EVALUATION OF SHORT, LARGE DIAMETER PILES FOR ARCTIC APPLICATION

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PROJECT SUMMARY

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Program Office	Proposal No.	Topic No.

TO BE COMPLETED BY PROPOSER

Name and Address of Proposer

The Earth Technology Corporation
3777 Long Beach Boulevard, Long Beach, CA 90807

Name and Title of Principal Investigator

Mr. Lino Cheang, Project Engineer

Title of Project

Evaluation of Short, Large-Diameter Piles for Arctic Applications

Topic

SubTopic

Technical Abstract (Limit to two hundred words)

Multiple large-diameter piles (spuds) can be effective as part of a foundation system for gravity structures in the Arctic. The main purpose of the spuds is to help transfer high lateral ice loads to more competent subsurface soils. There are currently no design guidelines for spuds. Test results and design criteria for long piles are not applicable because they do not involve significant lateral soil deformation and resistance near the pile tip. A work plan consisting of literature survey, analysis and evaluation of the current state-of-knowledge of spud behavior has been performed.

Keywords (8 max) Description of the Project, Useful in Identifying the Technology, Research Thrust and/or Potential Commercial Application

Arctic, Pile Foundation, Gravity Structure, Lateral Resistance

Anticipated Results/Potential Commercial Applications of the Research

A literature survey was performed to review the current state-of-knowledge on spud behavior. Analysis was performed to investigate the sensitivity of critical design parameters for spud foundations. Evaluation of available analytical tools for spud design and some recommendations on design guidelines wereprovided. Future studies was recommended to improve the understanding of spud behavior.

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ABSTRACT

The Minerals Management Service of the U.S. Department of Interior authorized The Earth Technology Corporation to perform a research study on the behavior of short, large-diameter piles (spuds) under lateral loading. A spud foundation was recently proposed for increasing the lateral capacity of a gravity structure in the Beaufort Sea. Spuds were proposed for transferring lateral ice loads on a gravity structure seated on soft surficial soils underlain by permafrost.

Current technology regarding pile or gravity structures alone cannot satisfactorily evaluate the problem. Available pile technology is derived based on data obtained for long slender piles and these procedures do not consider the interaction of the structure base; current practice for the analysis of gravity structures cannot account for the added resistance of the spuds.

In order to evaluate the structure-spud-soil interaction problem, a sequence of specific tasks were accomplished and documented in this report:

(1) Literature survey

(2) Development of analytical procedure for a single spud

(3) Two dimensional finite-element analysis to study skirt-structure-soil interaction

(4) Recommendation of simplified design guidelines

Future studies were also recommended, including centrifuge testing and three-dimensional finite-element analysis.

1.0 INTRODUCTION

1.1 Statement of Problem

Application of short, large-diameter steel piles (spuds) has been proposed as a means to increase the lateral load carrying capacity of a foundation system. A spud is different from a long slender pile because the length to diameter ratio of a spud is less than about 5; whereas for long slender piles, this ratio is usually bigger than 10. The fundamental difference between the response of a long slender pile and spud under lateral loading for various structural connections can be illustrated in Figure 1-1. As shown in this figure, the deflected shape of a long slender pile and a spud under lateral loading is quite different for each of the three boundary conditions: free, restrained and fixed-head. The deflection of a long pile is usually negligible at the pile tip, whereas significant tip deflection is associated with spuds. Due to this reason, most of the soil resistance is mobilized at shallow depth for the long pile foundation. Spud foundations, however, usually developed significant soil resistance at the deeper soils (usually stronger) near the spud tip.

A spud foundation is potentially useful for coastal and offshore structures subjected to large lateral loads; for example,

(1) Gravel island

(2) Substitute for deep skirts

(3) Mobile production and exploration structures.

More recently, spuds were proposed for transferring lateral ice loads on a gravity structure, past soft surficial soil layers, into more competent soil deposits such as permafrost (Earth Technology, 1983a). The safety of this gravity structure depends largely on the performance of the spud foundation. For design purposes, experience and understanding of the behavior of gravity structure and spuds separately is insufficient. Solution to this problem requires an understanding of the structure-spud-soil interaction.

Presently, there are no established design guidelines for design of spuds. When considering spud behavior, the unique characteristics of a short, large-diameter pile may be quite different from intermediate or long piles. Thus, design procedures used in selecting and sizing the spud system must be considered in a rational, mechanistic evaluation in its own right. Relationships should not generally be presumed from the semantics associated with the lateral behavior of long piles.

In March 1984, The Earth Technology Corporation (ETC) submitted a proposal to the Minerals Management Service of the U.S. Department of the Interior to investigate the behavior of spud foundation. Funding for this proposed study was approved in September 1984 (Contract No. 14-12-0001-30210). This report summarizes the results of this project.

1.2 Objectives and Scope

The primary objective of this project was to evaluate the principal parameters and mechanisms affecting the behavior of spud foundations for Arctic application. Some design guidelines are also recommended.

In order to accomplish the above objectives, the behavior of an isolated spud was first evaluated. Then, attempts were made to extend the single-spud solution to consider the interaction of a spud-structure-soil system. The following specific tasks were performed:

(1) Review state-of-knowledge on behavior of short (rigid) pile including drilled piers

(2) Develop analytical procedure for a single spud

(3) Perform sensitivity study for a single spud

(4) Perform simplified spud-structure interaction study

(5) Recommend design guidelines

(6) Recommend future work

This study was a preliminary evaluation of the spud problem. No experimental data was available to guide the development of design procedure. Therefore, the recommendations contained herein represent a first-cut estimate and should be verified when additional data becomes available.

In the review of publications, literatures related to large caissons in general were studied including the behavior of steel dowels and drilled piers. An analytical procedure was established based on the results of the literature survey, back-analysis of some literature data, and some in-house experience on pile analysis. A sensitivity study was necessary to establish rough quantitative estimation of factors affecting the total behavior.

In addition to the above task, a special case of spud application in the Arctic was introduced and studied. A single two-dimensional finite-element solution was presented for this problem. However, the results of the finite-element analysis are probably not valid for spuds because of the unrealistic boundary confinement (spud was modeled as a strip). Nevertheless, the results illustrated the feasibility of using the finite-element method for solving this type of foundation problem. Model testing and three-dimensional finite-element analysis offer the only hope for a realistic solution to the spud-structure-soil interaction problem.

1.3 Personnel

Mr. Charles E. Smith was the Technical Representative for Minerals Management Service (MMS) of the U.S. Department of the Interior. Mr. Lino Cheang was the project manager and was responsible for all phases of this study. Mr. Cheang was assisted by Mr. Ignatius Po Lam. This report was reviewed by Mr. Hudson Matlock.

1.4 Organization of Report

In this report, there are a total of seven chapters. Chapter 2 contains the results of the literature survey which includes some information on current design tools. Chapter 3 describes a computer code developed for this study. Some comparisons between calculated and measured results are also given in this chapter using the computer code. Chapter 4 presents the results of the sensitivity study which were used to define critical parameters affecting the behavior of spuds.

A special case of spud application in the Arctic was presented and analyzed in Chapter 5. Conclusions and recommendations are presented in Chapter 6, followed by a list of references.

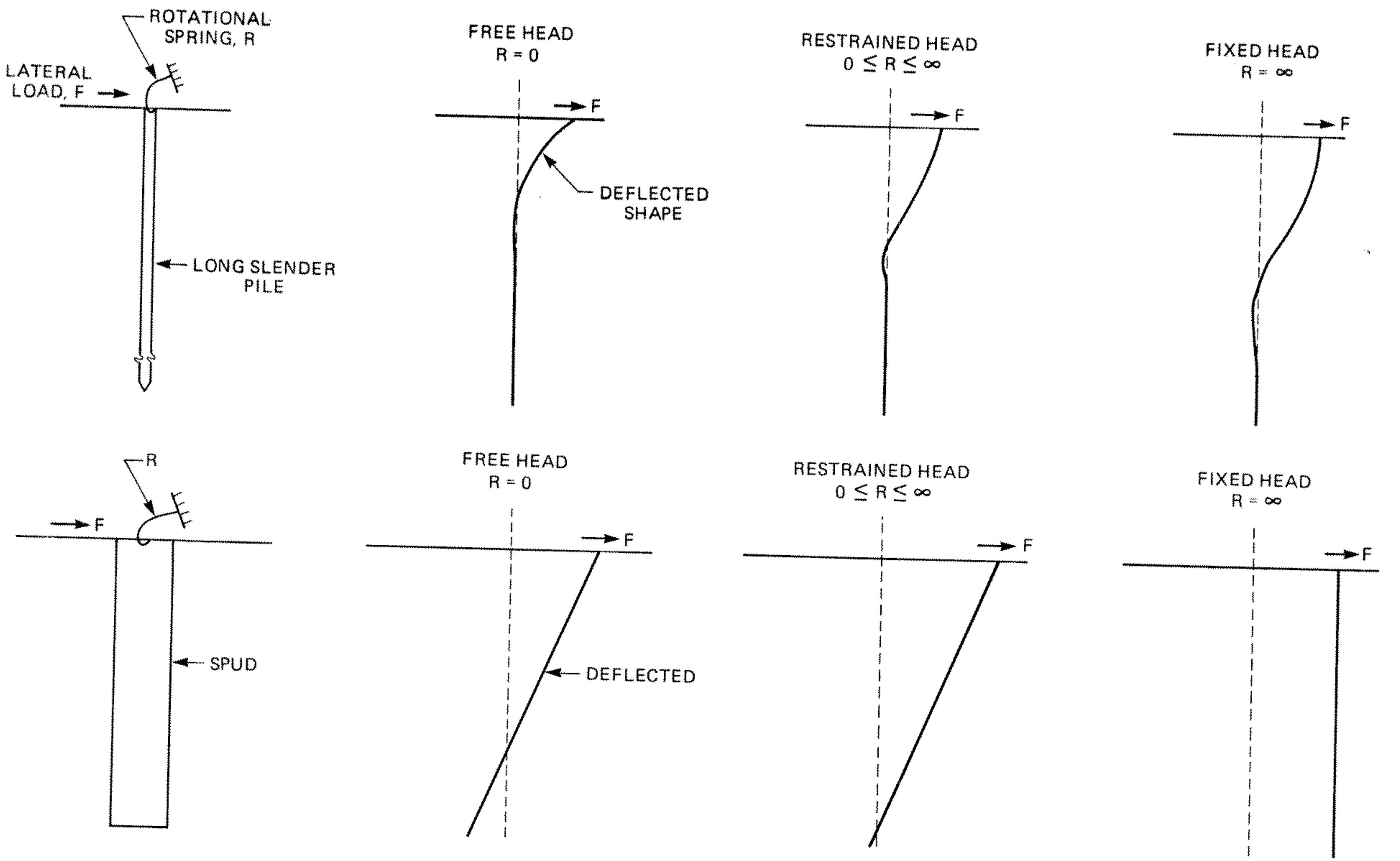


FIGURE 1–1 COMPARISON OF DEFLECTED SHAPES BETWEEN LONG PILES AND SPUDS

2.0 LITERATURE REVIEW

2.1 General

The purpose of the literature review was to summarize some pile analysis procedures possibly useful in spud analysis. The results of the survey also point out the limitations associated with the current state-of-knowledge which could help in the development of a new analytical procedure.

Most of the publications available in the public domain consider the behavior of single rigid piles or drilled piers. Only two references were found on the use of spuds for anchoring gravity structure in the Arctic to resist ice forces (Bea et al., 1983; Gerwick et al., 1983).

As mentioned in Section 1.2, it is essential to understand first the behavior of a single spud before an extension to a complex structure - spud interaction problem similar to the Arctic gravity structure. So, the survey results on rigid piles and piers were used to establish an analytical program to study the behavior of a single spud.

The survey was grouped into two categories: experiment and theory. In the area of theory, many solutions were summarized including elastic subgrade modulus procedures and nonlinear (p-y type) load-deflection methods. Some derivation of the ultimate lateral resistance was also presented.

2.2 Experimental Data

Lateral field pile load tests are limited, and the test program and pile configuration were usually tailored to meet the requirements of a long slender offshore pile (Matlock, 1970; Reese et al., 1974; Reese et al., 1975; Earth Technology, 1983b). Field testing of laterally-loaded drilled piers are also limited (EPRI, 1982; Reese and Allen, 1977; Bhushan et al., 1979; Bhushan et al., 1981; Bierschwale et al., 1981; Adams and Radhakrishna, 1973; Davisson and Salley, 1968) in the literature. Behavior of drilled pier with small length to width ratio under lateral loading in a manner resembles a spud or a rigid (short) pile. Both a spud and a drilled pier undergo some form of rigid rotation under lateral loading for a free-head condition. A series of drilled pier tests documented in a report submitted to EPRI by GAI Consultants Inc. (1982) was selected from the available publications for some comparison analysis using a computer code. Details of the test set-up and experimental and analytical results are documented in the next chapter.

Reese (1962) performed a series of experiments to determine the ultimate resistance of a laterally-moving rigid cylinder in sand. The cylinder was 1.125 inches in diameter with penetrations ranging from 1.5 to 6 inches. Lateral load was applied by pulling the cylinder through the sand by means of a tow line. The results of the experiment seemed to verify to some degree the Rankine failure wedge procedure for estimating ultimate lateral resistance.

Two steel tubular piles with diameters of 10.2 and 13.4 feet were test loaded by horizontal forces in connection with the damming of the Easterscheldt estuary (Lubking, 1977). The large-diameter piles were embedded in a marine sand. Load-movement behavior near the pile top and bending moment along the pile length were measured. Unfortunately, the prototype test was conducted to fulfill some design requirements, therefore the documentation of the test results and soil conditions was quite poor and could not be used for back-fitting analysis.

Many North-Sea gravity structures are equipped with dowels to prevent the platform from skidding along the seabed (NGI and DnV, 1982). Diameters of these dowels range from 9.8 to 14.8 feet with penetrations of 6.6 to 29.5 feet. However, documentation of the performance of dowels under lateral loading is not available from the survey.

2.3 Theoretical Data

Presently, laterally loaded pile design is based on guidelines provided by the American Petroleum Institute (API, 1984). Some recommendations are also given by DnV (1977). Both the API and DnV procedures recommend the use of p-y curves to represent the foundation. A set of nonlinear support curves characterizing the lateral soil reaction versus lateral pile deflection is normally referred to as a set of p-y curves. However, derivation of the p-y curves was based on limited load test data on long slender piles and may not be applicable for a spud foundation.

Although not documented in API and DnV, procedures are available to solve for the behavior of a short rigid pile (spud) under lateral loadings. In an attempt to present these various procedures in a orderly manner, they were grouped into three categories.

Ultimate Lateral Resistance. To establish a factor of safety for design, some estimate of the ultimate load is necessary. Earlier work performed by Czernicak (1957) considered the ultimate resistance to overturning of single, short piles. A rigid pile rotates about some point below the ground surface under lateral loading. The ultimate horizontal pressure against the pile may be estimated by:

$$P_h = K\gamma x \tan^2(45° + \frac{\phi}{2}) + 2c \tan(45° + \frac{\phi}{2}) \quad \ldots\ldots\ldots \quad (2.1)$$

where

γ = unit weight of soil

x = depth

c = cohesion

ϕ = angle of internal friction

k = efficiency factor (2 for round bored piles)

Brom (1964a and 1964b) estimated the ultimate resistance in cohesive soil to be zero down to a depth of 1.5 pile diameters and equal to 9 times the undrained shear strength below this depth. For cohesionless soil, the distribution of soil reaction was assumed to be zero at the ground surface and increase linearly to 3 times the passive Rankine earth pressure, $3\gamma LK_p$, at the bottom of the pile (L is the length and K_p is the coefficient of passive earth pressure).

Brinch Hansen (1961) proposed the following expression to compute the ultimate pressure per unit length:

$$p = B\,(q\,K_q + cK_c) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2.2)$$

where

B = width (diameter) of pile or spud

q = effective overburden pressure

c = cohesion

K_q and K_c = function of friction angle and depth to diameter ratio

Pile research performed at the University of Texas in the sixties and seventies by Matlock, Reese and their colleagues also resulted in some recommendations for ultimate lateral pressure (Thompson, 1960; Matlock, 1970; Reese and Welch, 1975; Parker and Reese, 1970).

Perhaps one of the more comprehensive models was developed by Ivey (1968). Ivey's model incorporates vertical side shear resistance, base shear, and a vertical base force as well as the Rankine type lateral earth pressures. It will be pointed out later in this report that the vertical side and base shear reactions are very important in determining the behavior of spuds.

Linear Load-Deflection Models Nonlinear pile solutions usually required the aid of computer models. In many cases, due to the insensitivity of overall pile behavior to the variation of soil support characteristics, linear representation of the soil stiffness would yield reasonable pile solutions. This type of linear model may also be applied to analyze the behavior of spuds.

In general, there are two types of linear solutions, subgrade modulus and elastic half-space. The subgrade modulus approach was clearly illustrated by the well-known beam on elastic foundation problem reported by Hetenyi (1946). Hetenyi idealized a simple relationship of soil reaction to deflection:

$$p = K_h \delta \qquad (2.3)$$

where

p = soil reaction

δ = deflection

K_h = lateral subgrade modulus

The subgrade modulus is then used to solve a differential equation to compute deflection, slope, shear and moment along an elastic beam. Of course, the relationship in Equation 2.3 is approximate; numerous other relationships have been published and the results of several more commonly used are presented below.

The modulus of horizontal subgrade reaction for sand recommended by Terzaghi (1955) has been widely used in practice. Reese el al (1974) also provided some recommendations for sand which corresponds to the initial tangent stiffness of the p-y characteristics. The Terzaghi and Reese subgrade moduli are plotted as a function of friction angle and relative density in Figure 2-1. Terzaghi (1955) also recommended a procedure to obtain the linear subgrade stiffness for clay using results from one foot square plate load tests.

Two elastic half-space solutions proposed by Douglas and Davis (1964) and Poulos (1971) are briefly reviewed below. These solutions are based on the well-known Mindlin's solution (1936) for a horizontal point load in a homogenous elastic half-space. Douglas and Davis proposed a method to compute displacement and rotation of a rigid vertical plate embedded in a uniform elastic half-space and subjected to a horizontal load or a moment applied to its upper edge. Poulos presented some solutions of displacement and rotation of a vertical strip embedded in a uniform elastic half-space and subject to the same loading condition as the Douglas and Davis solution.

Nonlinear Load-Deflection Models. A realistic analysis approach should account for the nonlinear and the layering nature of soil conditions. Current nonlinear analysis usually models the soil support along the pile by discrete nonlinear springs (p-y curves). Derivations of p-y curves for clayey and sandy soils have been proposed by Matlock (1970) and Reese et al (1974). Their approaches have been simplified and refined by others (O'Neill and Murchison, 1983; O'Neill and Gazioglu, 1984; Bogard and Matlock, 1980). Unfortunately, the p-y concept was developed based on load test data on long slender piles. Application of this concept to spud design is questionable because of the difference in the response of a long slender pile and a spud under lateral loading as discussed in Chapter 1.

