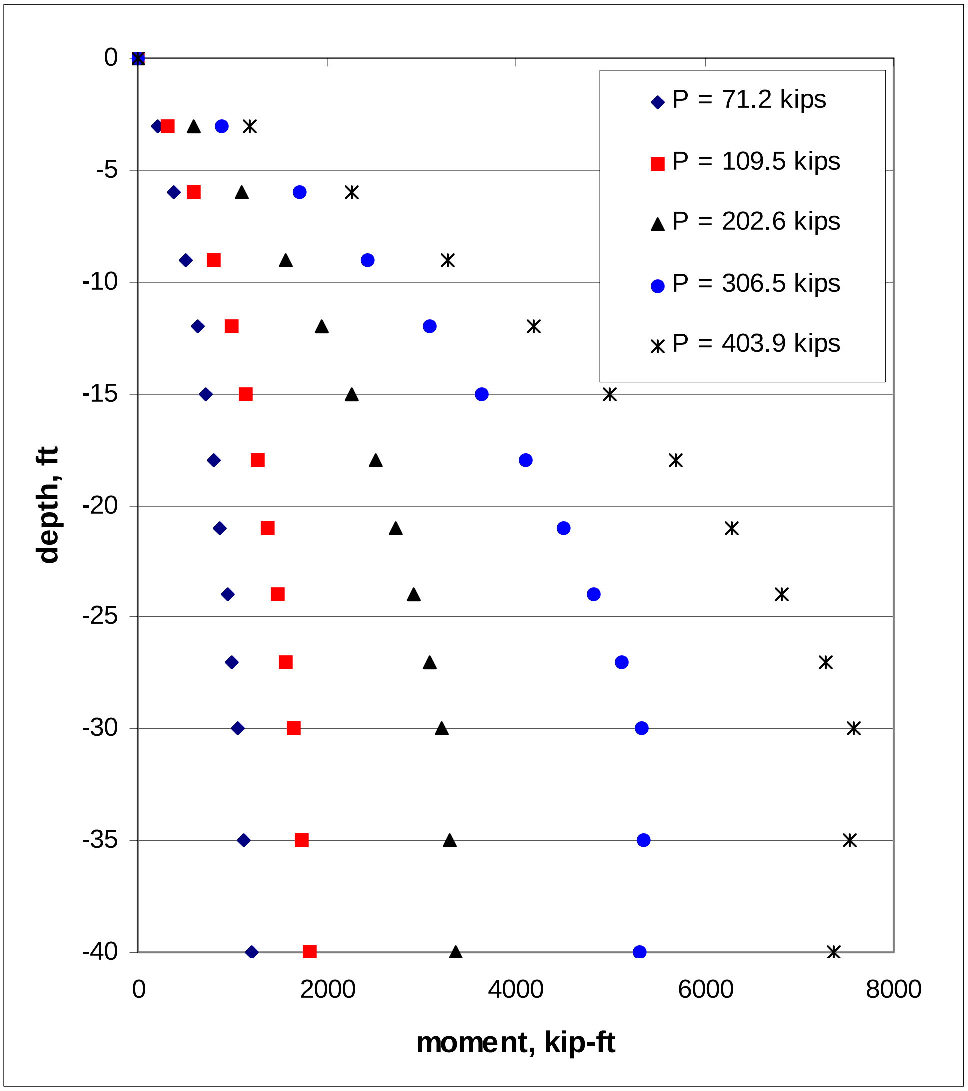


Figure C-11 Bending Moment Diagrams of Pier under Lateral Loads in Hard Clay

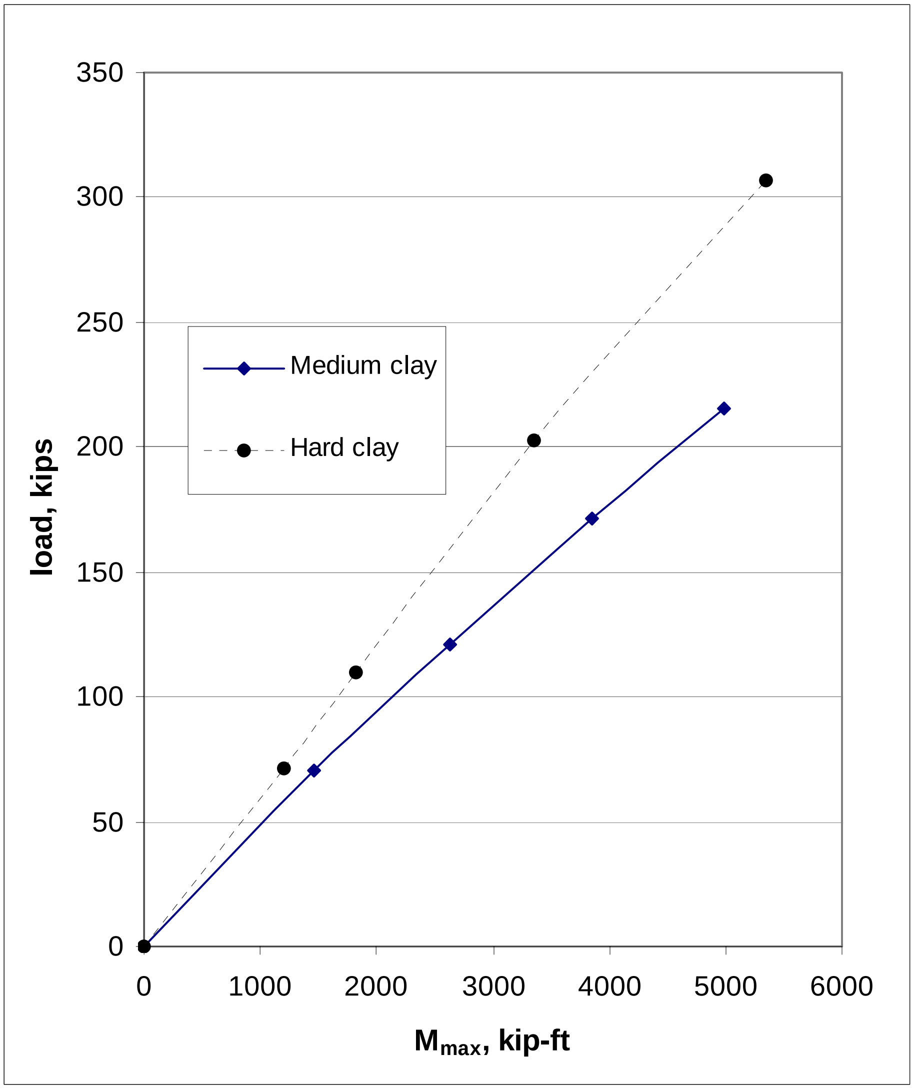


Figure C-12 Maximum Bending Moments in the Piers under Lateral Loads

C.4 Drilled Shaft under Moment

In this section, moment will be applied at shaft-top. Both rock socketed drilled shafts in medium and hard clays were analyzed. Soil, rock, and concrete properties are the same as lateral-load cases.

Failure modes of drilled shafts under moments are same as those under lateral loads. From NIKE3D results, the shaft behaviors can be investigated. Same results similar to piers under lateral loads can be produced for piers under moments.

Ground-line deflections of the piers can be plotted in Figure C-13. At deflection of 1.5 inches, moments of 3700 and 5500 kip-ft are required to produce this deflection for piers in medium and hard clay, respectively. Figure C-14 shows shaft-top rotations for piers under moments. For the same moments that produce the 1.5-inch deflection, the rotations of 0.45 and 0.55 degree are found for medium and hard clay, respectively.

Deformed shapes of piers in medium and hard clay are shown in Figure C-15 and C-16, respectively. From figure C-15, it is found that points of zero deflection are at the depth of 32 ft below ground surface. For pier in hard clay, the points of zero deflections are in between 28 to 32 ft below the ground surface.

Soil pressure distributions along the piers are plotted in Figure C-17 for pier medium clay and Figure C-18 for pier in hard clay. The distribution patterns are the same for piers under lateral loads and those under moments. The maximum soil pressures occur at soil-rock interfaces for both piers under moments and lateral loads.

Figure C-19 and C-20 show shear force diagrams of pier in medium and hard clay, respectively. Shear forces increase from zero at ground surface to the maximum values at the depth of 32 ft below ground surface for pier in medium clay. For pier in hard clay, the same distributions can be seen. The maximum shear forces occur at depths of 28 to 32 ft below ground surface for pier in hard clay.

Bending moment diagrams of piers in medium and hard clay are shown in Figure C-21 and C-22, respectively. Bending moment diagrams for both cases have the same pattern. The maximum bending moments at ground surface decrease nonlinearly with depth to minimum values at pier tips. The maximum bending moments equal to applied moments at pier top.

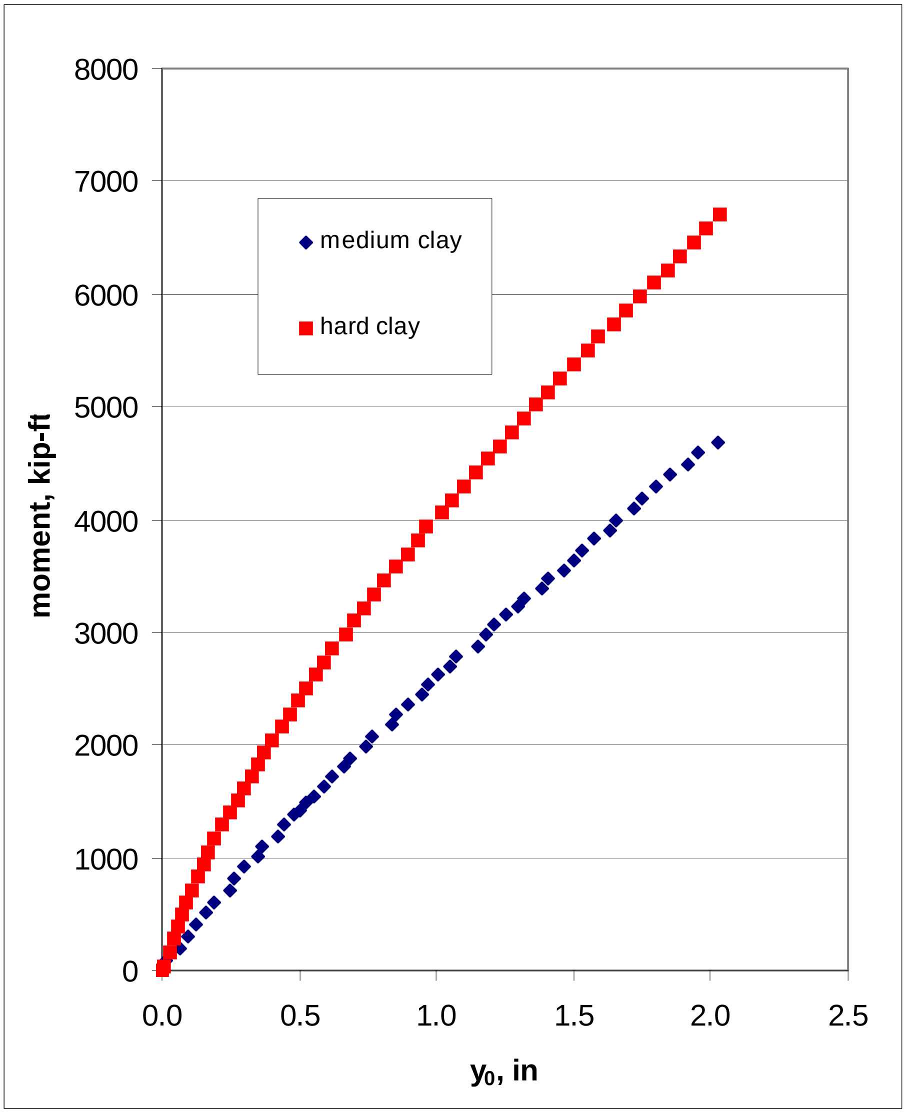


Figure C-13 Ground-line Deflections of the Piers under Moments