2.4 Summary

As shown by the above results, including drilled piers, very limited experimental data are available for large-diameter pile foundation. Most of the design experience on rigid piles is based on theoretical developments. These theoretical solutions are either limited by assumptions for problem simplification or simply not applicable for analyzing spud foundations. It is obvious from the survey that in order to solve the spud problem, a few research studies have to be initiated. The remaining portion of this report attempted to advance the technology a step forward by addressing the fundamentals that governed the behavior of a single spud and then extending these fundamentals to include the structure-spud interaction problem, leading to recommendations for further work.

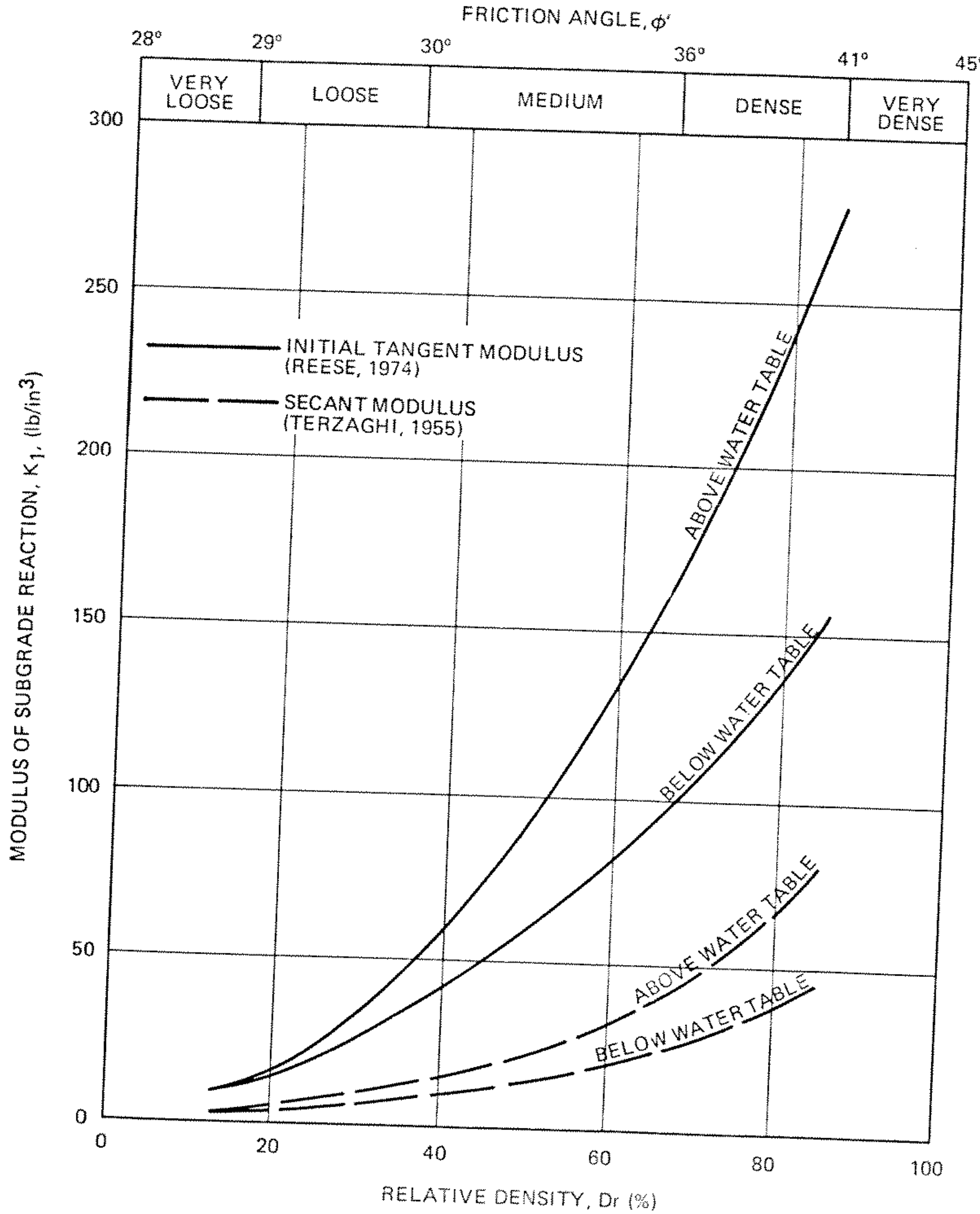


FIGURE 2–1 MODULUS OF SUBGRADE REACTION, K_1 FOR SAND
(After O'NEILL and MURCHISON, 1983)

3.0 ANALYTICAL CORRELATION WITH DRILLED PIER

3.1 General

In order to study the complex behavior of a group of spuds anchoring a gravity structure in the Arctic, some understanding of the general behavior of a single spud in uniform soils is essential for a first start. From the results of the literature survey, it is apparent that most of the analytical experience on rigid piles was supplemented from theories developed for long slender piles. These concepts have not been verified by prototype test data on spuds. In addition, there is simply very little information on the Arctic structure problem.

One foundation type which most nearly resembles a spud is the drilled pier. Design philosophy and procedures for drilled piers are well-documented in several publications (Reese and Wright, 1977; EPRI, 1982).

For this study development of an analytical model for use in evaluating spud behavior was focused on knowledge accumulated in pile and drilled pier design. The latter was used here for some back-analysis with a computer model. Details of this model and subsequent comparison of its results with field pier test data are presented below. Provided the results of the comparison analysis are satisfactory, the computer model would be extended to evaluate the behavior of a single spud.

3.2 Analytical Model

The fundamental difference between a drilled pier or a spud and a slender pile under lateral loading is, for a free-head drilled pier, in addition to the p-y soil resistance, significant soil resisting forces may be generated from the rigid rotation as well as sliding along the tip (base shear); and for a fixed or restrained-head condition, only the rotational reaction may be neglected. Whereas for a long-slender pile under any boundary restrains, the deformation is mainly resisted by the p-y soil resistance; the rotational and base shear soil reactions are generally small because the overall rotation and tip movement are negligible. Therefore, in deriving an appropriate model, both rotational and base shear reactions must be considered.

Discrete-Element Model. Pile design problems, in general, have been successfully solved by the discrete-element model developed by Matlock et al (1981). This discrete-element model was coded into a computer program BMCOL 76 (Matlock et al., 1981). Program BMCOL 76 is a general purpose program capable of analyzing the behavior under static loading of a wide range of structural members, for example: structural beam columns, piles, and pipelines. Lateral soil support for the beam-column can be described as nonlinear curves of load transfer versus displacement. Various restraints can be assigned to simulate different boundary conditions encountered for most pile problems: free, restrained and fixed-head conditions. The pier model developed for this comparison analysis is a spin-off of the above discrete-element model.

A simplified mechanical analogy of a segment of the pier model, modified from the original Matlock's model, is shown in Figure 3-1. The only modification to the original model is the implementation of a nonlinear-elastic rotational support (moment reaction versus pier rotation). Derivation of this rotational support curve is discussed below.

Rotational Support Curve. As discussed in Chapter 1, when a pier or short pile is subjected to a lateral load (F), equilibrium is provided by several forms of foundation reactions. These reactions are shown in Figure 3-2 and consist of

(1) p-y soil resistance,

(2) base shear,

(3) rotational reaction along the spud due to axial frictional resistance, and

(4) rotational reaction at the spud tip due to end bearing resistance.

Calculation of the p-y and shear resistances is presented later. Calculation of the rotational reaction is presented below.

Because of the lack of available theoretical and experimental data, a first attempt is to correlate the rotational reaction to axial load transfer behavior. As shown in Figure 3-2, the reaction along the shaft was correlated to the frictional resistance (t) versus axial displacement (z) relationship. T-z relationships have been published and adopted for practice frequently (Coyle and Reese, 1966; Vijayvergiya, 1977; Kraft et al., 1981). As the pier rotates, upward or downward frictional resistance is generated along the outside surface. The magnitude of this frictional resistance is a function of the prescribed t-z relationship and the amount of rotation. These frictional resistances result in couples which are identified as rotational reactions (see Figure 3-2). The total rotational reaction acting on a pier element is the summation of the couples over the surface area.

Another component of rotational reaction exists at the tip of the pier. This component is correlated to the end bearing resistance (q) versus tip displacement (z) relationship. Q-z relationships have been documented by Vijayvergiya (1977). The total tip reaction is obtained by integrating the couples over the cross-sectional area.

The above derivation represents a first-cut estimate of a nonlinear rotational reaction curve. The validity of correlating the rotational reaction to the axial soil response must be verified by experimental data. Therefore, two sets of drilled pier test results were used to compare with predictions obtained by the pier model. This comparison is presented in the next section.

3.3 Comparison with Experimental Data

A well-documented case history on the lateral behavior of a steel spud was not found. Therefore, case histories on drilled piers with a small length to diameter ratio were selected for the comparison. Two well-documented load tests were selected from a series of drilled pier experiments conducted by GAI Consultants, Inc. Documentation of GAI's research was published in a report submitted to the Electric Power Research Institute (EPRI, 1982). The diameter of the drilled pier was about 5 ft with a length to diameter ratio of about 3. The site soil for the first test consisted of predominantly clay (cohesive) soil. For the second test site, the soil was predominantly sands, gravel and silts (cohesionless). Additional soil information are given in Figures 3-3 and 3-4 for the first and second sites, respectively.

Loading was applied by pulling a cable attached to the top of a load head. This load head was 80 ft long and was bolted to the test pier at the ground level. This loading condition resulted in a positive shear and moment and an axial (compressive) force on the pier top. The axial force was neglected in the analysis.

Analysis. The beam-column model described in Section 3.2 and shown in Figure 3-5 was used in the comparison analysis. As shown in Figure 3-5, the various components of soil supports for lateral loading were modeled, including:

(1) p-y soil support,

(2) lateral shear resistance at the tip of the pier,

(3) rotational support along the shaft, and

(4) rotational support at the tip.

Characteristics of p-y curves have been empirically correlated to pile load tests (Matlock, 1970; Reese et al., 1974). Procedures to compute these p-y curves are documented by the American Petroleum Institute (API, 1985). Validity of these procedures have been confirmed by numerous recent studies (Earth Technology, 1983b; Barton et al., 1983; O'Neill and Murchison, 1983; O'Neill and Gazioglu, 1984). The p-y curves derived for the soils encountered in the two test sites are shown in Figures 3-6 and 3-7.

The base shear resistance was modeled as a bi-linear spring (elasto-plastic). The ultimate resistance (τ_s) was computed using the following equations:

(1) For cohesive soil

$$\tau_s = A * S_u \quad \ldots \ldots \quad (3.1)$$

where

A = cross-sectional area of tip

S_u = shear strength

(2) For cohesionless soils

$$\tau_s = W' * \tan \phi \quad \ldots \ldots \quad (3.2)$$

where

W' = effective weight of pier

ϕ = friction angle

The yield deflection was assumed to be 2 and 3 percent of the tip diameter for cohesionless and cohesive soils, respectively.

The rotational support curves are presented in Figures 3-8 and 3-9 for the two test sites. These curves were computed using the method summarized in Section 3.2.

The loading condition was represented by a shear and moment applied at the pier top. This representation intended to model the field condition (see Figure 3-5). This loading condition and the above foundation supports were modeled in the computer code to solve for the response of the test pier.

Results. Two figures were used to summarize the computed and measured results, one figure for each test pier. Figure 3-10 presents the pier top load-deflection and load-rotation behavior at the ground line for the cohesive test site and Figure 3-11 summarizes the similar results for the cohesionless test site. In these summary plots, solutions are shown for three separate analyses to illustrate the effects of the prescribed foundation supports. Variation of foundation support included

(1) using p-y only,

(2) p-y plus base shear, and

(3) p-y, base shear and rotational reaction.

The general findings are summarized below:

(1) Modeling soil supports by p-y curves alone would lead to a gross over-estimate of pier deflection and rotation.

(2) For a clay site (see Figure 3-10), the rotational support along the shaft provided a significant amount of reaction to resist the moment loading.

(3) For a sandy site (see Figure 3-11), the contribution of the rotational support to the total foundation reaction is less than the p-y and base shear reactions.

(4) The lateral base shear resistance appears to be more significant for sandy than clayey materials.

(5) In both cases, the rotational resistance at the tip could be ignored.

As indicated by items (2) and (3) above, the rotational resistance along the length of the shaft appears to be very significant in affecting the load-deformation behavior of the drilled pier for the clay site, whereas the significance of the rotational soil resistance greatly diminishes for the sandy sites. The rotational resistance along the length of the pier is related to the skin friction characteristics (t-z curves). Therefore, it is of interest to compare the unit skin friction capacity with the ultimate resistance of the p-y curve (p_u). The ratio of tD/p_u is plotted in Figure 3-12 for both the clay site and the sand site (D is pier diameter). It could be noted that the ratio of skin friction to ultimate p-y resistance is much higher for the clay site, thus explaining the relatively high contribution of the rotational resistance for the clay site.

3.4 Summary

Based on the results of the above comparison, it was concluded that for sand, analyzing the load-deformation behavior of a single drilled pier using the p-y curves alone would lead to over-prediction of displacements; the tip (base) shear resistance could potentially be significant especially for a heavily loaded pier. For a clay soil, the rotational resistance along the length of the pier could be very significant. Therefore, using the p-y curves alone could lead to gross error.

At the present time, there is no generally accepted procedure to develop the rotational support curves. Furthermore, the procedure introduced here to compute rotational reaction is sensitive to

(1) Installation procedure of the pier because skin friction behavior can be altered by augering, or drilling

(2) Diameter of the pier

(3) Boundary condition at the top of the pier. For example, for a fixed-head pier, the significance of the rotational reaction would be reduced due to a smaller pier rotation

The excellent correlation obtained for the comparison analysis indicated that the rotational resistance and base shear concepts can be applied together with the conventional p-y concept to solve for the behavior of a drilled pier; this pier was subjected to lateral loading resulting in a high ratio of positive moment versus positive shear load at the pier top (free-head condition). For other boundary conditions such as the fixed and restrained-head cases, the rotational reaction can be backed out and the total soil resistance can be represented by the p-y and base shear components only. The rotational reaction would be insignificant for these latter boundary conditions because the magnitude of rotation is generally small. Since some confidence has been established on the representation of the foundation support in a drilled pier analysis, these concepts are ready for extension to the case of a single spud under lateral loading.

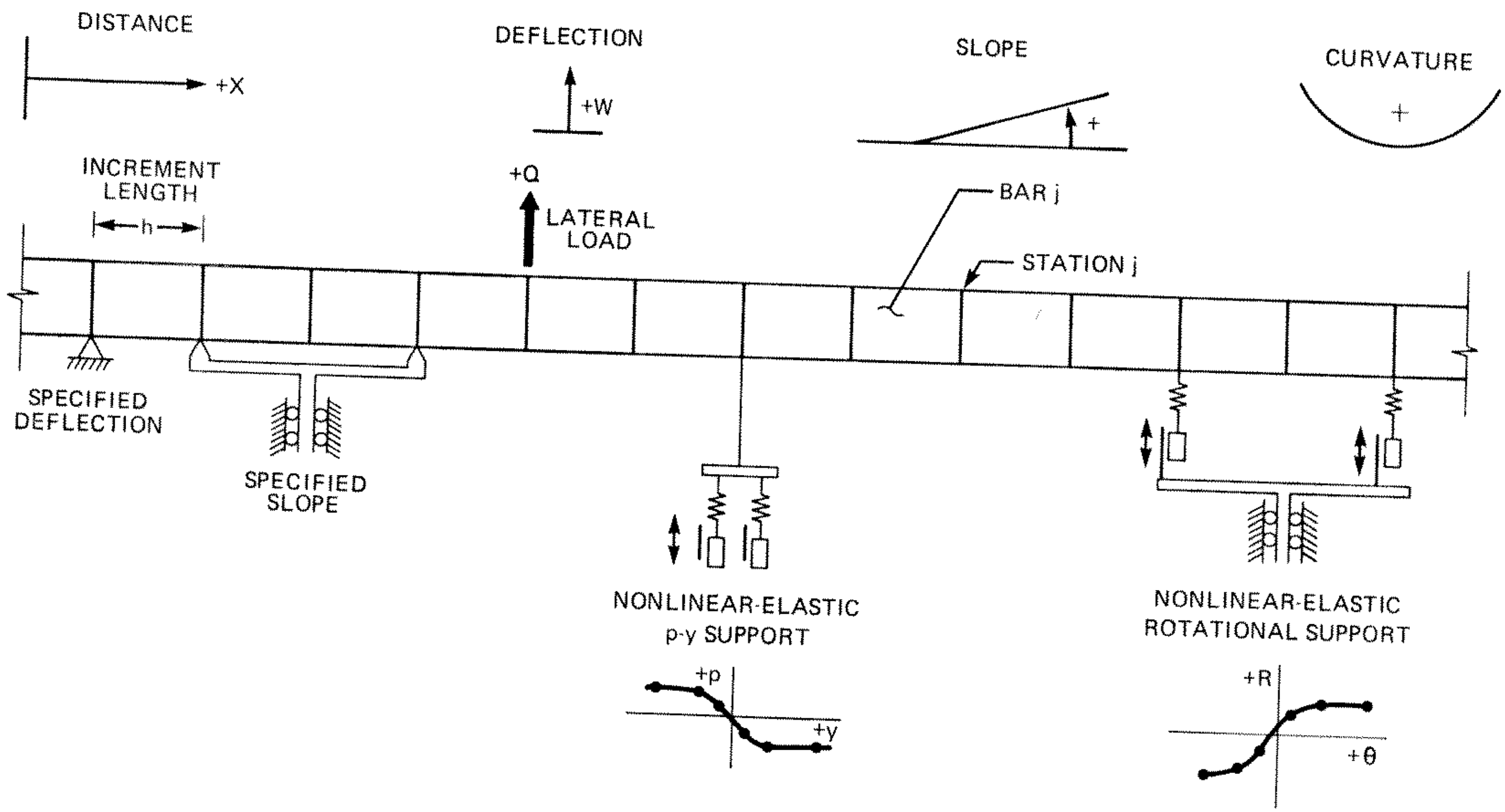


FIGURE 3–1 MECHANICAL ANALOGS AND SIGN CONVENTION FOR BEAM–COLUMN MODEL (AFTER MATLOCK ET AL, 1981)