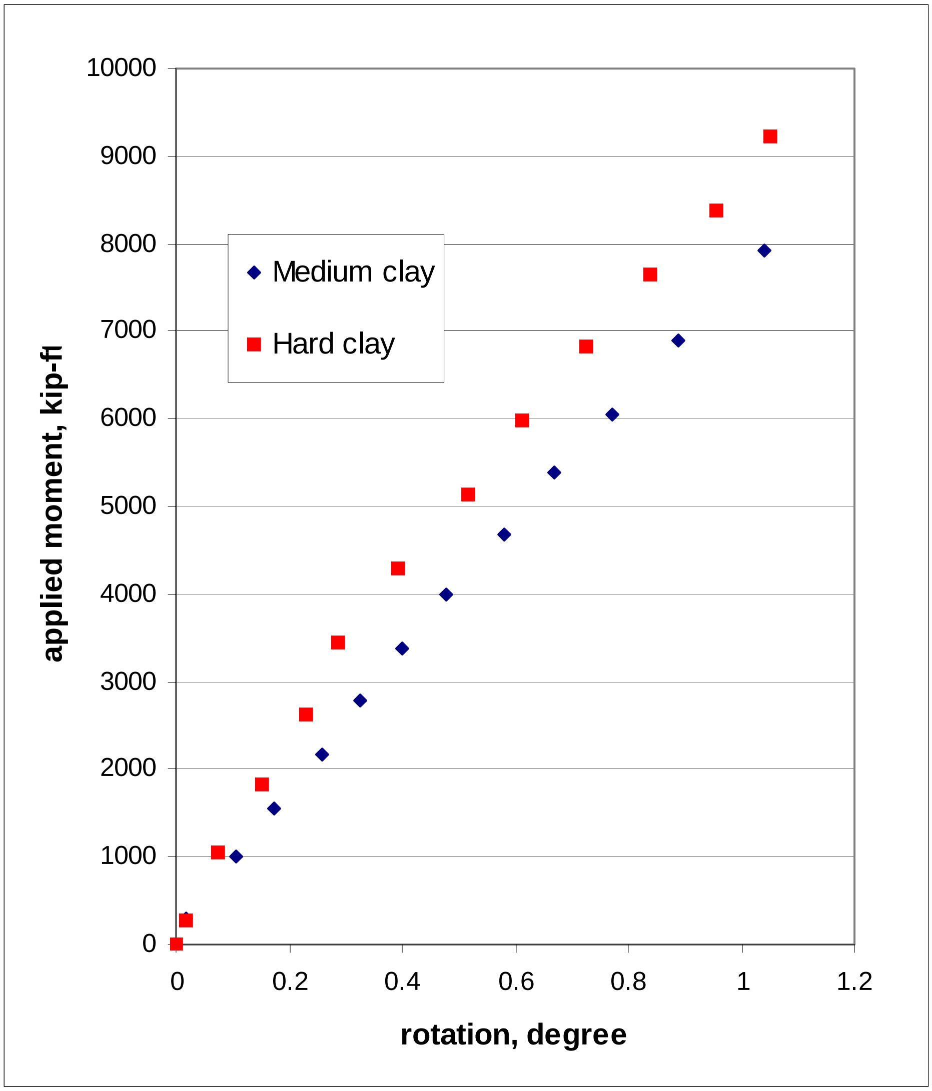


Figure C-14 Shaft-top Rotations for Piers under Moments

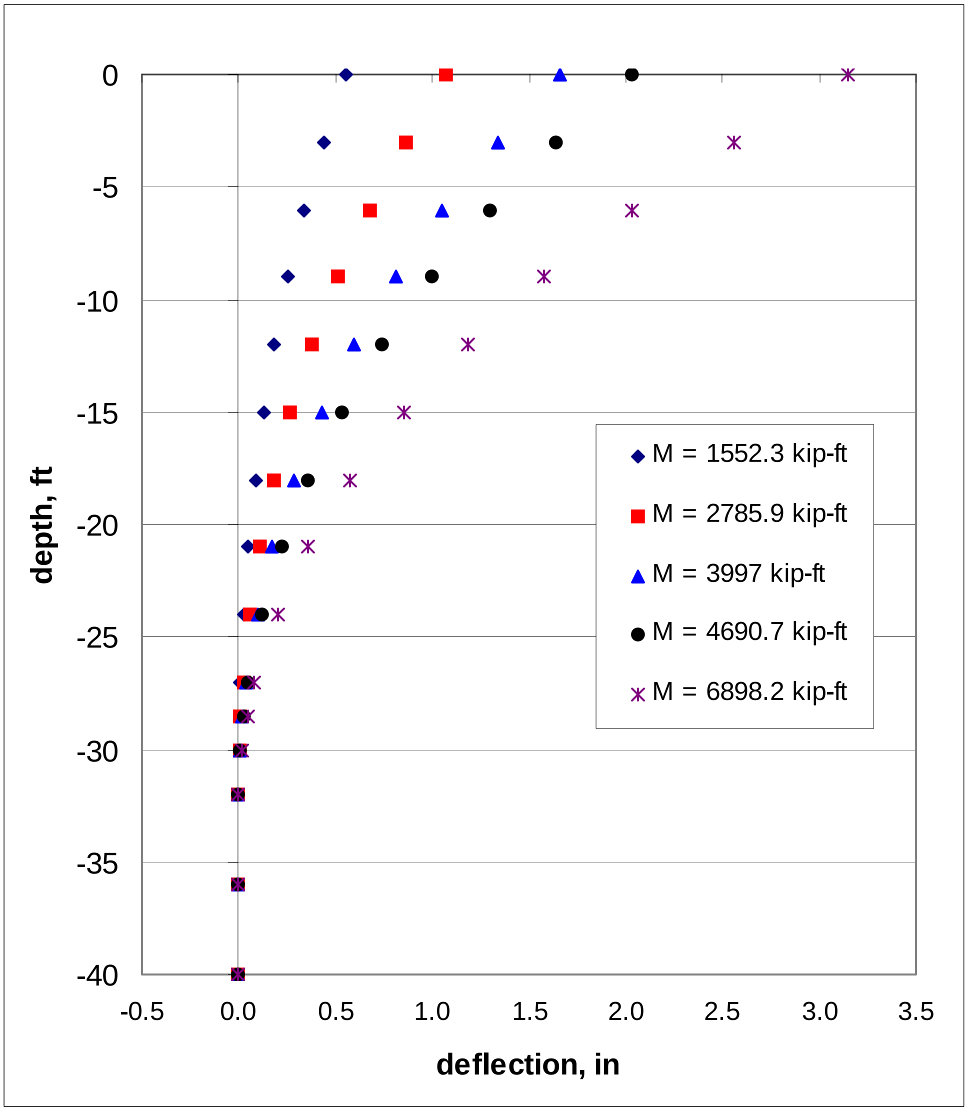


Figure C-15 Deformed Shapes of Pier under Moments in Medium Clay

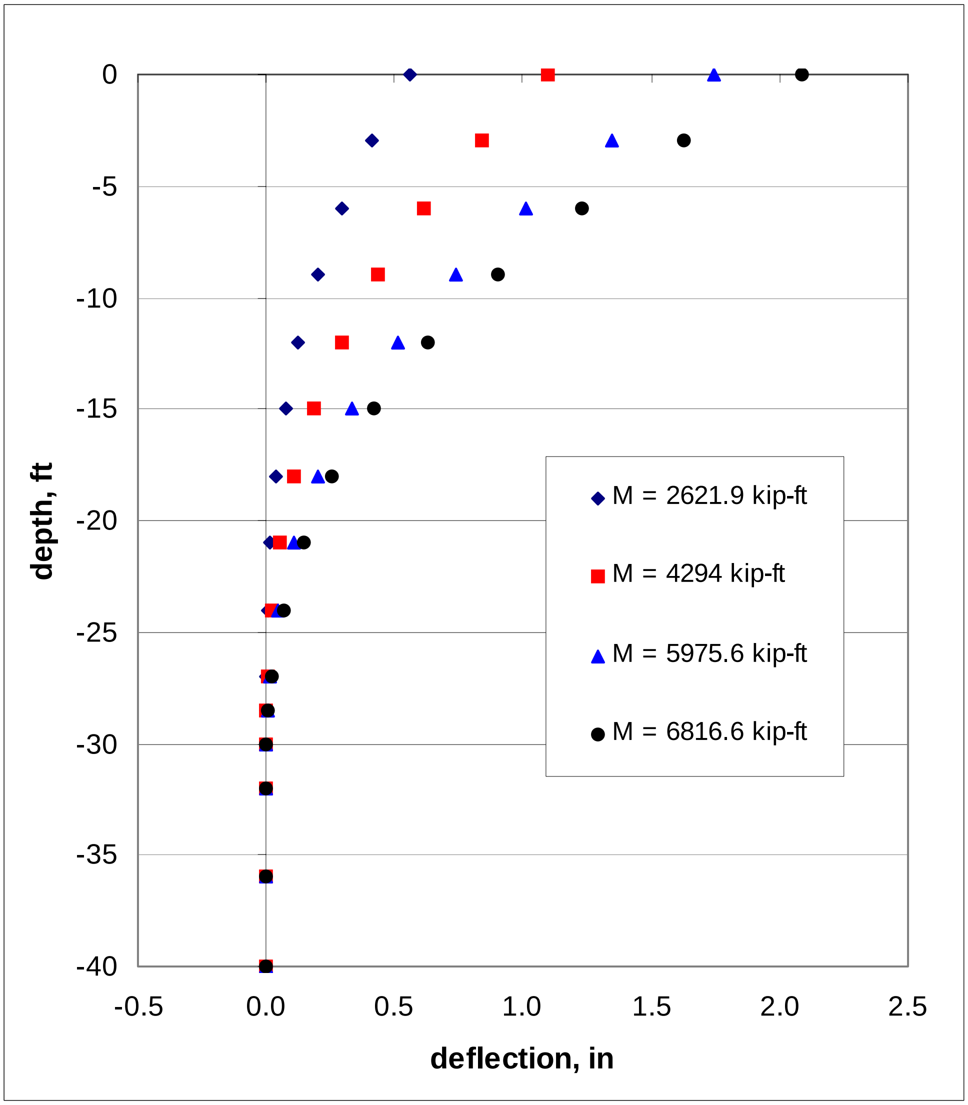


Figure C-16 Deformed Shapes of Pier under Moments in Medium Clay

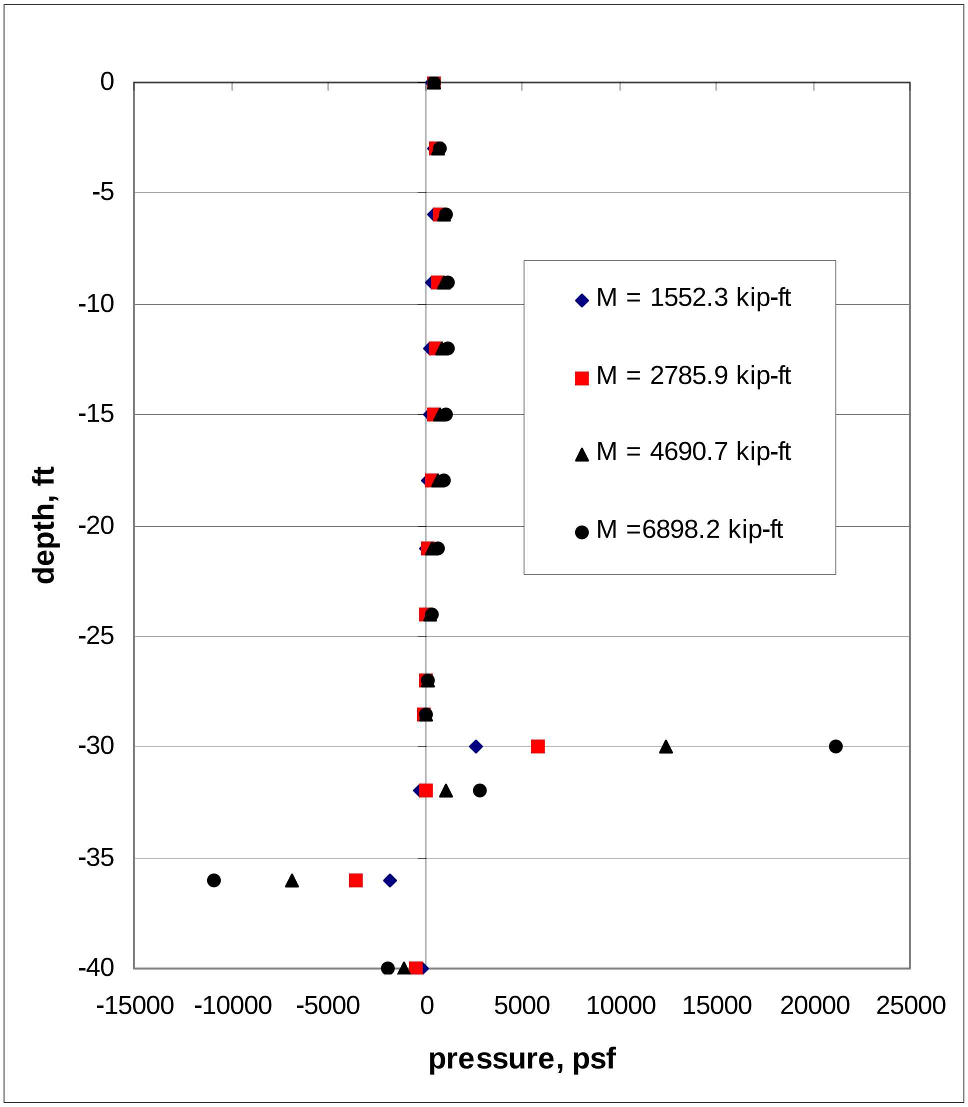


Figure C-17 Soil Pressure Distributions along the Pier under Moments in Medium Clay

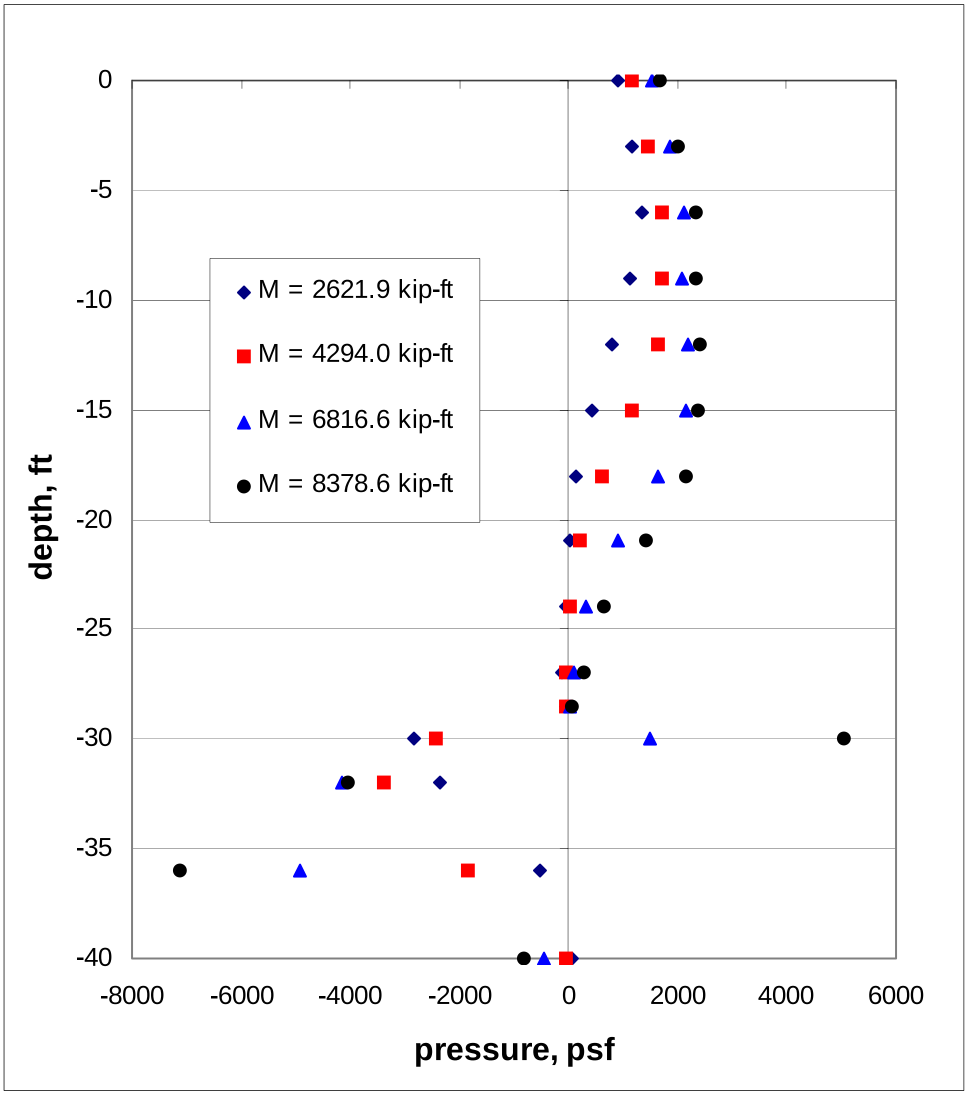


Figure C-18 Soil Pressure Distributions along the Pier under Moments in Hard Clay

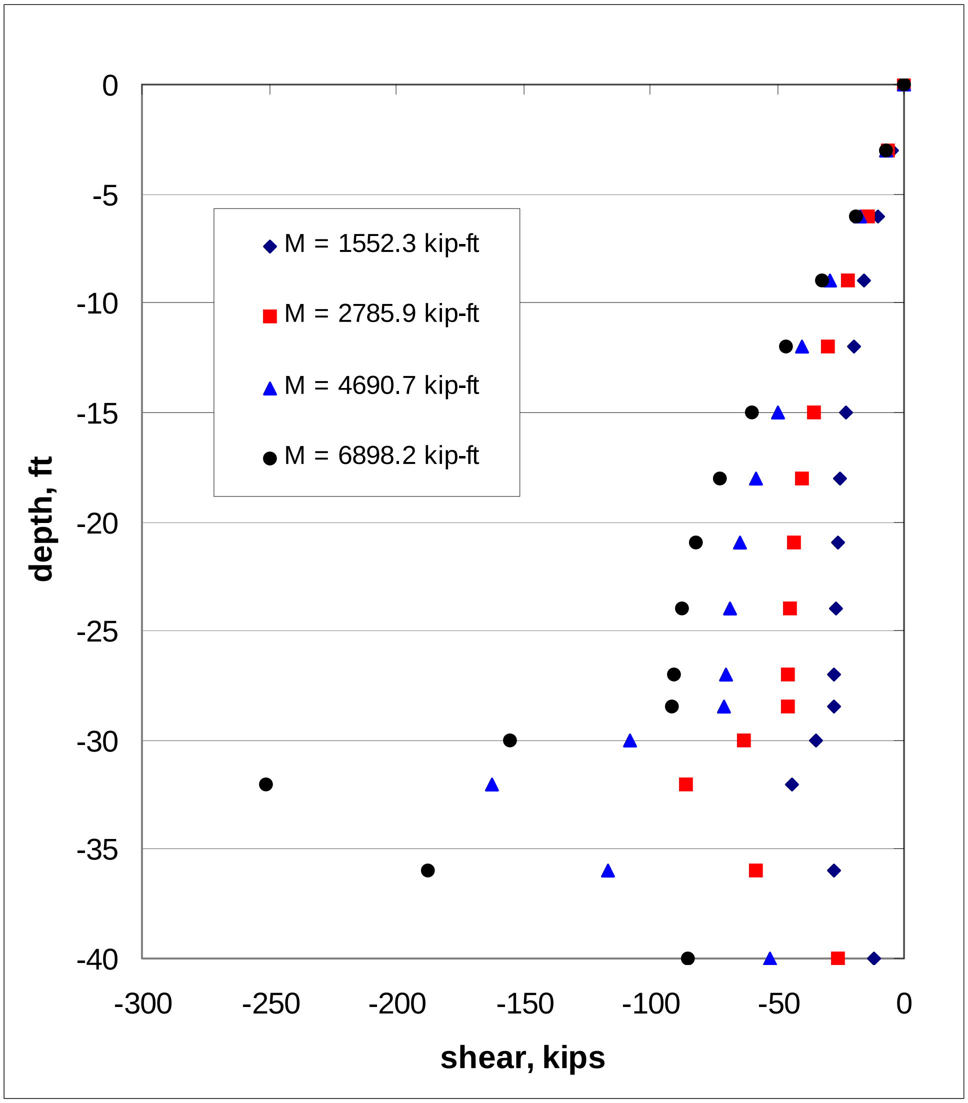


Figure C-19 Shear Force Diagrams of Pier under Moments in Medium Clay

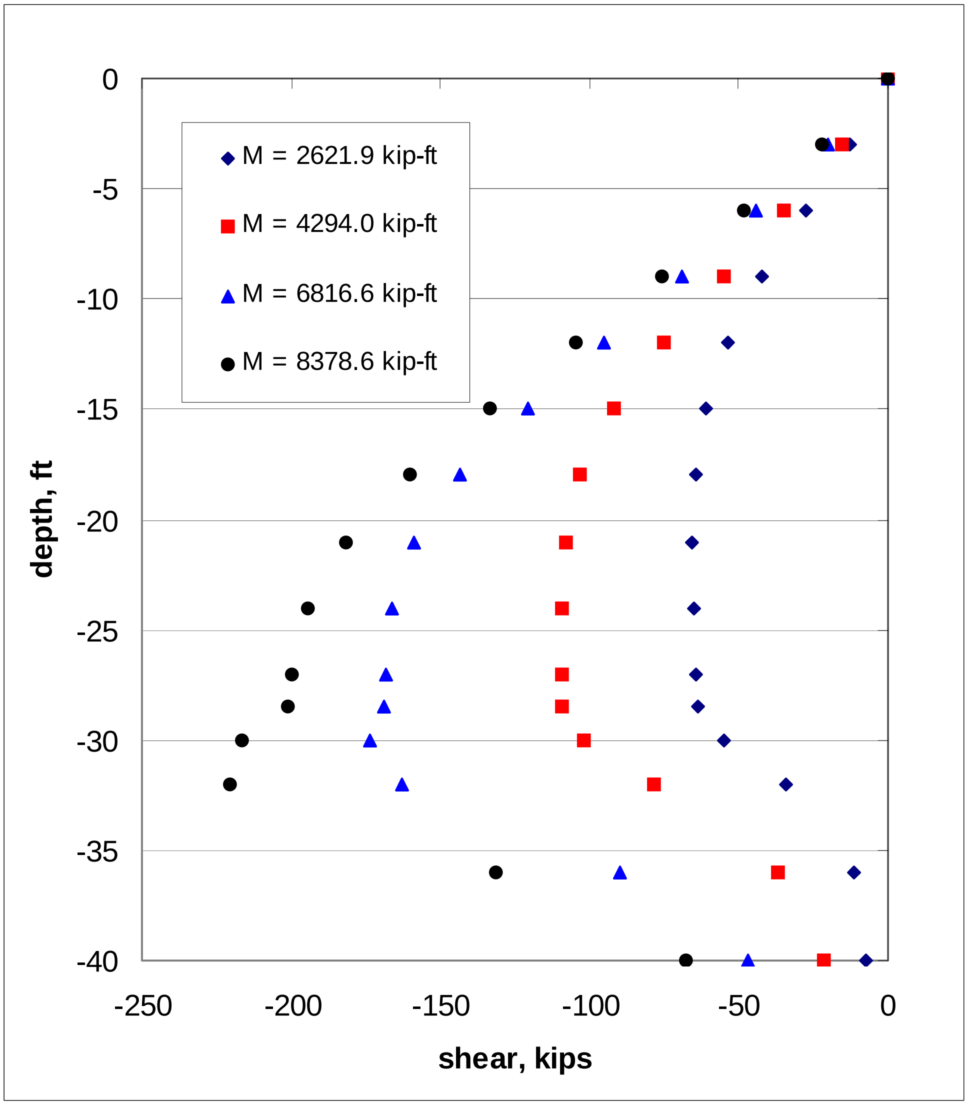


Figure C-20 Shear Force Diagrams of Pier under Moments in Hard Clay

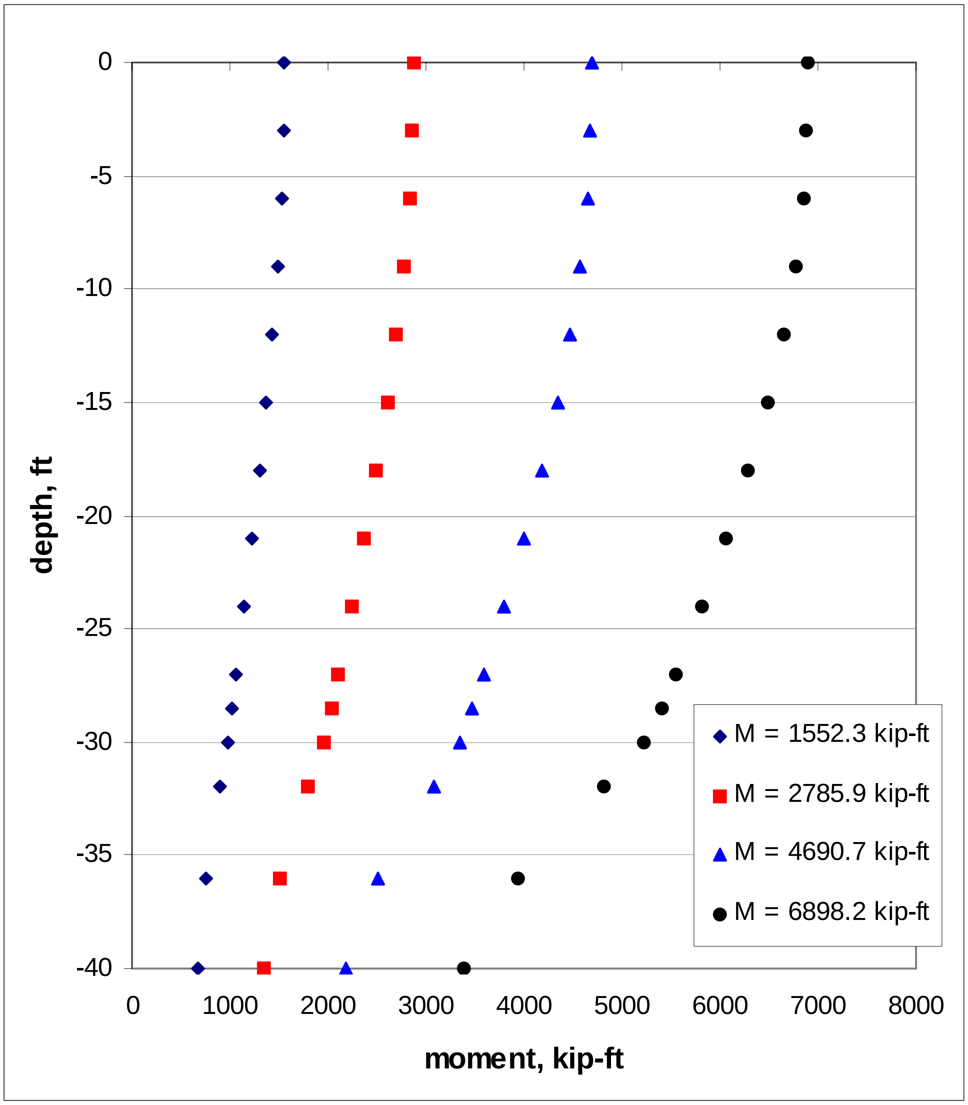


Figure C-21 Bending Moment Diagrams of Piers under Moments in Medium Clay

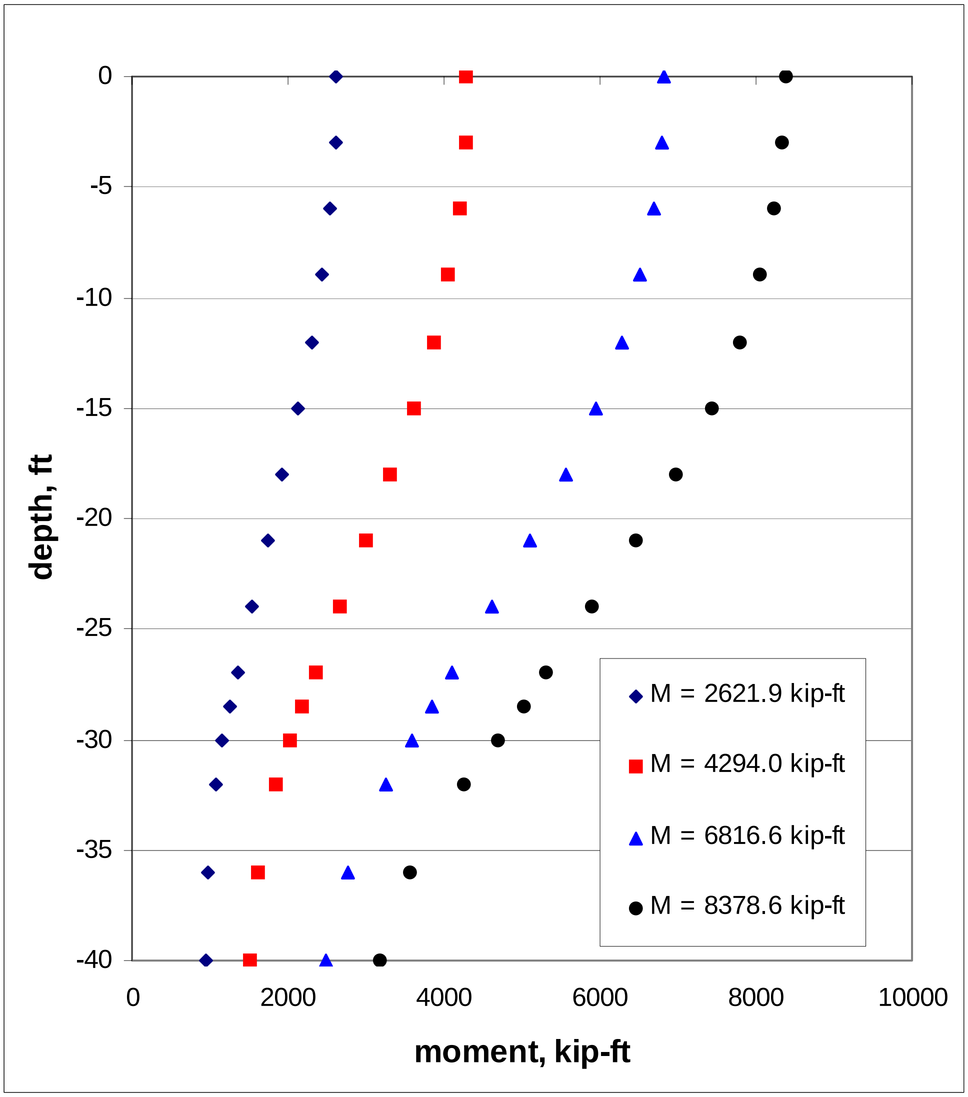


Figure C-22 Bending Moment Diagrams of Piers under Moments in Hard Clay

C.5 Shaft under Torsion

Torsions were applied at the top of drilled shafts. Drilled shafts under torsions could be failed by maximum shear stresses in shafts, maximum shear stresses in the soil, and excessive axial rotation of the shafts.

This section will be devoted to investigate the behavior of rock socketed drilled shafts under torsions. Total of five piers under torsions were investigated. Four cases were analyzed by NIKE3D. The first two cases (interfaced piers) contained interfaces between clay (or rock) and piers. The second two cases (glued piers) glued clay (or rock) and piers together. The last pier was fixed-end pier under torsion without soil (or rock) and analyzed using mechanics. This pier is shown in Figure C-23.