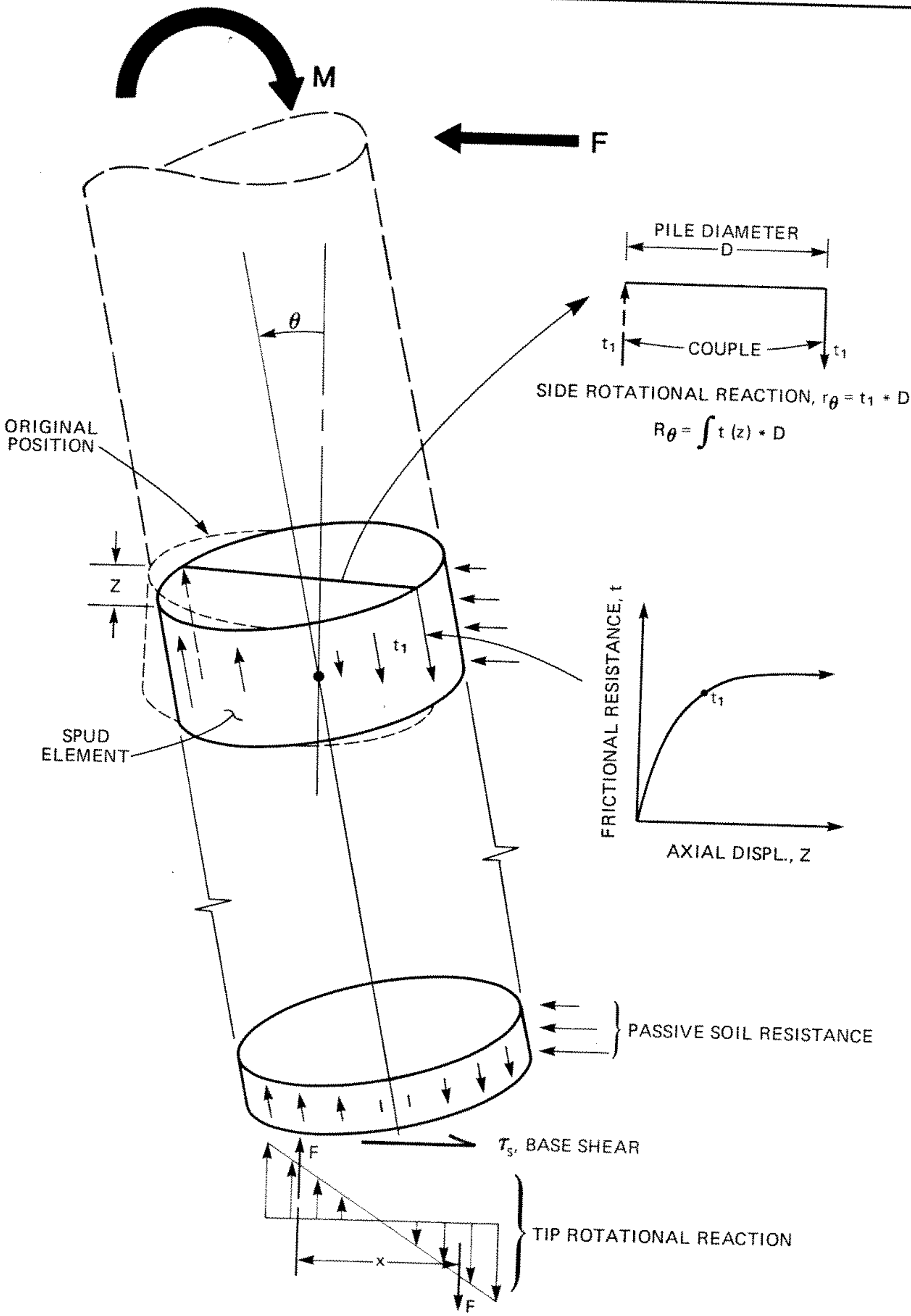
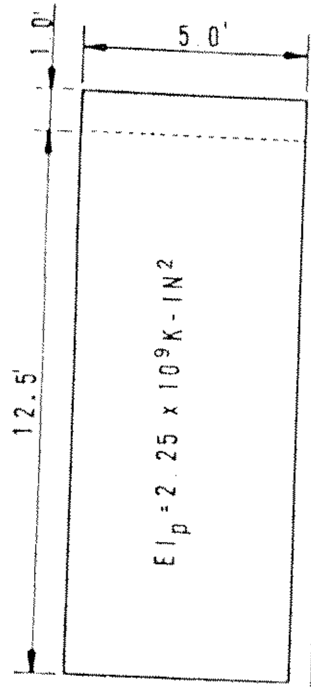
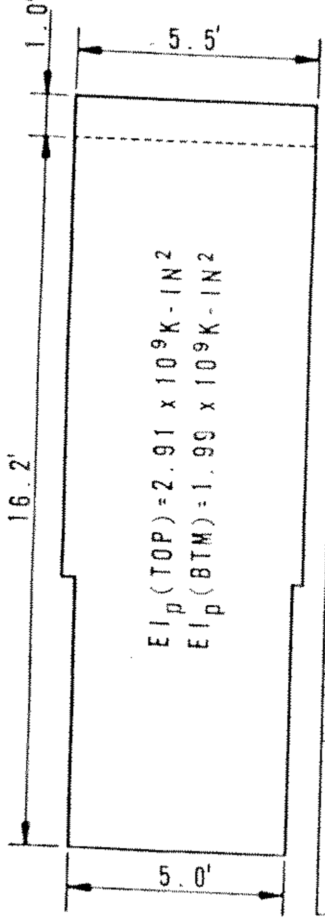


FIGURE 3–2 DERIVATION OF ROTATIONAL SUPPORT



DEPTH (FT)	GENERALIZED DESCRIPTION	UNIT WEIGHT $\overline{\gamma}$ (PCF)	MODULUS OF ELASTICITY E_p (KSI)	INTERNAL FRICTION ANGLE ϕ (°)	UNDRAINED COHESION C_u (KSF)	STRENGTH REDUCTION FACTOR α_r
	MED. STIFF TO STIFF CLAY (FILL)	129	0.6	—	1.5	0.60
5.5	V. STIFF SILTY CLAY	137	1.47	—	2.0	0.45
8.5	HARD SILTY CLAY FEW ROCK FRAGMENTS	137	1.47	—	3.75	0.40
10.0	TOP OF ROCK VERY SOFT RED CLAY SHALE	137	8.53	—	9.0	0.40
13.0						

FIGURE 3–3 IDEALIZED SUBSURFACE PROFILE FOR CLAY SITE
(AFTER EPRI, 1982)



DEPTH (FT)	GENERALIZED DESCRIPTION	UNIT WEIGHT $\bar{\gamma}$ (PCF)	MODULUS OF ELASTICITY E_p (KSI)	INTERNAL FRICTION ANGLE ϕ (°)	UNDRAINED COHESION C_u (KSF)	STRENGTH REDUCTION FACTOR α_r
3.5	LOOSE SILT TRACE SAND AND GRAVEL	110	0.65	32	—	1.0
4.5	LOOSE SILTY SAND					
6.0	MED. DENSE SAND (GWT)	110	1.08	38	—	0.94
7.5		110	1.08	36	—	0.98
10.0		48	1.48	36	—	0.98
12.0		48	1.48	36	—	0.98
13.5	DENSE SAND	48	3.78	45	—	0.80
17.5	MED. DENSE SAND	48	1.92	36	—	0.98

FIGURE 3–4 IDEALIZED SUBSURFACE PROFILE FOR SAND SITE
(AFTER EPRI, 1982)