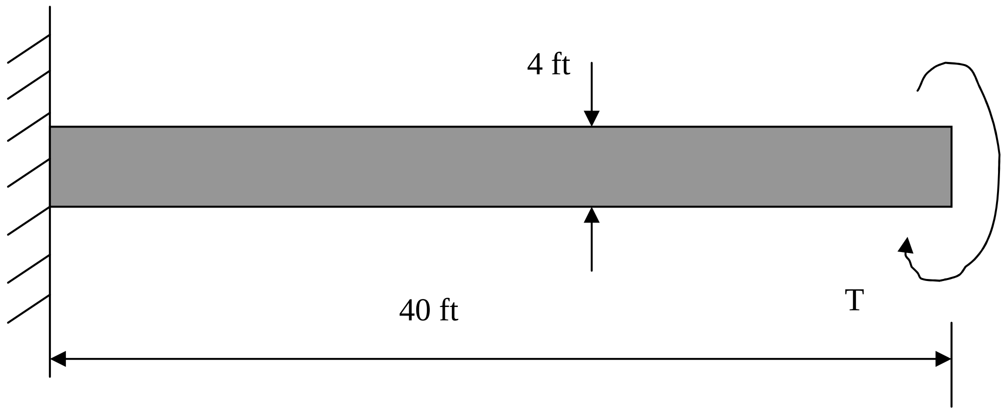


Figure C-23 Fixed-end Pier under Torsion without Soil

Axial rotations of piers under torsions can be shown in Figure C-24. There is no difference between interfaced pier in medium and hard clays. Results from both glued cases are the same. Fixed-end pier serves as lower bound of the rotations. This case gives the highest rotation for the given torsion. Glued piers give the lowest rotation for the same torsion. For the rotation of 1.0 degree, fixed-end pier requires torsion of 3000 kip-ft. Torsions of 3400 and 3800 kip-ft are required to product one-degree rotation for interfaced and glued piers.

Figure C-25 shows the maximum shear stresses in the piers for all cases. For the limiting shear stress of 24,400 psf, fixed-end pier shows the lowest required torsion of 307 kip-ft. The glued piers required the highest torsion of 3700 kip-ft in order to produce the limiting shear stress. For interfaced piers, torsion of 1950 kip-ft can produce the limiting value of 24,400 psf.