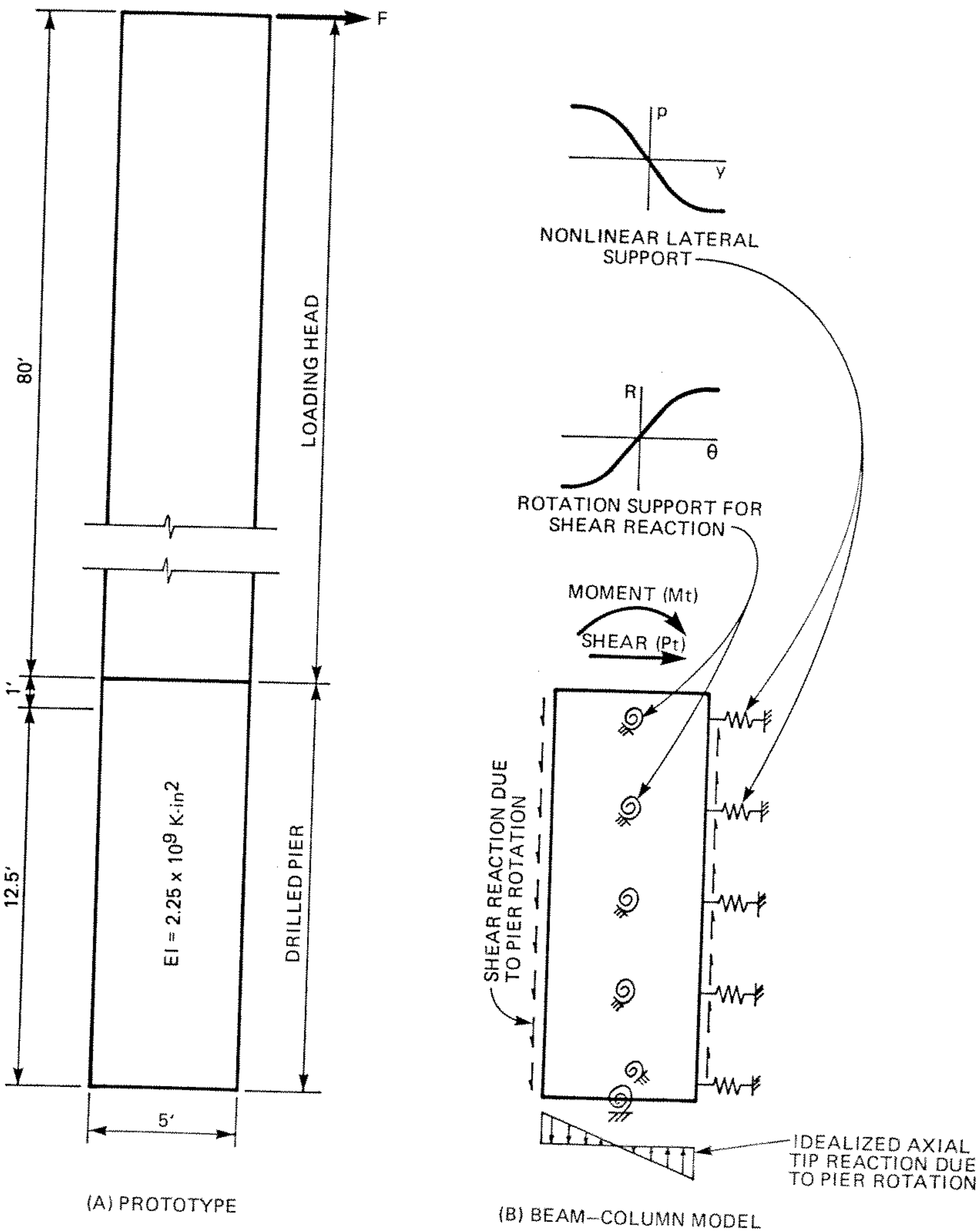


FIGURE 3–5 PROBLEM DESCRIPTION OF A DRILLED PIER

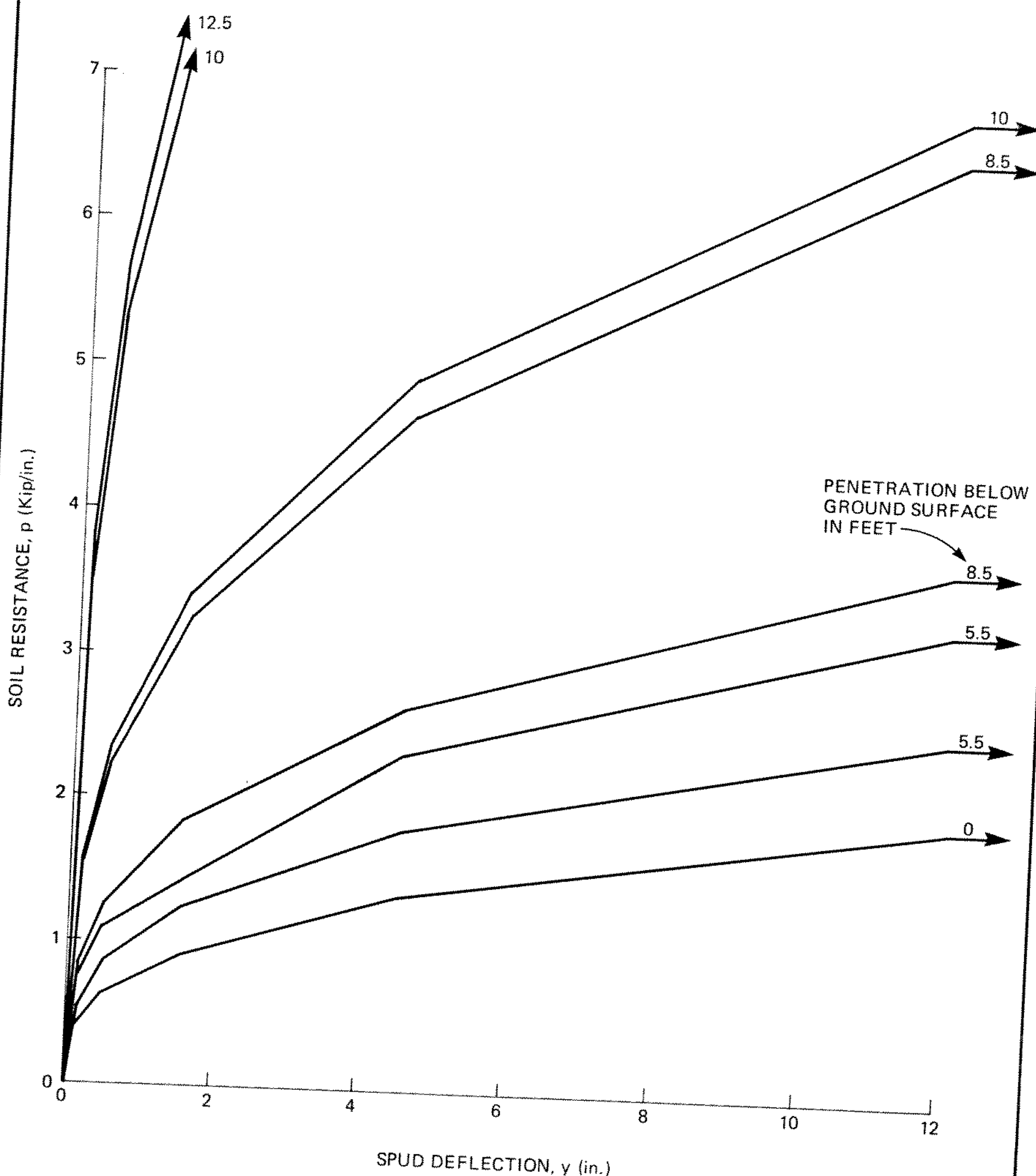


FIGURE 3–6 SOIL RESISTANCE (p) VERSUS SPUD DEFLECTION (y) CURVES FOR CLAY SITE

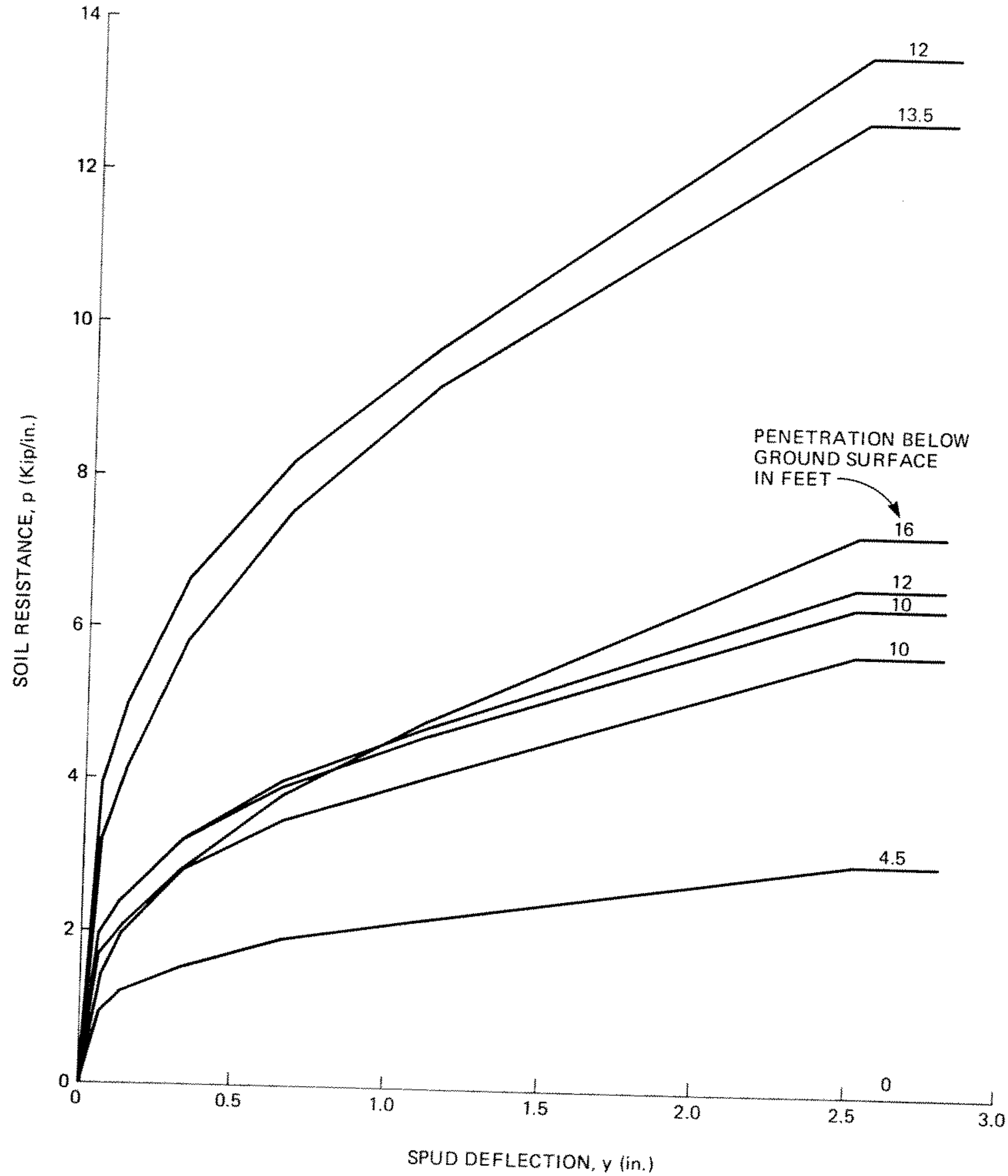


FIGURE 3–7 SOIL RESISTANCE (p) VERSUS SPUD DEFLECTION (y) CURVES FOR SAND SITE

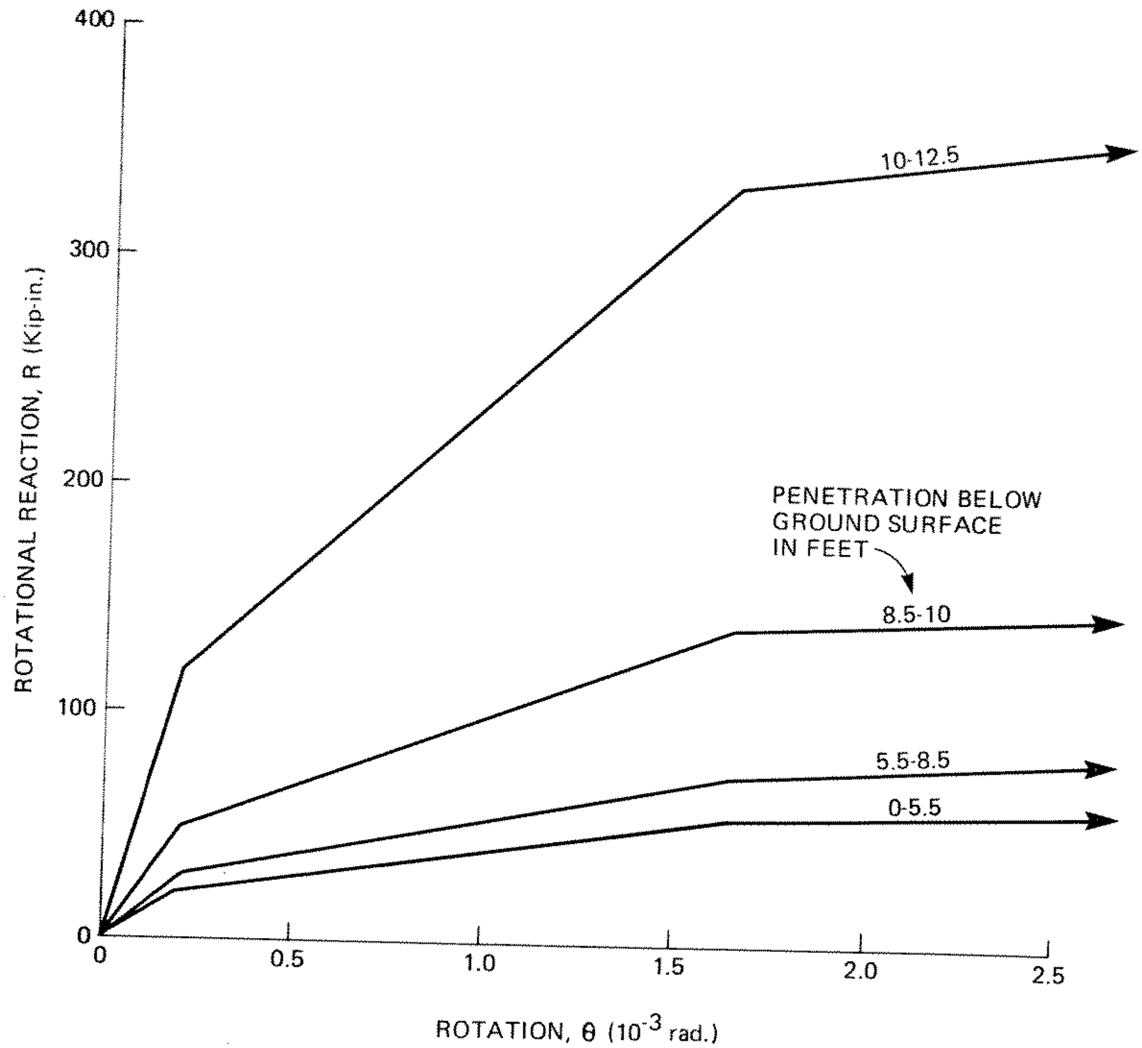


FIGURE 3–8 ROTATIONAL REACTION (R-θ) CURVES FOR CLAY SITE

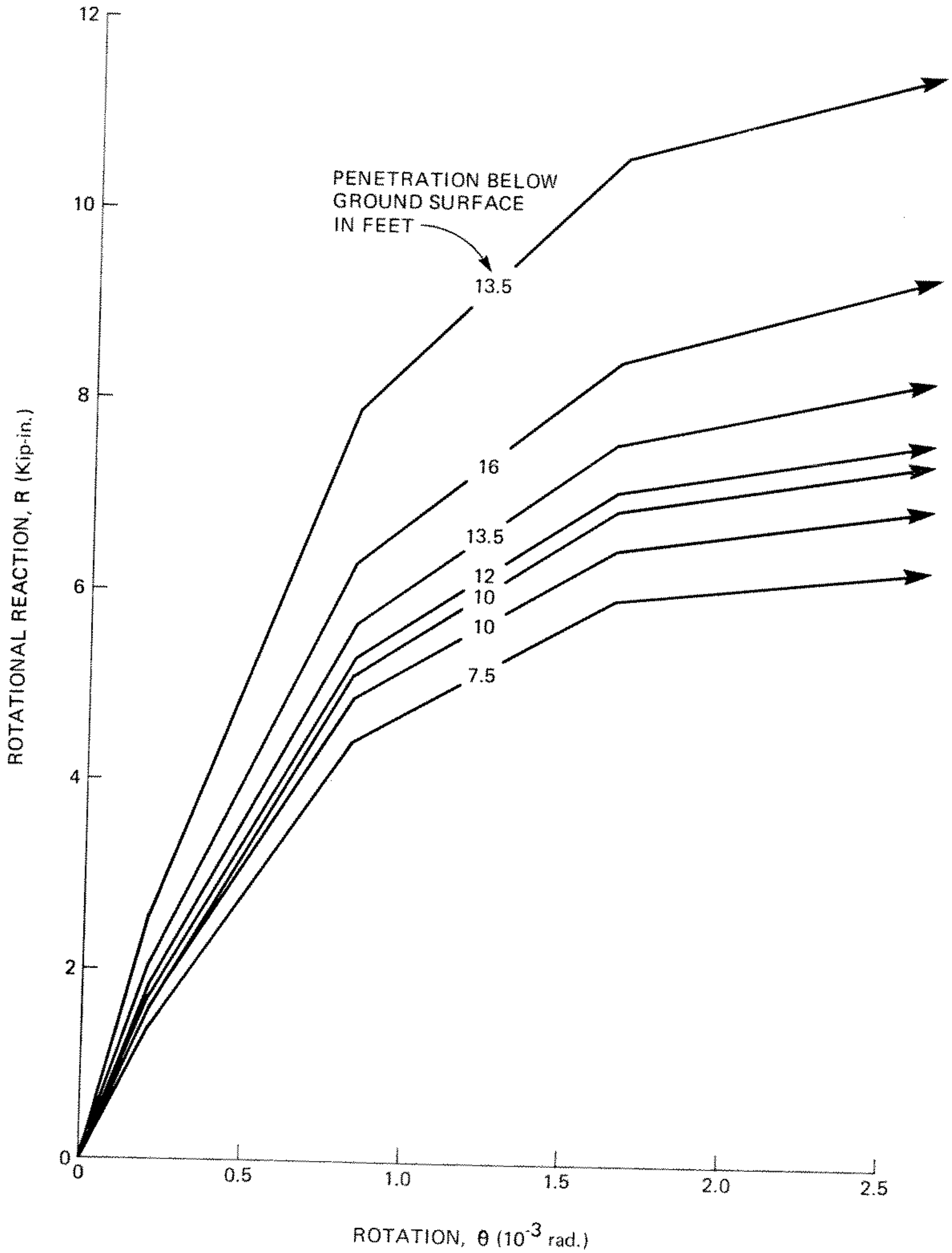


FIGURE 3–9 ROTATIONAL REACTION (R-θ) CURVES FOR SAND SITE

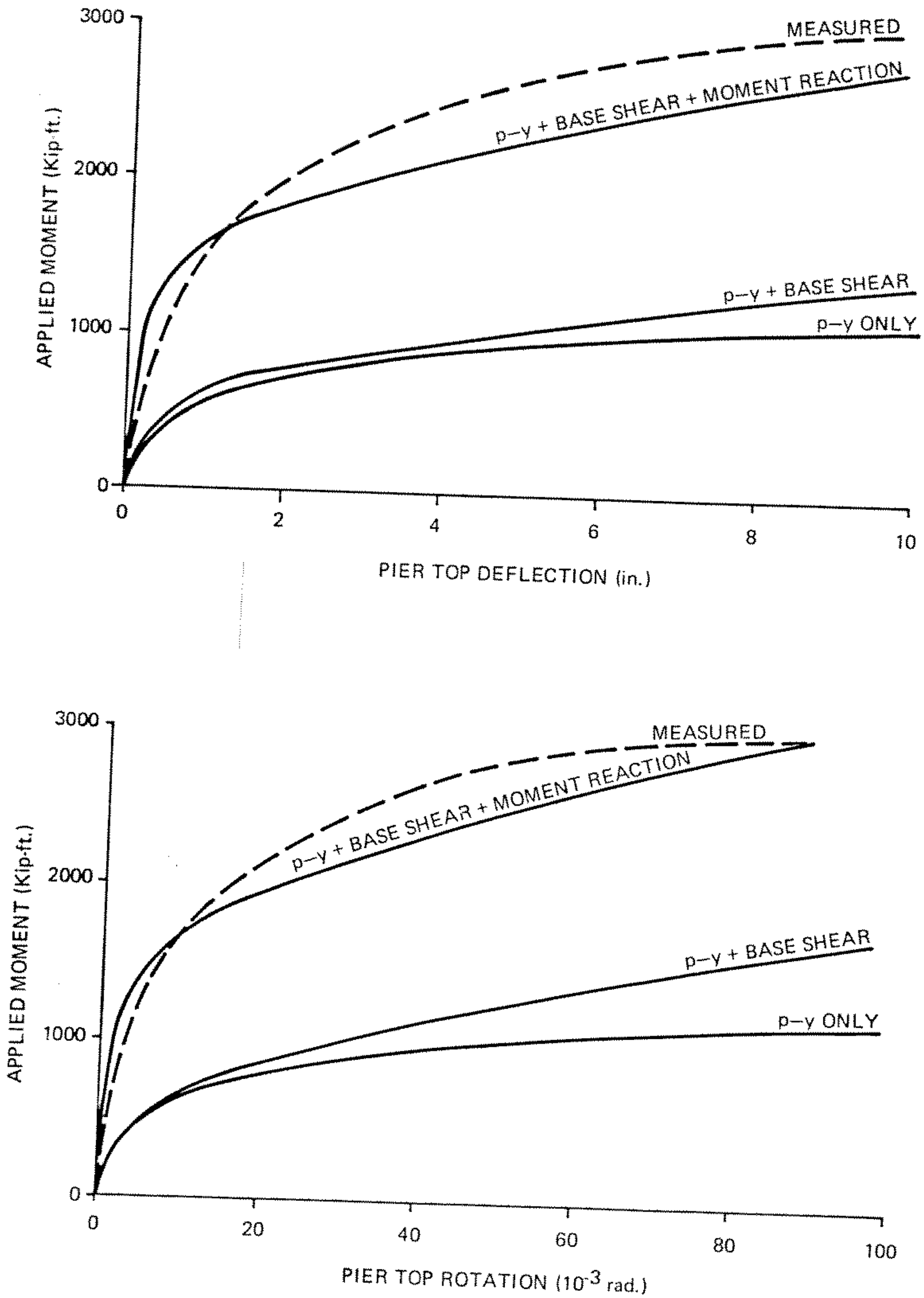
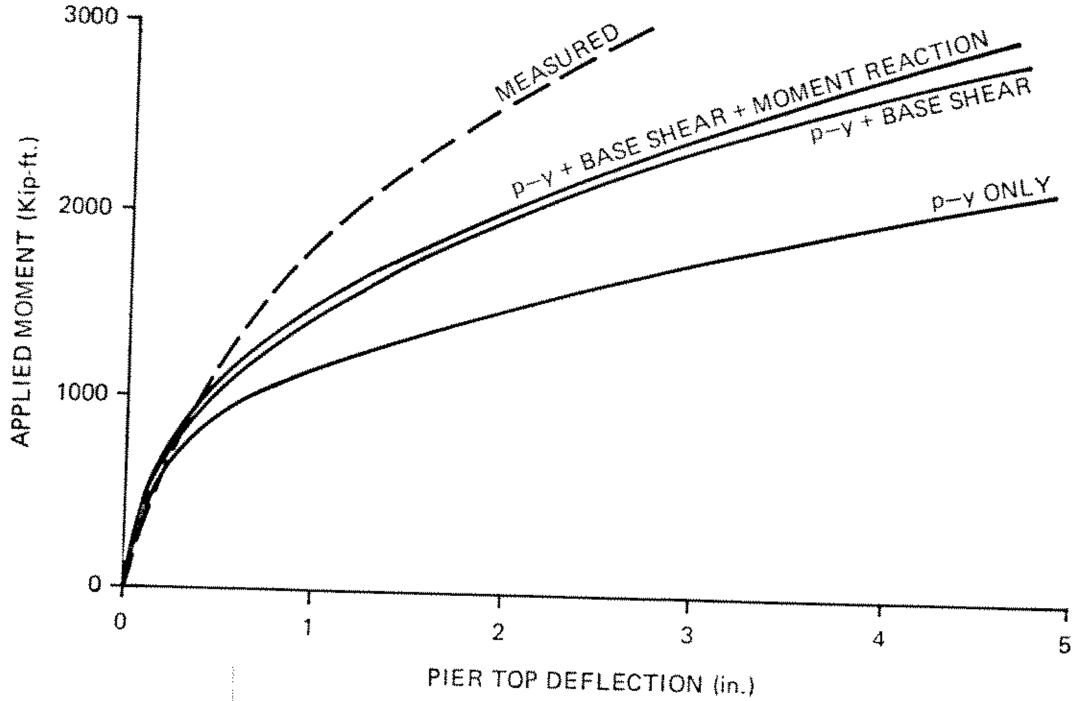


FIGURE 3–10 COMPARISON ANALYSIS OF PIER TEST FOR A CLAY SITE



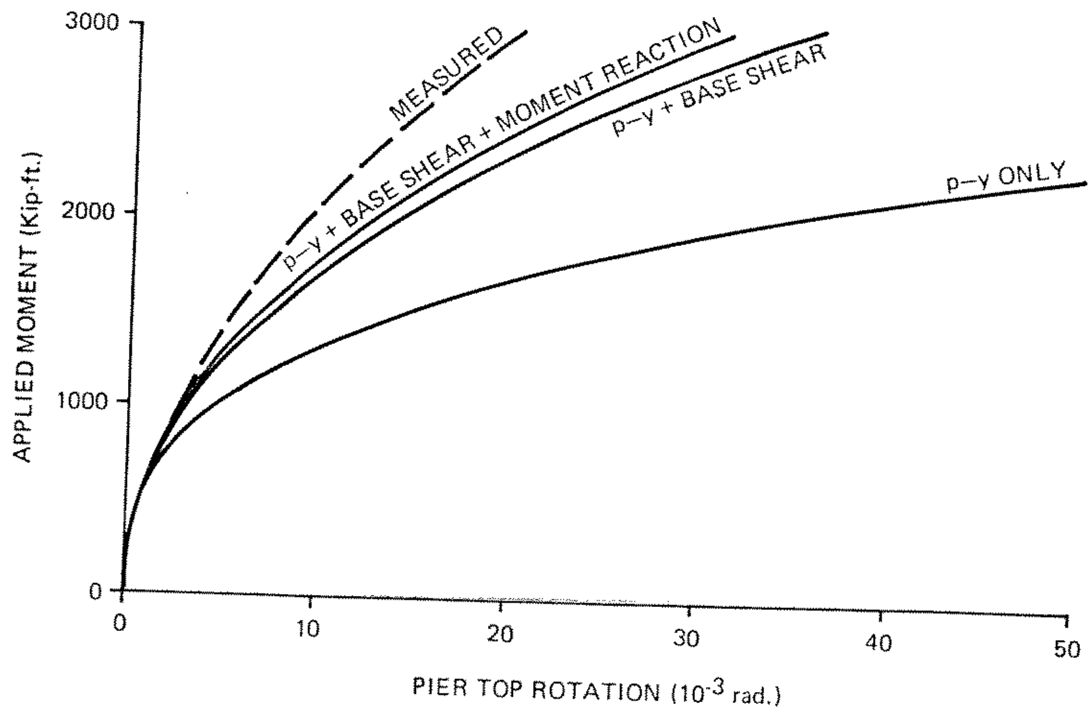
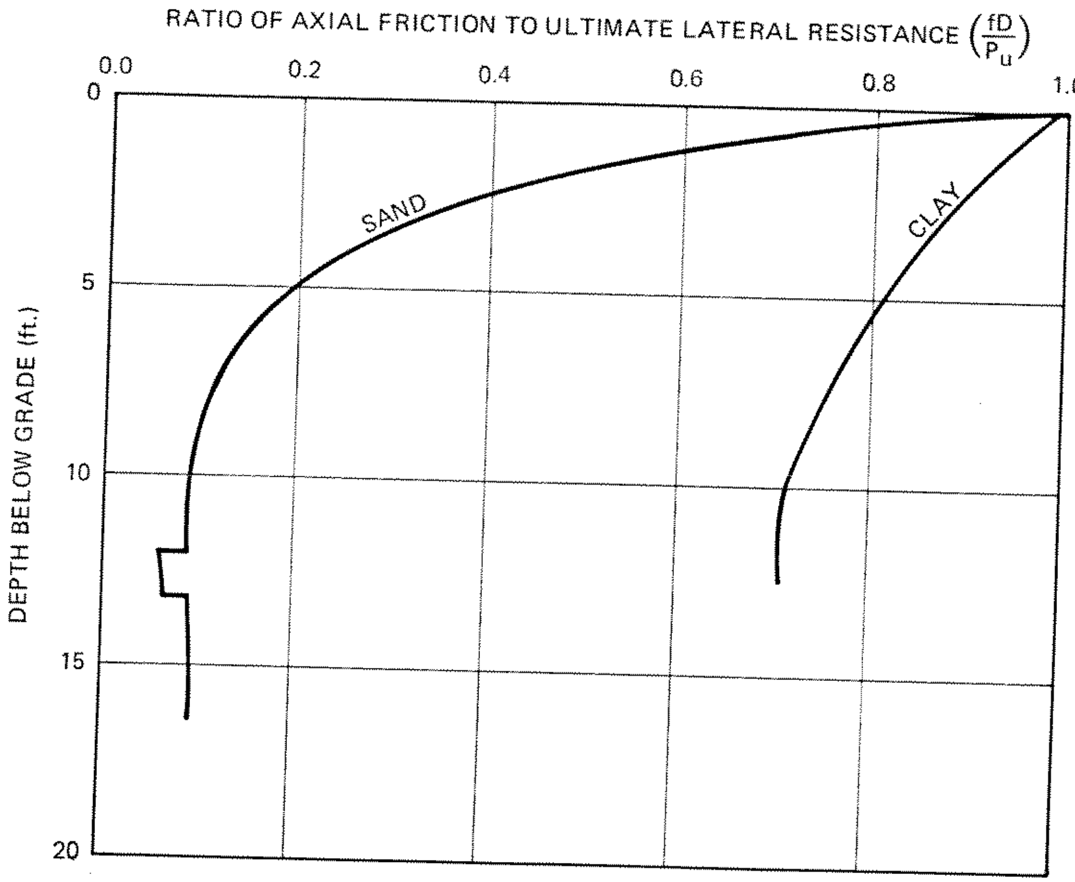


FIGURE 3–11 COMPARISON ANALYSIS OF PIER TEST FOR A SAND SITE



NOTES:

(1) $\frac{P_u}{D}$ = ULTIMATE LATERAL RESISTANCE ON p–y CURVE PER UNIT AREA

(2) f = UNIT SKIN FRICTION

(3) SEE FIGURES AND FOR PROFILE INFORMATION

FIGURE 3–12 COMPARISON OF AXIAL SKIN FRICTION WITH ULTIMATE LATERAL RESISTANCE ON p–y CURVE

4.0 EXTENSION TO SPUD DESIGN INCLUDING SENSITIVITY STUDY

4.1 Factors Affecting Spud Behavior

The sensitivity of the various foundation supports (p-y, rotational and base shear) to the behavior of a drilled pier has been investigated in the last chapter. Some analytical concepts were also established based on the results of a comparison analysis using a computer model and drilled pier test data. This computer code is applied here for the analysis of a single spud, including a more comprehensive evaluation of factors which may influence the performance of a spud under lateral loading:

(1) Boundary condition

(2) Variation of soil condition

(3) Spud length

(4) Spud diameter

Several example problems were set up to investigate the above parametric variations for conditions applicable to spuds. Some input data were varied for each computer run to accommodate the objective of each individual analysis. For example, both cohesive and cohesionless soils were used. For cohesionless soils, only the p-y and base shear reactions were coded in the computer model; the rotational reaction was neglected because it was shown to be small in the analysis of the drilled pier data in cohesionless soils. All three soil reaction components (p-y, base shear and rotation) were considered for cohesive soils. The problem description together with some of the input data used in the sensitivity study are summarized in Figure 4-1.

The pile-head boundary condition has been identified as an extremely important consideration for lateral pile design (Earth Technology, 1983b). The maximum bending stress on the spud is often a function of the specified boundary condition. Influence of the condition of the foundation soils should be studied because of the likelihood of such variations in the field. Spud length and diameter potentially affect the rotational component which was identified in the last chapter as an important parameter.

4.2 Boundary Conditions

In reality, spuds are connected to or reacted against the structure. Therefore, compatibility at the connection point must be properly modeled in any computer analysis. Three different boundary conditions are normally assumed which cover a wide range of spud-structure connections; the spud top can be (1) fully free (zero moment), (2) partially restrained (some finite rotational restraint), and (3) fully fixed (zero slope). In reality, a fully-free spud is unlikely. However, for generality these three types of connections were analyzed in the following example and their implications compared and discussed.

Input Data. As shown in Figure 4-1A, the steel spud was 30 feet long with a diameter of 6 feet. The wall thickness was assumed to be 3 inches. The spud was assumed to be fully embedded into a uniform sand with an angle of internal friction of 35 degrees. Loading was applied using incremental deflection at the spud top. The magnitude of the rotational restraint (4×10^9 16-in/rad.) was chosen to produce a more balance of distribution of positive and negative moment. Significant of this balanced distribution is discussed below.

Results. In order to demonstrate the effects of boundary condition, the resultant spud-head load and spud-head deflection curves are plotted in Figure 4-2A for the three spud-head connections being modeled. The solutions for the partially-restrained condition falls between the free and fixed-head case. This verifies the assumption used in practice that the free and fixed-head conditions serve as extreme bounds for load-deflection relationship. However, this assumption is not valid for peak moment, as shown in Figure 4-2B.

Plots of maximum moment versus spud-head load are presented in Figure 4-2B. This plot shows that the result of the restrained case is not bounded by the free and fixed-head curves. This anomaly is explained in Figure 4-3 which presents the moment distribution along the spud length at a spud-head load of 500 kips for the three boundary conditions. As shown by the moment curves, the restrained case gives a more balanced distribution of moment (maximum negative moment at spud top is roughly the same as maximum positive moment at depth). This balanced distribution resulted in a lower peak moment as compared to either the free or fixed-head case.

The deflection profiles are also included in Figure 4-3. As shown by the deflected shapes, the spuds behave as rigid members pivoting at a point under the imposed lateral load of 500 kips. This response is typical for spuds in which, unlike a long-flexible pile, reversals in curvature are generally expected.

4.3 Variation of Soil Conditions

In practice, soil conditions vary from site to site. It is desirable for a foundation designer to realize the sensitivity of soil variation to the overall behavior of spuds. This section summarizes the analytical results of a spud under lateral loading and embedded in four vastly different soil profiles.

Input Data. The spud dimensions for this series of analyses were identical to those used for the boundary condition study. As shown in Figure 4-1B, the four soil profiles assumed were:

(1) Uniform sand with a friction angle of 35 degrees

(2) Uniform sand with a friction angle of 30 degrees

(3) Normally consolidated clay with a c/p ratio of one-third

(4) Overconsolidated clay with a constant shear strength of 1 ksf

In Item (3) above, c is the shear strength and p is the effective overburden.

For simplicity, a fixed-head condition was modeled. The effects of other boundary restraints have been studied earlier in Section 4.2.

Results. The results of this sensitivity study were presented in two separate figures: one figure summarizing the solutions for the cohesionless soils and the other for the cohesive soils. These figures illustrated the relationship of spud-head load and maximum bending moment versus spud-head deflection.

As shown by the results of the cohesionless soils in Figure 4-4 and the cohesive soils in Figure 4-5, it appears that the load or moment versus deflection relationships are sensitive to strength changes based on the selected profiles.

It should also be pointed out that the selected soil profiles represent significant variations which can be encountered in real practice. Therefore, uncertainties in soil strength parameters should be carefully evaluated rather than judged to be insignificant for the spud problem.

4.4 Spud Length and Diameter

Variations in spud length and diameter affects the behavior of spuds by increasing or reducing the significance of the rotational reaction of the soil. An increase in spud length reduces the importance of the rotational component as the problem approaches to that of a long-slender pile. On the contrary, a larger-diameter spud increases the magnitude of the rotational reaction because of the higher couples generated by pairs of upward and downward frictional resistance along the spud wall.

In order to clearly highlight the effects of spud length and diameter, two series of analysis were performed for one spud configuration and clay soil profile:

(1) Fixed-head boundary condition with and without the consideration of rotational reaction

(2) Free-head boundary condition with and without the consideration of rotational reaction

The fixed and free-head boundary conditions would cover the extremes and thus binding the solution of a restrained-head spud. The soil was a uniform clay with a shear strength of 1 ksf and all solutions were conducted for a lateral load of 50 kips applied at the spud top.

The results of the above analyses were summarized in plots of length or diameter versus

(1) peak deflection ratio (δ/δ_R), and

(2) peak moment ratio (M/M_R).

The subscript R (δ_R, M_R) represented the solutions with consideration of the soil rotational reaction. Unity for the peak deflection or peak moment ratio would indicate that the effect of the rotational reaction could be neglected. A ratio in excess of unity illustrated the degree of importance of the rotational reaction.

Length. The spud has the same diameter (6 ft) and wall thickness (3 inches) as the one used for the earlier analyses except now the length was varied from 10, 20, 30, 60 to 120 feet (see Figure 4-1C).

The results for spud length effect are presented in Figure 4-6. As shown in this figure, the fixed-head spud is insensitive to the rotational reaction. However, the response of the free-head spud is highly dependent on the rotational reaction and aspect ratio. For example, the ratio of peak deflection increased exponentially for spud lengths of 30 ft or less. The same trend occurred for the peak moment ratio except the increase was not as dramatic. Still, a 50 percent increase in maximum bending moment is expected for spud length of about 10 ft if rotational reaction was not included in the analysis for a free-head spud.

Diameter. As shown in Figure 4-1D, four spud diameters were considered in this analysis: 4, 6, 9 and 15 feet. To limit this series of computation to diameter effect only, a constant bending stiffness was used which is not a realistic approach.

The plotted results are presented in Figure 4-7. As shown in Figure 4-7A, no difference in peak deflection is shown for the full range of spud diameters that were examined for the fixed-head spud. For the free-head case, a 40 percent overprediction in peak deflection was shown for spud diameters between 4 and 7 ft if rotational reaction was not considered. This variation increased up to more than 100 percent at a diameter of 15 ft. The influence of spud diameter on peak moment is presented in Figure 4-7B. Peak moment ratio in excess of unity was recorded for both the fixed and free-head cases with the former showing a ratio of 1.1 at a diameter of 15 ft and tapering off to about 1.05 at and up to a diameter of 4 ft. For the free-head spud, a more significant influence was observed. The peak moment ratio was about 1.3 at a diameter of 15 ft and gradually reduced to 1.2 at a diameter of about 10 ft; then it remained almost constant at about 1.15 between diameters of 4 to 10 ft.

Some of the pertinent results obtained from the above analysis to study the effects of spud length and diameter are further discussed in the summary at the end of this chapter. It should also be pointed out that this set of results is unique for the assumed spud configurations, stiffness, soil profile and loading condition. At this moment, the results cannot be generalized to cover other soil profiles and loading conditions.

4.5 Summary of Sensitivity Study

Boundary Condition. The proper simulation of boundary condition is critical in any analytical method for the assessment of magnitude and distribution of bending moment on the spud. Estimation of the magnitude of deflection and deflected-shape are also highly sensitive to the prescribed boundary condition. In reality, fixed and restrained-head conditions are more common for spud foundation.

Soil Variation. If there exists an uncertainty in soil strength parameters for a particular site, a proper account of the possible variation should be considered. In other words, appropriate lower and upper bound strength parameters should be developed and analyses performed to study the effect of these bounds on the response of the spud.

Length and Diameter. For long slender piles where the overall rotation is small, the effects of the rotational reactions from the soil can be neglected. However, for spuds embedded in cohesive soils, gross error may result if the rotational component is not accounted for in the analysis. From the limited results obtained, in general, the rotational reaction can be neglected for a fixed-head spud because the amount of rotation is limited by the fixity at the spud top. For free-head spuds, rotational reaction must be included in the analysis. For realistic restrained-head case, the spud-head response should be closer to a fixed-head condition and thus, the rotational reaction can also be neglected. If, for some reasons, the amount of restraint approaches a free-head condition, proper account of the rotational reaction must be included in the analysis.

Other Considerations. In addition to the above factors, several other parameters that were not studied here could also affect the response of spud. One area of concern is spud group effects. If the spuds are placed far enough apart (say three times the spud diameter or more), no modification is needed for the present analytical procedure. The spud group essentially behaves as individual spud. However, for closely-spaced spuds (less than three times the spud diameter), significant changes in the response may occur. Experimental evidence for long slender piles have shown that for the same lateral load, more deflection is recorded for individual pile in a closely-spaced group than a single pile (Bogard and Matlock, 1983; O'Neill, 1983). This is due to stress transfer from neighboring piles through the soil medium. Several mathematical models are available to solve for the group effects for long slender piles (O'Neill, 1983); however, there exists a significant need for experimental data to verify and simplify these mathematical methods. Due to the complexity of the problem and the limited scope of this study, spud group effects are not considered here.

Other potential areas of concern are soil liquefaction and gapping. Loose deposits are prone to liquefy during an earthquake. During cyclic loading, the formation of a conical gap at the soil surface has been observed by a number of researchers (Barton et al., 1983; Matlock, 1970). The liquefaction and gapping effects can result in a loss of soil resistance and may affect the spud behavior significantly.

As mentioned in Section 1.1, spuds are recently being considered for anchoring offshore structures in the Beaufort Sea. The results obtained so far for a single spud might provide some assistance to solving the spud-structure interaction problem. However, the interaction of the spud, soil and the base of the structure is far more complex than the solution of a single spud with loading applied at the spud top. This problem justifies a separate consideration and is dealt with in the next chapter.