From NIKE3D glued pier results, the maximum shear stresses in soil occur at ground surface. The maximum shear stresses decrease drastically with the distance from pier surface. The maximum shear stresses in the soil can be plotted along the distance from pier surface and shown in Figure C-26.

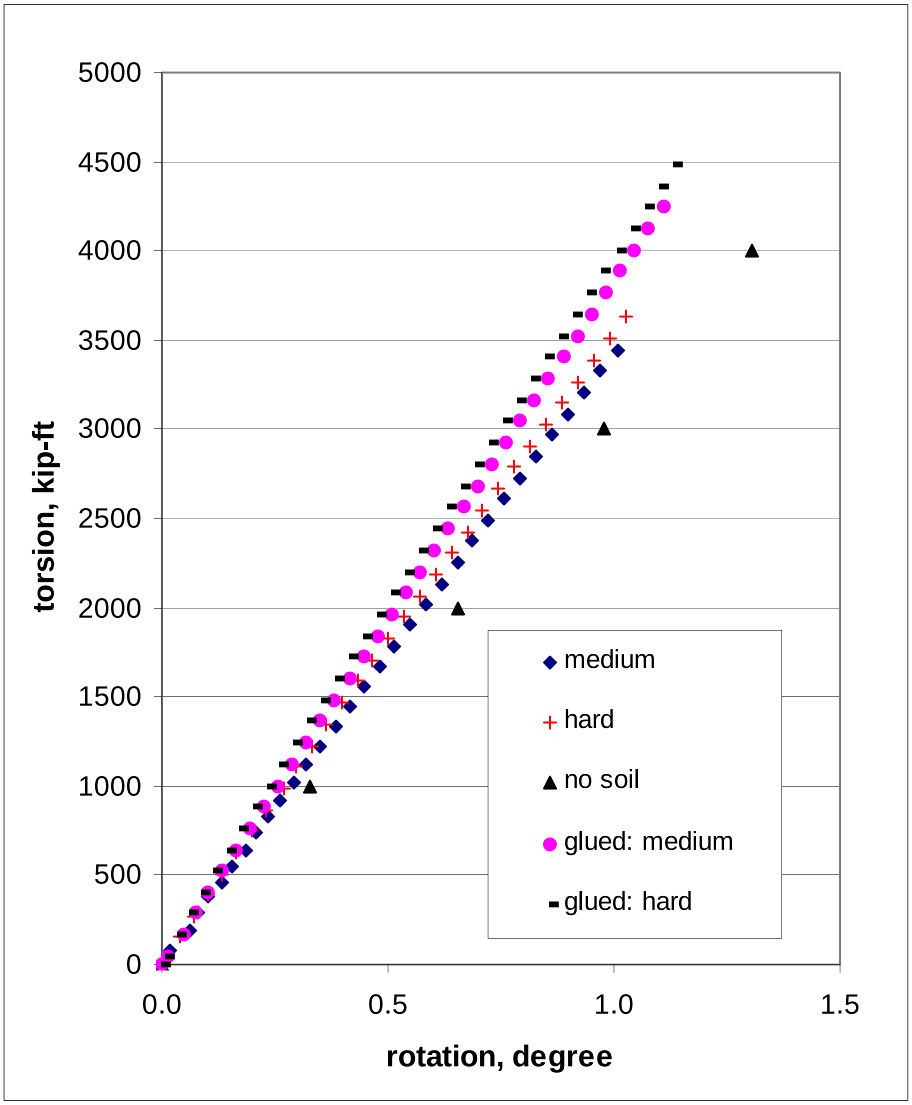


Figure C-24 Axial Rotations of Piers under Torsions

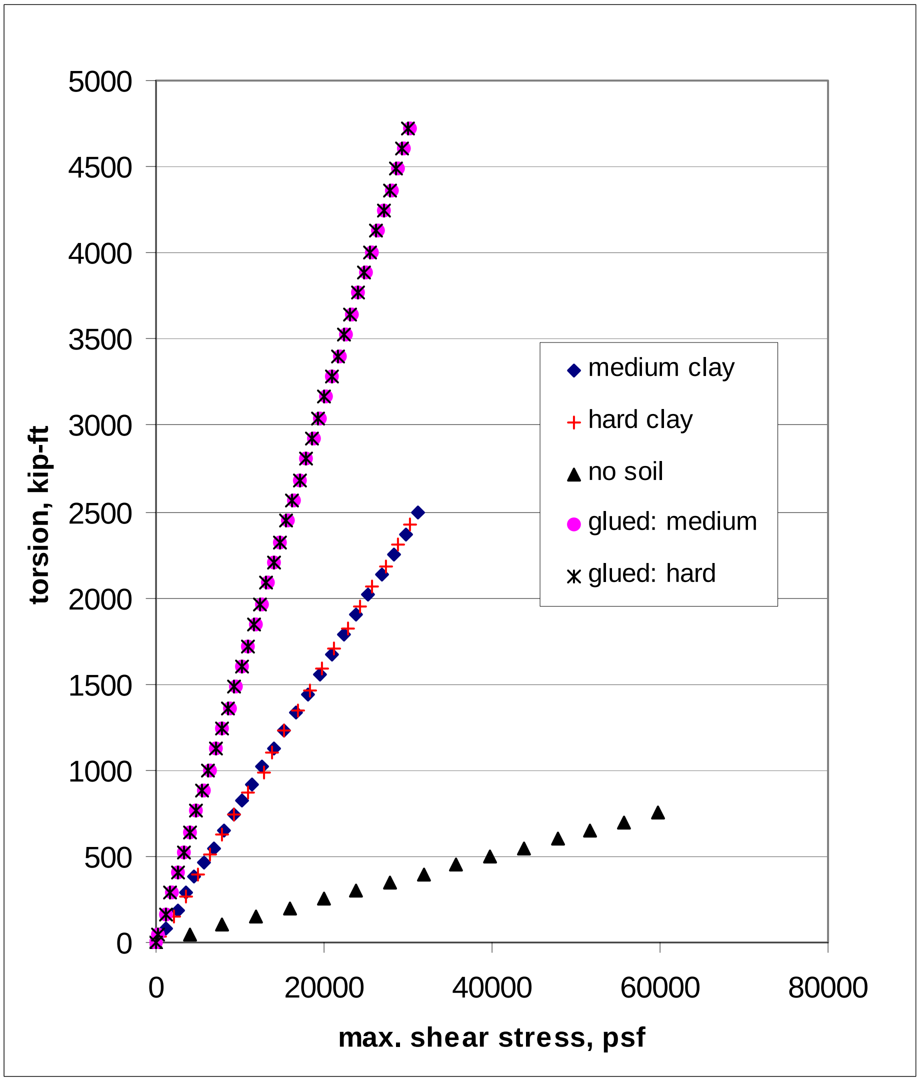


Figure C-25 Maximum Shear Stresses in Piers under Torsions

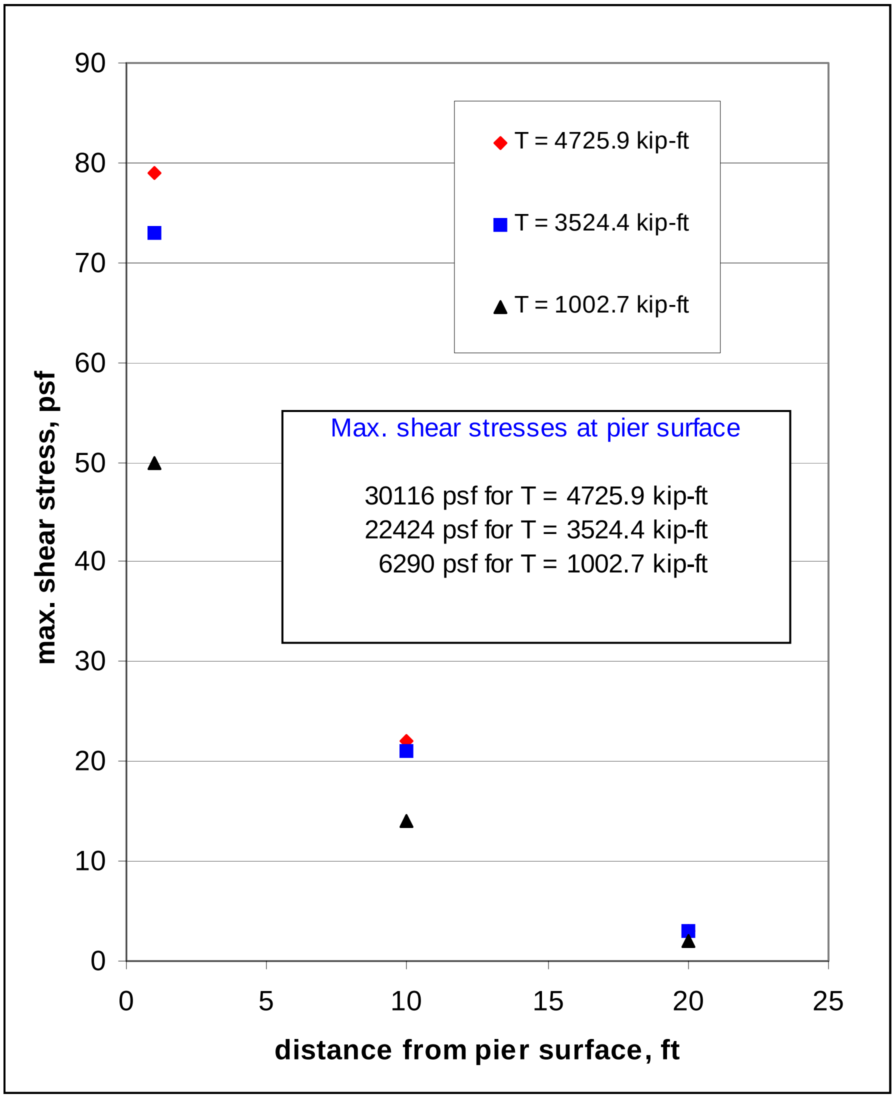


Figure C-26 Maximum Shear Stresses in the Soil along the Radial Direction for Glued Pier

The maximum shear stresses in soil along the depth for glued pier can be plotted in Figure C-27. Figure C-27 shows that the maximum values at ground surface decrease to very little values at depth of 12 ft below ground surface.

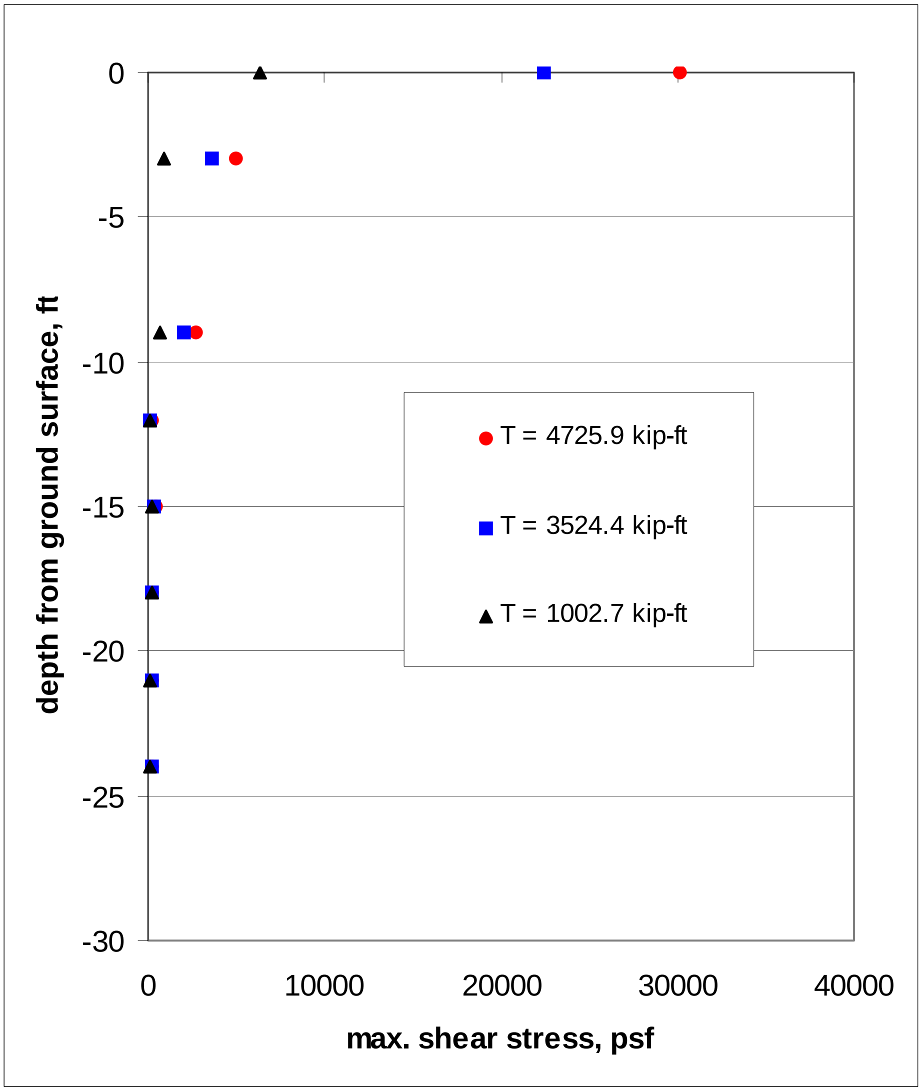


Figure C-27 Maximum Shear Stresses in the Soil along the Depth for Glued Pier

C.6 Conclusions

This chapter presented the results of analyses of rock socketed drilled shafts under lateral load, moments, and torsions, separately. The shaft has 4-ft diameter and 40-ft length with the bottom 10 ft embedded in the rock Shaft, soil, and rock properties were given.

For shafts under lateral loads, ground-line deflections, shaft-top rotations, deformed shapes, soil pressures, shear force diagrams, and bending moment diagrams were obtained. The failure mode for drilled shaft under lateral load in medium clay was excessive lateral deflection at failure load (P_f) of 170 kips. The failure mode for drilled shaft under lateral load in hard clay was maximum bending moment with the failure load (P_f) of 230 kips.