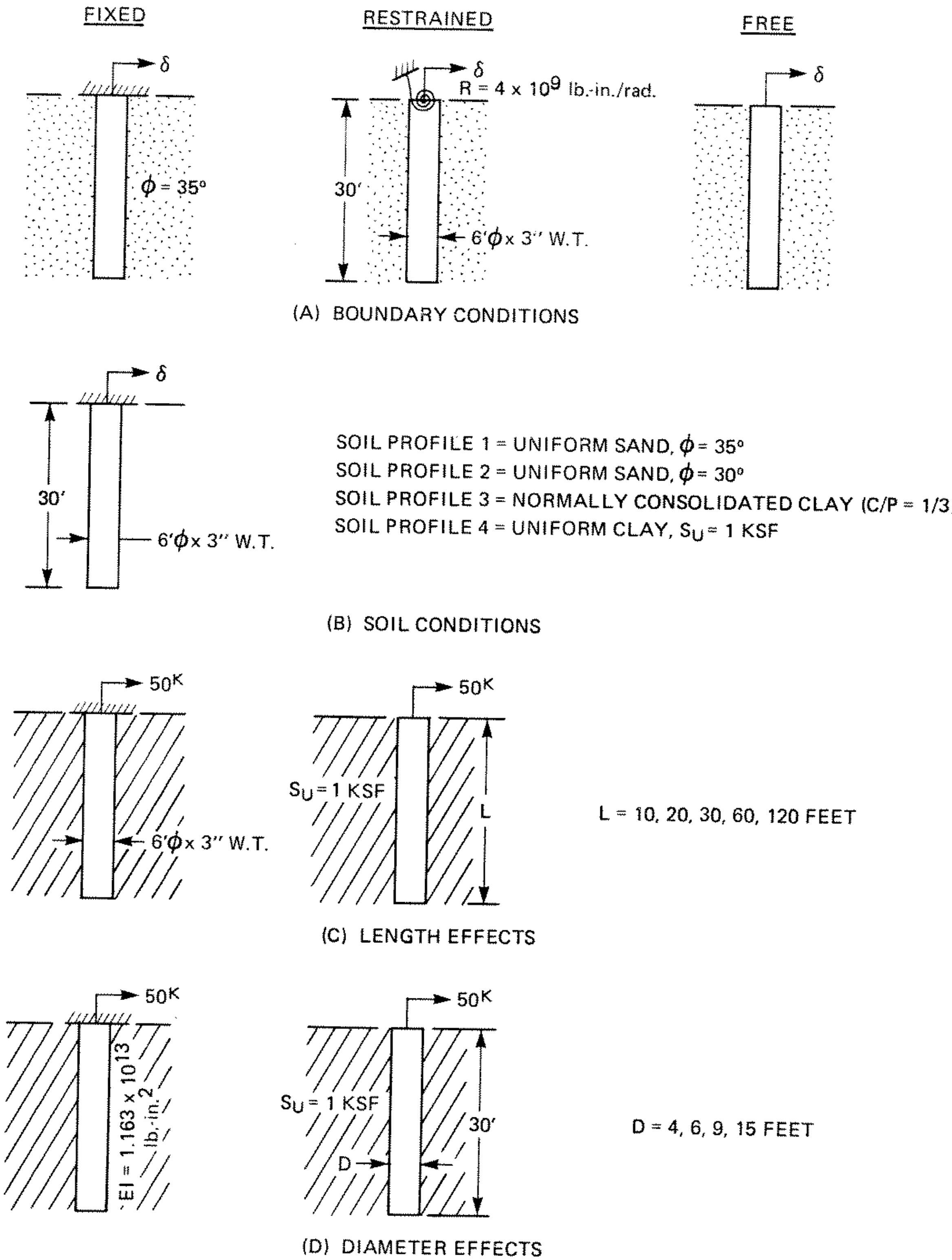


FIGURE 4–1 INPUT PARAMETERS FOR SENSITIVITY STUDY

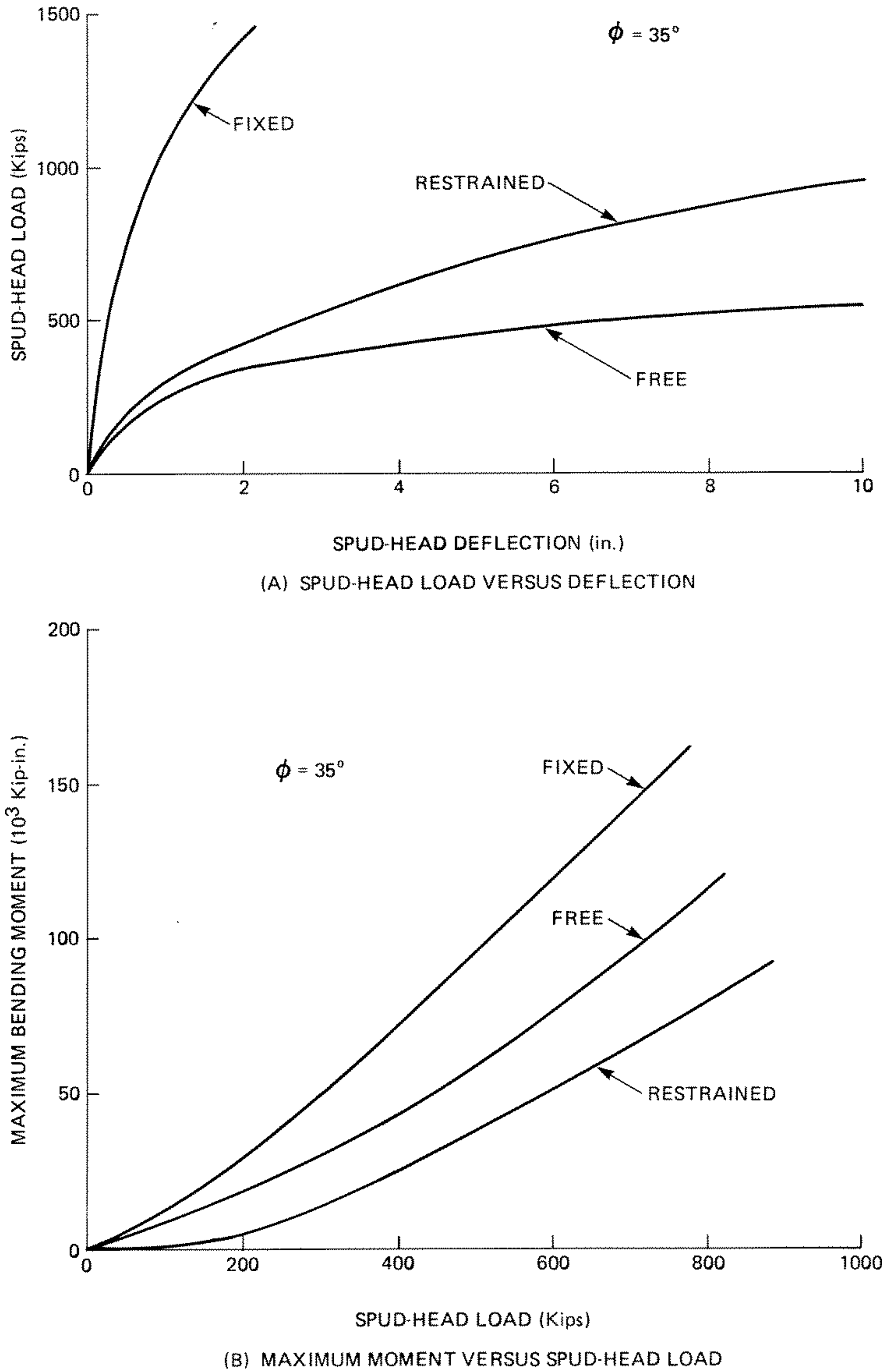


FIGURE 4—2 SENSITIVITY OF BOUNDARY CONDITIONS IN COHESIONLESS SOIL

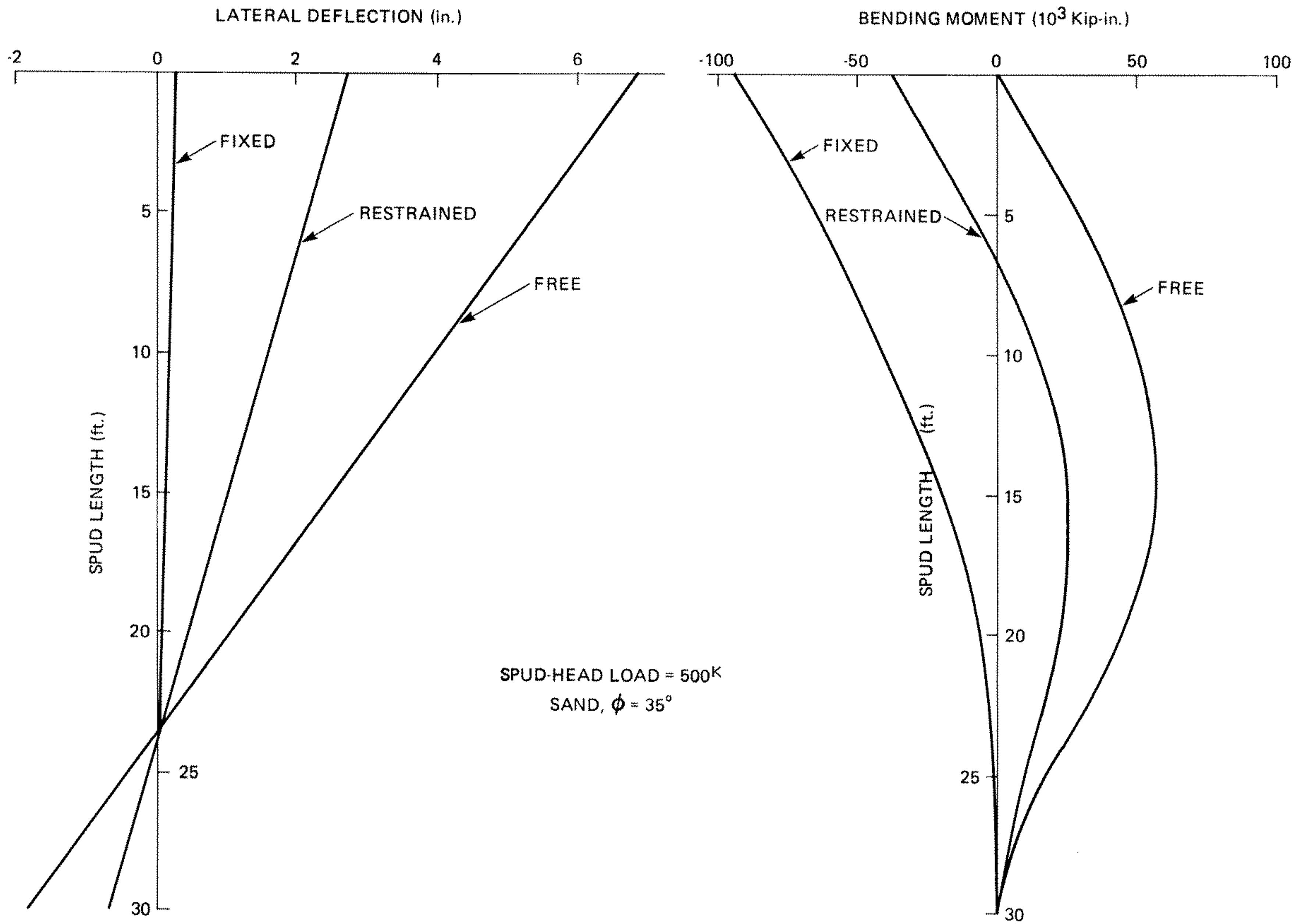


FIGURE 4–3 DEFLECTION AND MOMENT DISTRIBUTIONS ALONG SPUD LENGTH

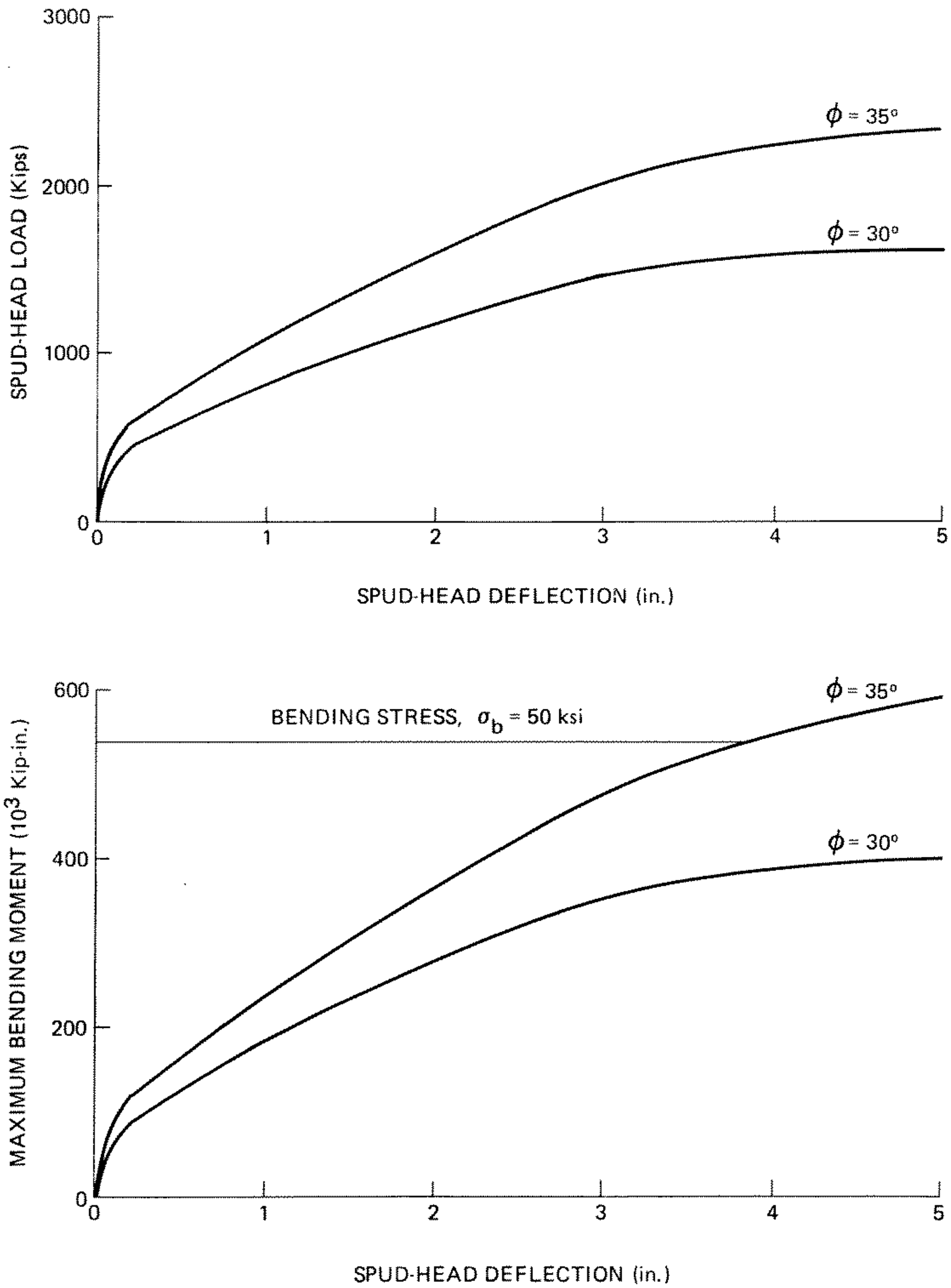


FIGURE 4–4 EFFECTS OF VARIATION IN FRICTION ANGLE FOR A FIXED-HEAD SPUD

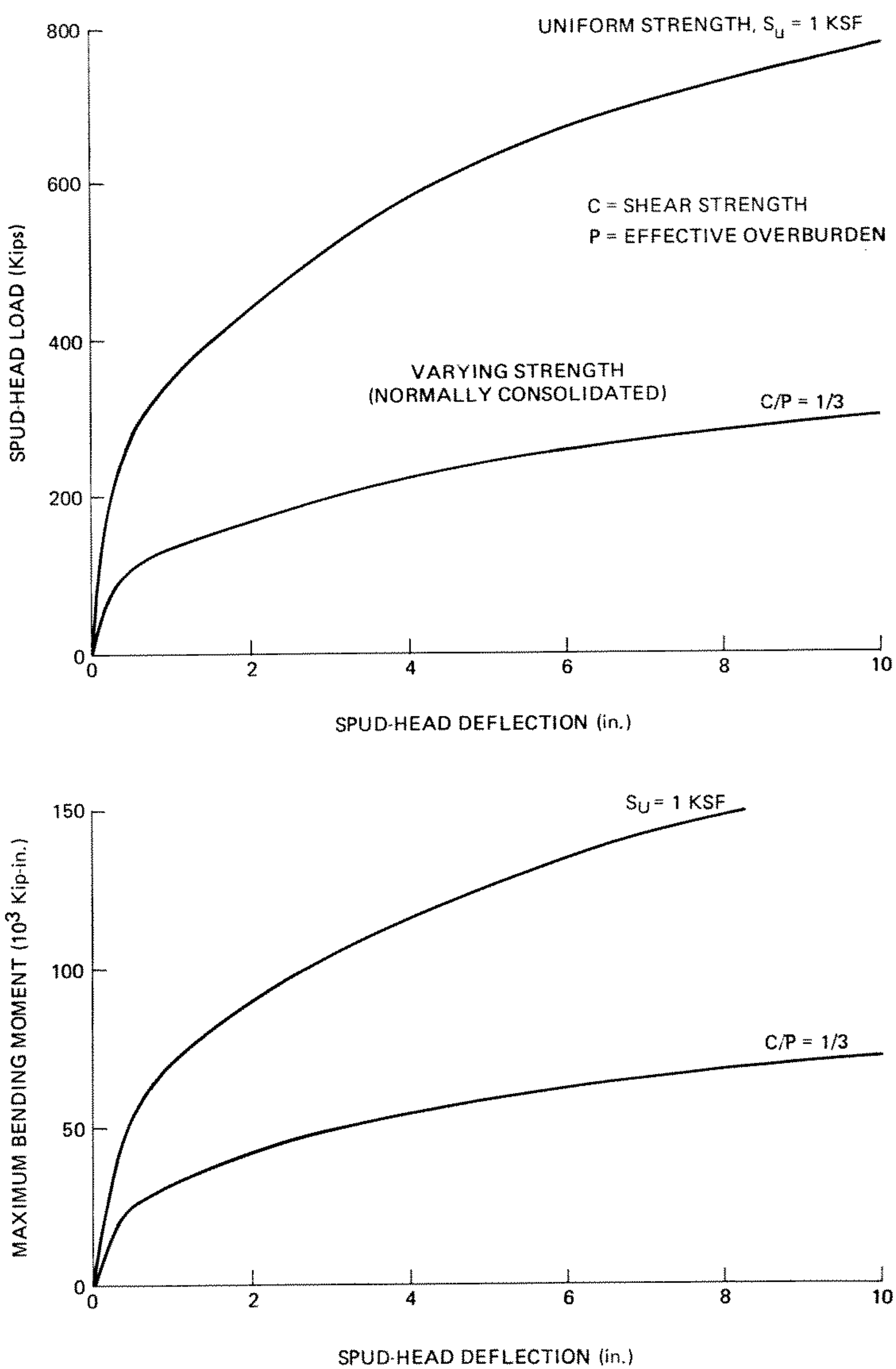
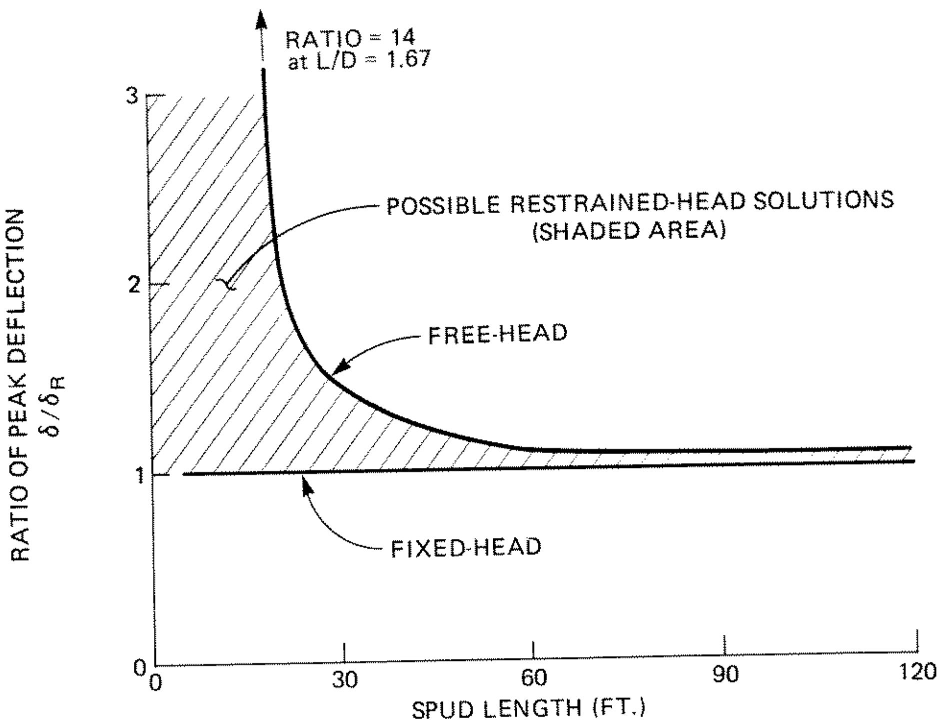
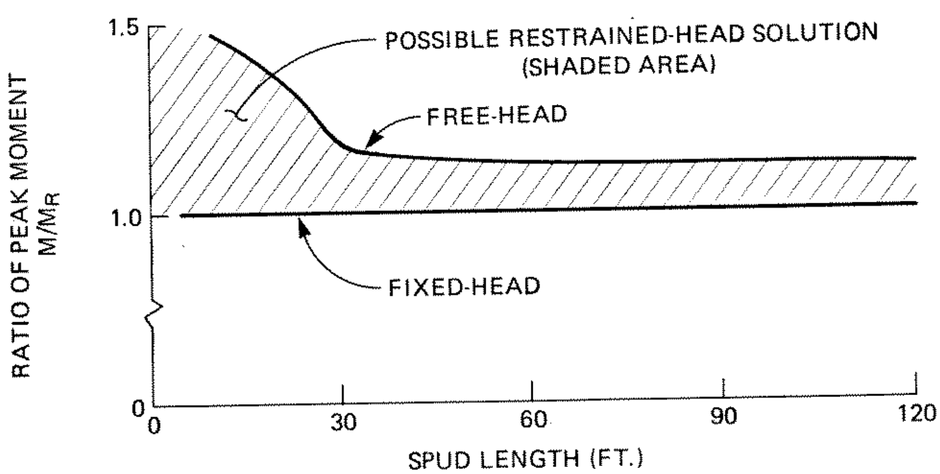


FIGURE 4–5 EFFECTS OF VARIATION IN SHEAR STRENGTH FOR A FIXED-HEAD SPUD



(A) EFFECT OF SPUD LENGTH ON PEAK DEFLECTION



(B) EFFECT OF SPUD LENGTH ON PEAK BENDING MOMENT

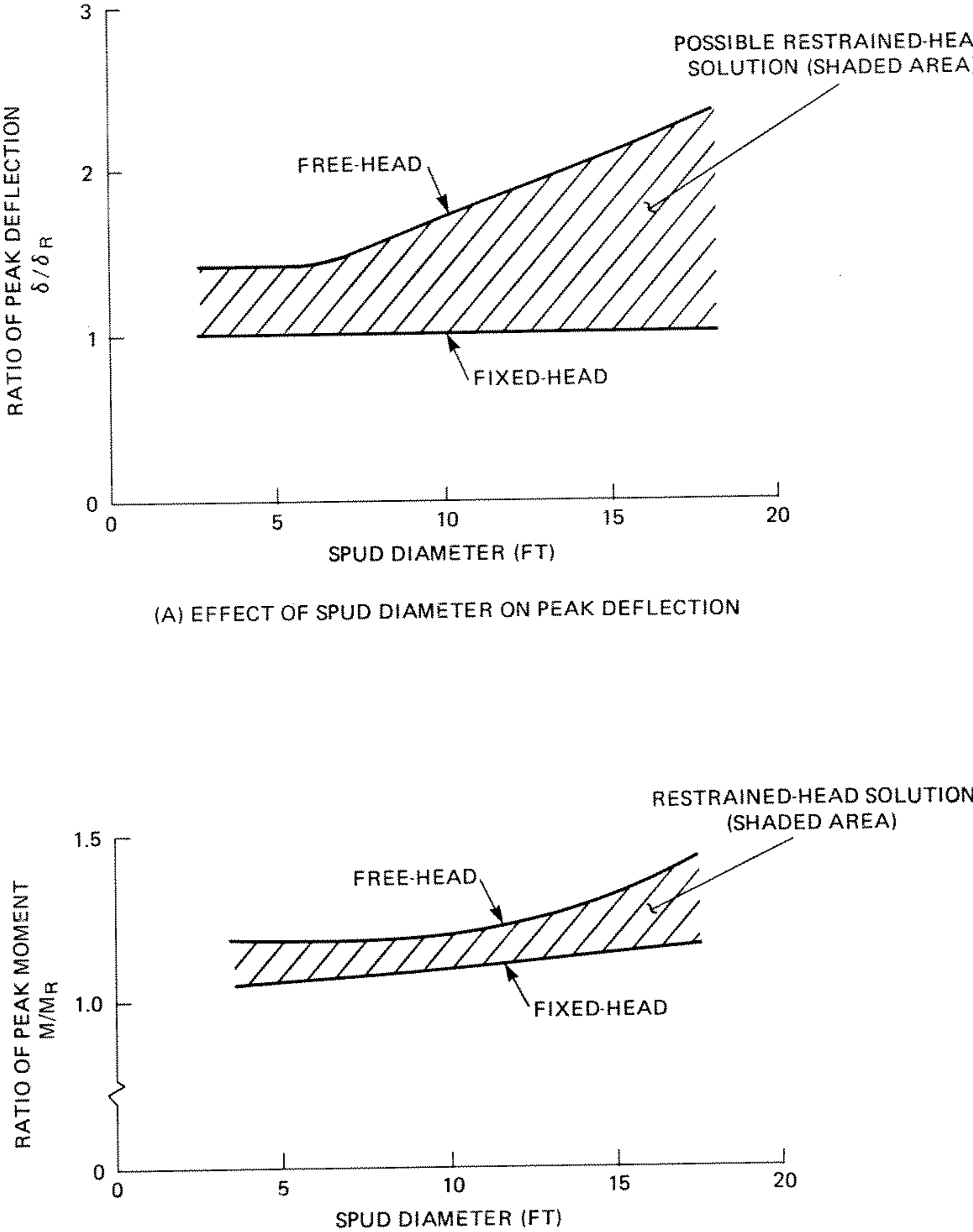
NOTES:

(1) THE SUBSCRIPT R (δ_R, M_R) REPRESENTS SOLUTION WITH CONSIDERATION OF ROTATIONAL REACTION

(2) SOIL IS UNIFORM CLAY WITH S_u = 1 KSF

(3) LATERAL LOADING OF 50 KIPS AT SPUD TOP

FIGURE 4–6 EFFECTS OF SOIL ROTATIONAL REACTION WITH SPUD LENGTH VARIED



(A) EFFECT OF SPUD DIAMETER ON PEAK DEFLECTION

(B) EFFECT OF SPUD DIAMETER ON PEAK BENDING MOMENT

NOTES:

(1) THE SUBSCRIPT R (δ_R, M_R) REPRESENTS SOLUTION WITH CONSIDERATION OF ROTATIONAL REACTION

(2) SOIL IS UNIFORM CLAY WITH S_u = 1 ksf

(3) LATERAL LOADING OF 50 KIPS AT SPUD TOP

FIGURE 4-7 EFFECTS OF SOIL ROTATIONAL REACTION WITH SPUD DIAMETER VARIED

5.0 SPUD APPLICATION FOR ARCTIC STRUCTURE

5.1 Description of Problem

Spuds have recently been proposed as a new foundation concept to increase the lateral capacity of gravity structures in the Beaufort Sea where high lateral forces are generated from sea ice (Bea et al., 1983; Gerwick et al., 1983). The site soil consists of low strength surficial deposits underlain by relatively competent soils or permafrost. Spuds are ideal for this application because enlarging the structure base and dredging the low strength surficial soils are not always desirable due to increase in construction and installation costs. Another benefit is that spud foundations can be designed for on-site adjustments to tailor the design to load requirements. For example, additional load capacity can be obtained either by elongation of the spud length or installation of more spuds through spud slots fabricated in the gravity structure. Spuds can also be extracted and re-installed to allow transportation of the gravity structure to various drill sites.

One of the challenges in the design of this spud system is the uncertainties in load distribution among the spud, structure base, and possibly within the soil itself (slippage of weak zone) and the failure mode. Three potential failure modes were postulated and shown in Figure 5-1:

(1) Spud plowing through soil

(2) Soil-to-soil slippage near spud tip

(3) Soil-to-soil slippage within weak zone

Spuds should be designed with sufficient yield strength so that under extreme loading conditions, failure is in the form of spud plowing through soil to minimize the damage to the structure. Soil slippage near the spud tip may occur because a group of spuds tends to act as a block at close spacing. Subsurface soft cohesive soil layer is likely to be present in some areas in the Beaufort Sea; thus, sliding failure along a thin weak zone is possible. Both the load distribution and failure mode depend on the (1) soil conditions, (2) spud configuration, and (3) structure weight and size.

For design purposes, available experience in gravity structure and general spud behavior alone is insufficient. The conventional limit-equilibrium and effective area concept used for gravity structure (DnV, 1977) cannot consider the added resistance of the spuds. The beam-column approach for spud as described in Chapters 3 and 4 cannot include the induced soil mass deformation and the sliding resistance at the structure base or near the spud tip. To solve the problem, the interaction among the soil, the spud, and the structure base must be analyzed.

5.2 Spud-Structure Interaction

Presently, finite-element analysis and model testing are the only approaches for a rational solution of this interaction problem. Three-dimensional finite-element analysis should be used because the soil movement around the spud and below the structure base must be properly modeled. In terms of experimental work, centrifuge testing is the only valid approach because (1) the size of the prototype structure can be properly modeled, and (2) the soil overburden stress at a scaled distance below the mudline will be identical to the prototype condition. Centrifuge testing on gravity structures (without spuds) have been successfully performed by many researchers (Prevost et al, 1981; Rowe and Craig, 1979; Nolan-Ertec, 1985).

Three-dimensional finite-element analysis is too costly and may require a "super" computer for enormous storage capacity and high-speed number crunching. Therefore, 3-D finite-element analysis is out-of-scope for this study. However, to demonstrate the feasibility of finite-element analysis for this interaction problem, a single example two-dimensional analysis is presented below. For this example, a three-foot thick soft soil overlying stiff soil with a structure surcharge of 0.9 ksf was analyzed. In a 2-D model, the spud can only be modeled by an infinite wall (strip).

The computer program DYNAFLOW was used. This program was developed by Professor Jean H. Prevost (Prevost, 1981) and has been used extensively by The Earth Technology Corporation (Earth Technology 1985b; Nolan-Ertec, 1984). This program contains a two and three-dimensional finite-element model which can efficiently evaluate static and quasi-static loading problems. The program incorporates a variety of soil models including elastic, elastic-plastic and truly nonlinear soil behavior.

Input Data. The undeformed mesh together with some pertinent input data are shown in Figure 5-2. The spud was modeled by a five-foot deep strip (wall) and was fixed (zero rotations) at the strip top. Horizontal loading was controlled by incremental deflections, also imposed at the strip top. Contact elements were used to simulate tensile separation between the strip and soil medium. A fixed base was assumed for the bottom boundary. A roller boundary condition (no lateral movement) was assumed for the two side surfaces.

A multi-linear inelastic model commonly referred to as the Prevost model was used to simulate nonlinear inelastic soil behavior under undrained loading conditions. Details of the Prevost model can be found in the literature (Prevost, 1979). The stiffness and shear strength characteristics for each element were chosen so that the soft soil has a shear strength value close to zero and a shear strength of 4 ksf was used for the stiff soil.

Load-Displacement Relationship. The load-displacement relationship at the spud top for the two-dimensional finite-element solution is shown in Figure 5-3. The ultimate lateral capacity for this problem was 32 kips per foot. The yield displacement is about 0.4 inch.

Deformed Mesh and Displacement Vectors. One set of deformed mesh and displacement vector plots is also presented here for discussion. The scale used for these plots is grossly exaggerated and the results should be interpreted accordingly. The deformed mesh plot for a strip-head displacement of 4.18 inches is shown in Figure 5-4. In this figure, the first three rows of elements represented the soft surficial soil layer. As shown, there is a distinct slippage occurring near the soft and stiff soil interface. This failure envelope is confirmed by the displacement vectors plot in Figure 5-5. As shown by the displacement vectors plot, the slip surface extended at an angle from the strip tip to within the soft soil layer, then it projected horizontally towards the outside boundary.

The above finite-element solution was a good indication of the complexity involved in analyzing the spud-structure interaction problem. It should again be pointed out that the above finite-element solutions were based on a simplified two-dimensional model and the results might not be valid for spuds.

However, the solutions demonstrated that finite-element method can be applied for this interaction problem.

5.3 Preliminary Guidelines

For preliminary analysis, some simplified procedures are available. The structure-spud-soil system can be approximated by an elastic beam on Winkler foundation. However, development of a representative beam model may be questionable because of the gross simplification of the problem. Another approach was proposed by Bea et al (1983). In this model, the soil is modeled by a number of discrete layers. The ultimate soil resistance of each layer is determined as 9 times the shear strength for cohesive soil and for cohesionless soils, Reese's equation (1962) was used. By assigning this ultimate value for all the soil layers, the model assumes a physical barrier of the structure base to allow the soil at the mudline to behave in a confined fashion. In reality, this condition is not always true as shown by the 2-D finite-element solutions. The total resistance of the structure-spud system is then equal to the lateral resistance at the structure base plus the resistance of each of the foundation spud provided the (1) maximum bending moment is within the design limit, and (2) the reaction force on the structure-spud connection is tolerable.

In light of the above uncertainties, a more simplistic approach using conventional analytical procedure for gravity structure and spud response is proposed. The method draws on the concept of limit-equilibrium analysis and the beam-column approach outlined in Chapters 3 and 4.

Guidelines. The following guidelines are meant to be first-cut estimates developed under the constraints of lack of experimental and analytical data for this spud-structure-soil system. The procedure must be checked and modified if necessary when additional data are available.

The beam-column procedure outlined in the earlier chapters can be used to estimate the ultimate lateral resistance of the spud. The same approach can be used to calculate the bending moment along the spud length. Sizing and yield strength of the spud can be determined from the bending moment calculation. Some considerations should be given to the effects of neighboring spuds provided they are spaced apart closer than three times the diameter. These considerations may include adjustments of the characteristics of the p-y and base shear support springs such as modification in the ultimate resistance as well as stiffness. Some recommendations for group effects are given in the literature (O'Neill, 1983).

To calculate the total resistance of the system (structure, spuds and soil), an optimization procedure must be used which considers the failure mode. Specifically, the three potential failure modes in Figure 5-1 are evaluated separately; the design requirements will then be based on the case which results in the lowest lateral capacity. Some details of this calculation procedure to evaluate the lateral capacity for each of the mentioned failure modes are given below.

Spud Plowing Through Soil. For this case, the beam-column approach is used to calculate the ultimate resistance of each spud. The structure surcharge should be accounted for in the development of the p-y curves by including the overburden at the soil surface due to the gravity force of the structure. Proper boundary condition must be used to model the spud-structure connection. The boundary condition can be (1) free head, (2) fixed bead, or (3) restrained head. The resistance from all the spuds is obtained by multiplying the individual spud resistance by the total number of spuds.

The resistance at the structure base is computed using either Equation 3.1 or 3.2 depending on the surficial soil conditions. The total lateral capacity of the gravity structure is the sum of the total spud resistance and the structure base resistance.

Soil-to-Soil Slippage Near Spud Tip. Stability against sliding near the spud tip can be evaluated using classical slope-stability formulae. This method of analysis begins with the assumption of the shape of the slip surface; the shape may be assumed circular, logarithmic spiral, sliding block, or wedges. Then, a set of equations based on static equilibrium are solved to determine the overall stability of the sliding mass just on the verge of slip. This procedure can be found in many references (Bishop, 1955; DM-7, 1971) and therefore is neither discussed nor demonstrated in detail here.

Soil-to-Soil Slippage Within a Weak Zone. If the weak zone is below the spud tip, the stability analysis discussed above can be used. If the spud penetrates past a weak zone, then a slight modification of the assumed slip surface is needed (see Figure 5-1). In addition to the slippage within the weak zone, the passive resistance against the spud must be included in the analysis.

The above computations for lateral capacity of a gravity structure anchored by spuds considered static loading only. Cyclic loading effects can be accounted for in an approximate fashion by lowering the strength parameters of the foundation soils. This procedure is frequently adopted for analysis of gravity structure under wave loading (Nolan-Ertec, 1981; Rahman et al., 1979).