For shafts under moments, the same results as those under lateral loads were observed. The failure mode for shaft under moment in medium clay was excessive lateral deflection at failure moment (M_f) of 3700 kip-ft. For drilled shaft in hard clay, it failed by maximum bending moment (M_f) of 4000 kip-ft.

Axial rotations and maximum shear stresses in the piers for frictional interface, glued interface, and fixed-end piers were given. The maximum shear stresses in the soil at different radial distances and depths for glued piers are also provided. For drilled shafts under torsions in both medium and hard clay, shafts failed by maximum shear stresses in concrete at failure torsions (T_f) of 1950 kip-ft. Table C-5 shows the capacities of drilled shafts under different loads.

Table C-5 Capacities of Drilled Shafts under Different Loads

Failure Mode for	**Shafts in Medium Clay**	**Shafts in Hard Clay**
Lateral Loads	Lateral deflection at P_f = 170 kips	Max. moment at P_f = 230 kips
Moments	Lateral deflection at M_f = 170 kips	Max. moment at M_f = 230 kips
Torsions	Max. shear stress at T_f = 1950 kip-ft	Max. shear stress at T_f = 1950 kip-ft

For drilled shafts under lateral loads or moments in clays, shaft capacities depend on strength and stiffness of clays. Shaft capacities under torsions are independent of strength and stiffness of clays but depend on coefficients of friction between soil and pier.