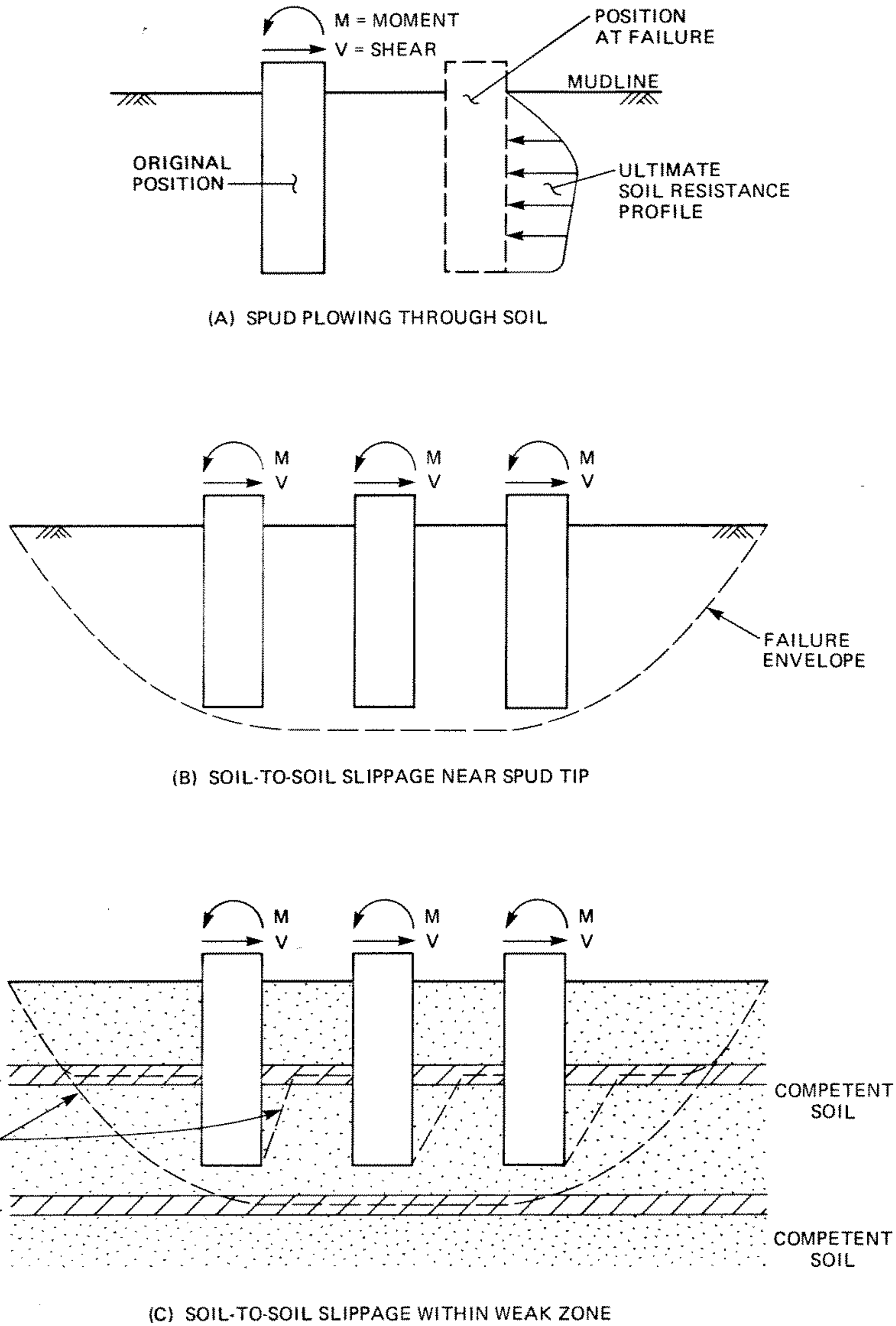


FIGURE 5–1 POSSIBLE FAILURE MECHANISMS

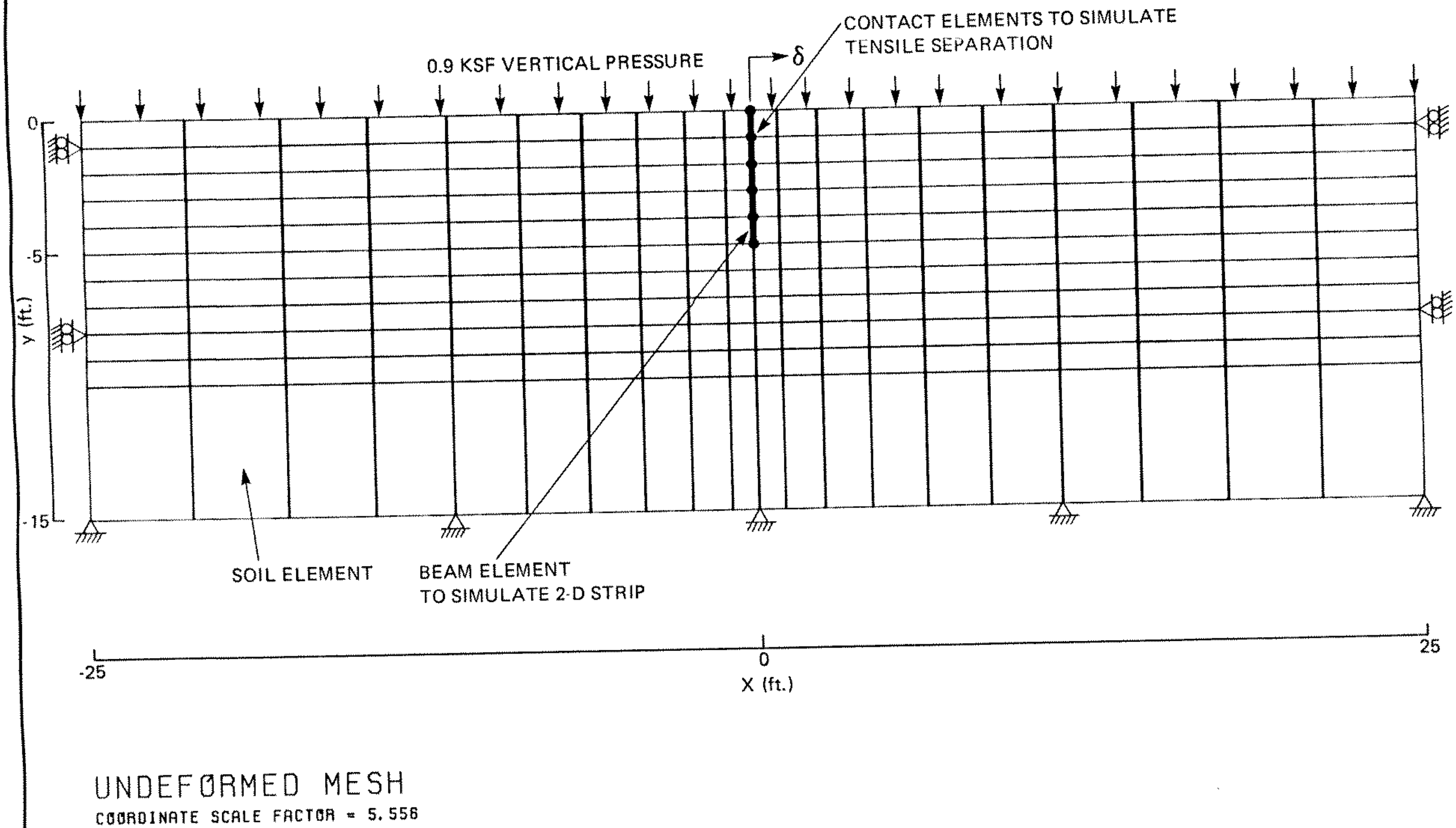


FIGURE 5–2 PROBLEM DESCRIPTION FOR FINITE-ELEMENT ANALYSIS

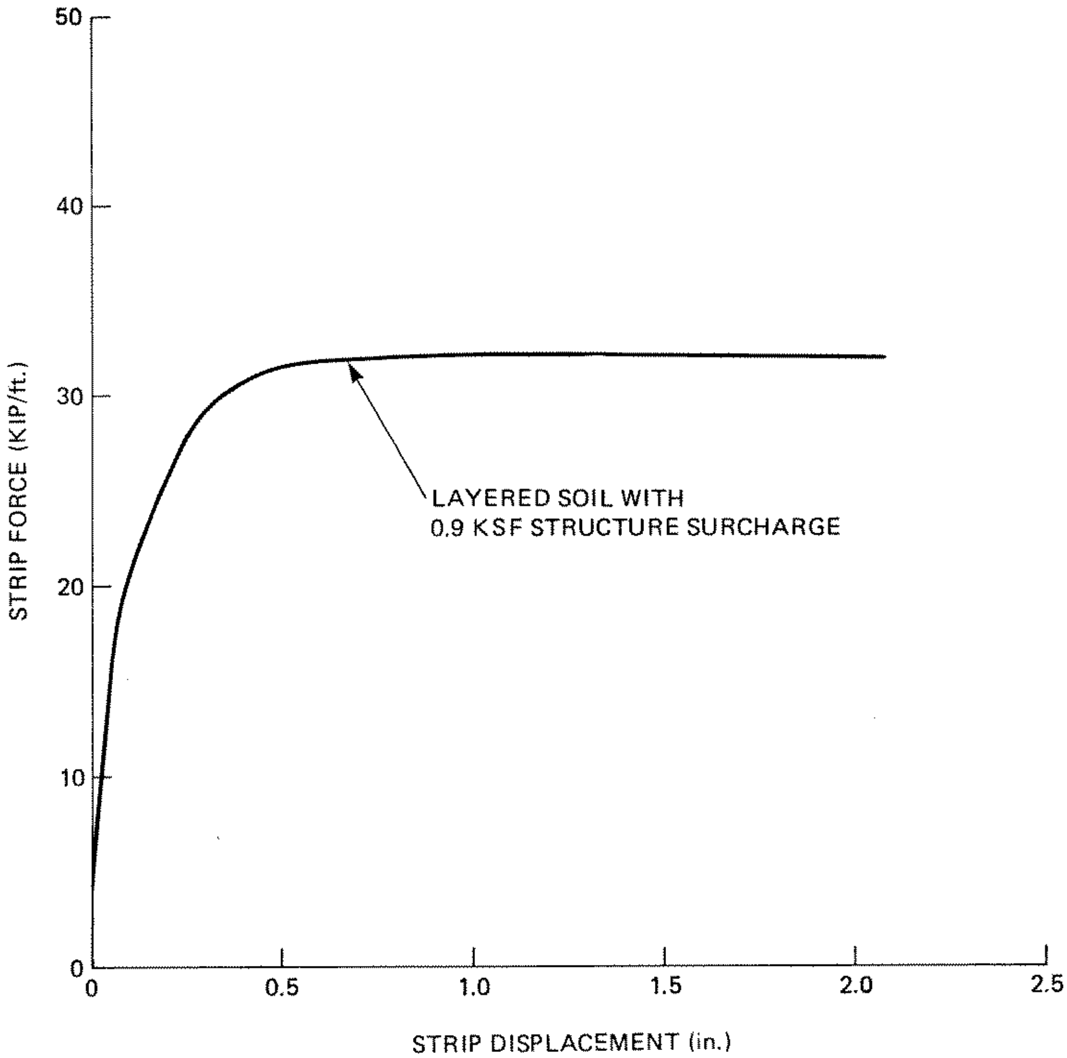


FIGURE 5–3 2-D STRIP LOAD-DISPLACEMENT RELATIONSHIPS

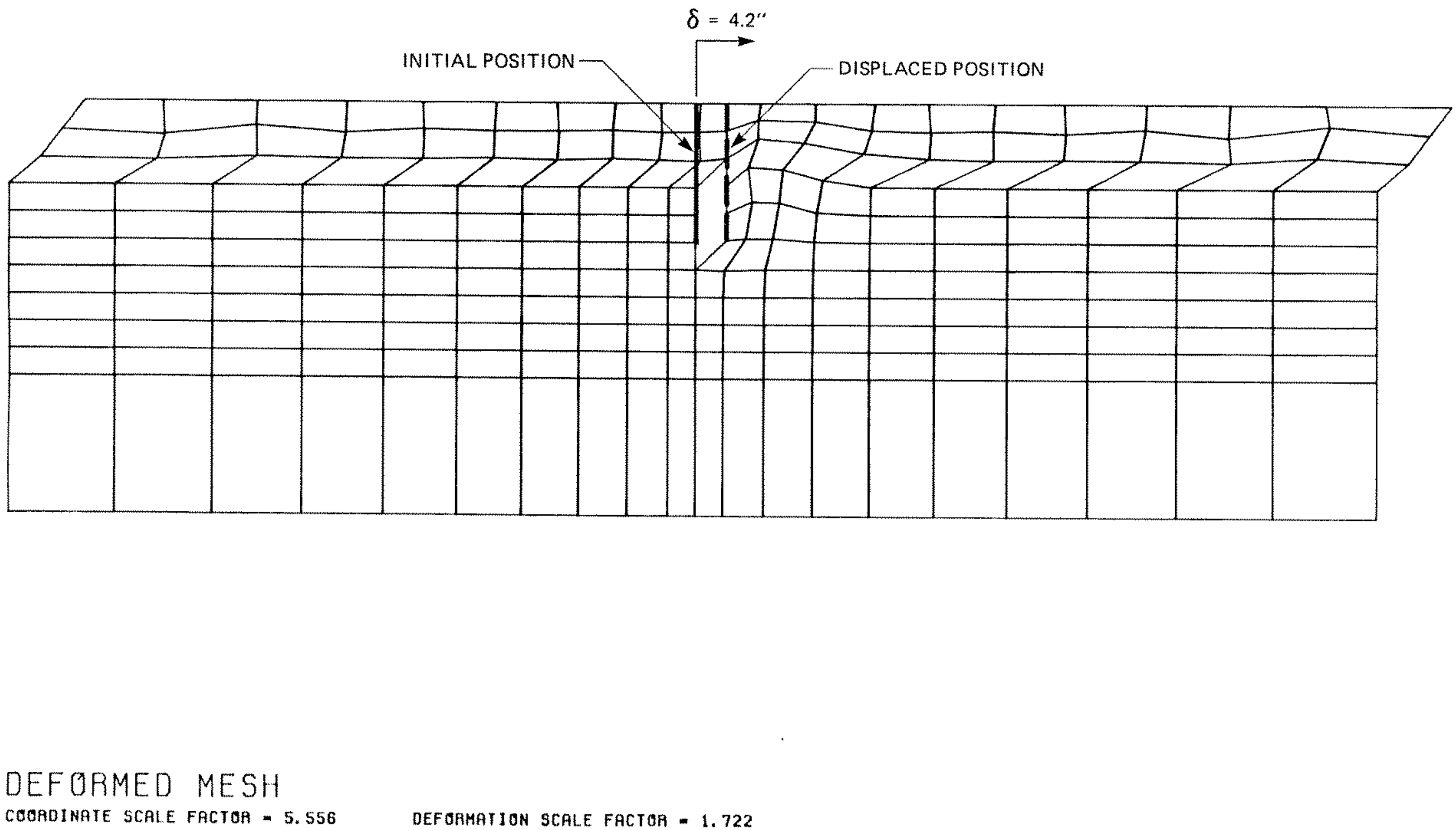


FIGURE 5–4 DEFORMED MESH PLOT
(LAYERED SOIL WITH 0.9 KSF STRUCTURE SURCHARGE)

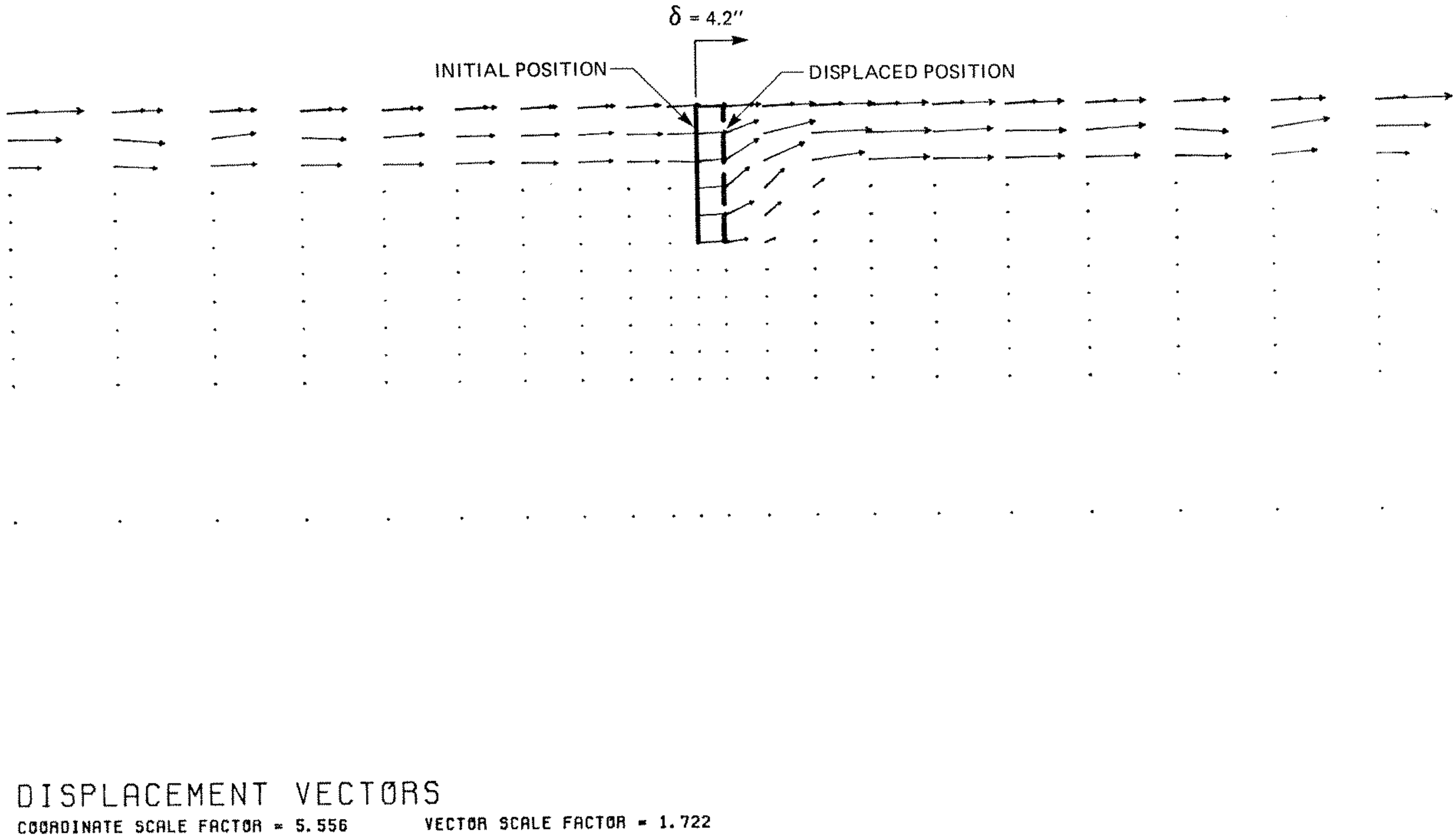


FIGURE 5–5 DISPLACEMENT VECTORS PLOT
(LAYERED SOIL WITH 0.9 KSF STRUCTURE SURCHARGE)

6.0 CONCLUSIONS AND RECOMMENDATIONS

6.1 General

Recent expansion of offshore drilling activities at the Arctic region has introduced many unique engineering challenges. For example, short, large-diameter piles (spuds) have been proposed to increase the lateral load capacity of a gravity structure. Response of these spuds under large lateral loads may be quite different from long slender piles. Thus, a proposal was submitted by The Earth Technology Corporation to the Minerals Management Service of the U.S. Department of Interior to develop design guidelines for spuds. This project was initiated in September 1984 and the results are documented in this report.

The behavior of short, large-diameter piles (spuds) has been investigated by means of a literature survey, beam-column analysis using a simplified discrete-element model and finite-element analysis. The literature survey was intended to summarize the state-of-knowledge for spud design under lateral loading. Experiments and theoretical solutions were reviewed for dowels, drilled piers, rigid piles and footings.

Some comparison analyses with drilled pier test data were performed using a discrete-element model to investigate the contribution of various components of soil reactions: p-y, rotation and base shear. Excellent correlation was obtained between the calculated and measured values. Some preliminary design guidelines were established based on the results of this comparison analysis. These guidelines were then applied to solve for the behavior of a single spud under lateral loading. A sensitivity study including variations in

(1) boundary conditions, (2) soil conditions, and (3) length and diameter was also performed for the single spud analysis.

Finite-element analyses were conducted to model a two-dimensional skirt for a gravity structure. The analytical solution was only used to demonstrate the feasibility of the finite-element method; the results were not used to derive any design guidelines.

Some of the pertinent conclusions obtained from the literature survey and the beam-column analyses are given below, together with a short summary on the guidelines for the spud-structure interaction problem. Recommendations are also included to promote additional studies.

6.2 Conclusions

Literature Survey. Mathematical solutions are available for rigid piles and drilled piers based on theories developed for long, slender piles. Validity of these solutions is questionable because no comparison was made with prototype test data. No experimental data could be found for gravity structure anchored by spud group. However, a heavy emphasis on rigid drilled pier testing was evident from the survey because drilled piers are more widely used.

Beam-Column Analysis of a Single Spud. A computer model was developed and checked using available rigid drilled pier test data to analyze the behavior of a single spud under lateral loading. A discrete-element model was used with nonlinear-elastic supports to represent the foundation resistance. This resistance included (p-y) and rotational components and base shear at the spud tip. Results from the drilled pier comparison analysis and a sensitivity study conducted for a single spud are summarized below.

(1) The beam-column approach documented in this report can be used to evaluate the behavior of a single spud.

(2) The rotational reaction must be considered in cohesive soils but can be neglected for cohesionless soil.

(3) The rotational reaction can be neglected for a fixed-head spud independent of the soil type because the rotational deformation of the spud is generally small.

(4) The base-shear component at the spud tip should be included in the analysis.

(5) Proper simulation of the boundary condition is important for assessment of magnitude and distribution of bending moment on the spud.

Guidelines. Preliminary guidelines for the determination of the lateral capacity of the gravity structure required an evaluation of three possible failure modes: spud plowing through soil and soil slipping near the spud tip and within a weak zone. For design purposes, the lowest of the three failure loads should be used.

The beam-column procedure can be used to analyze the case of a spud plowing through soil. Conventional slope stability analysis using the slip circle concept can be used to evaluate the soil-to-soil slippage problem.

As shown by the above conclusions, practical solution is available to analyze the behavior of a single spud. However, for the Arctic structure-spud system, additional data must be obtained to finalize a more definite design procedure. Yet at this moment, reliance should be placed on the simple guidelines summarized in this report. In view of the requirement for additional data, some recommendations for future work are given below.

6.3 Recommendations

In general, more experimental and analytical data are necessary to advance the current understanding of the spud problem. Specifically, the following tasks should be performed:

(1) Study the effects of closely-spaced spud groups.

(2) Perform three-dimensional finite-element analysis to study the spud-structure-soil interaction problem. Emphasis should be placed on the effects of

- o spud group,
- o structure surcharge, and
- o soil conditions.

(3) Perform centrifuge tests to calibrate the finite-element analysis.

(6) Improve on the current guidelines for spud design.

Recommendations to achieve the above objectives are elaborated below.

At this moment, group effects are being evaluated for long, slender piles only. Recommendations on group effects are mostly based on analytical models (O'Neill, 1983). Some of these concepts may be extended to study spud group effects.

The computer program, DYNAFLOW (Prevost, 1981), should again be used for the three-dimensional finite-element analyses. In the three-dmensional model, the problem should be simplified to minimize the cost and set-up effort of the computer run. These finite-element analyses should be guided by experimental data. Experiments should focus on centrifuge testing. In these experiments, the load-movement behavior should be monitored as well as the pressure distribution along the spud length. The contact pressure and shear at the base of the gravity structure should also be measured. To observe the pattern of soil flow and slip surfaces at failure, colored soil layers can be used. In summary, three-dimensional finite-element analysis and centrifuge tests are vital for the solution of the spud-structure interaction problem. It can be envisioned that these results can be summarized to establish a more general design guideline for spuds.

7.0 REFERENCES

Adams, J. I., and H. S. Radhakrishna (1973), "The Lateral Capacity of Deep Augered Footings," Proceedings, 8th International Conference of Soil Mechanics and Foundation Engineering, Moscow, ISSMFE, Vol. 2.1, pp. 1-8.

American Petroleum Institute (API), (1984), "API Recommended Practice for Planning, Designing, and Constructing Fixed Offshore Platforms," Washington, D.C., Fifteenth Edition.

Barton, Y. O., W. D. Liam Finn, R.H.G. Parry and J. Towhata (1983), "Lateral Pile Response and p-y Curves from Centrifuge Tests," Proceedings, 15th Annual Offshore Technology Conference, Houston, Texas, May.

Bea, R. G., et al. (1983), "Design for Dynamic Loadings," Presented at Conference on Geotechnical Practice in Offshore Engineering, Austin, Texas, April 27-29.

Bhushan, K., et al. (1979), "Lateral Load Tests on Drilled Piers in Stiff Clays," Journal of the Geotechnical Engineering Division, ASCE, Vol. 105, No. GT8, pp. 969-985, August.

Bhushan, K., et al. (1981), "Lateral Load Tests on Drilled Piers in Sand," Proceedings of a Session on Drilled Piers and Caissons, St. Louis, Missouri, October 28.

Bierschwale, M. W., et al. (1981), "Lateral Load Tests on Drilled Shafts Founded in Clay," Proceedings of a Session on Drilled Piers and Caissons, St. Louis, Missouri, October 28.

Bishop, A. W. (1955), "The Use of the Slip Circle in the Stability Analysis of Slopes," Geotechnique, 5, 7-17.

Bogard, Dewaine and H. Matlock (1983), "Procedures for Analysis of Laterally Loaded Pile Groups in Soft Clay," Proceedings, Conference on Geotechnical Practice in Offshore Engineering, University of Texas at Austin, Austin, Texas, April.

Bogard, D., and H. Matlock (1980), "Simplified Calculation of p-y Curves for Laterally Loaded Piles in Sand," In House Report, The Earth Technology Corporation, April.

Brinch Hansen, J. (1961), "The Ultimate Resistance of Rigid Piles Against Transversal Forces," The Danish Geotechnical Institute Bulletin, No. 12, pp. 5-9.

Broms, B. (1964a), "Lateral Resistance of Piles in Cohesive Soils," Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 90, No. SM2, pp. 27-63, March.

Broms, B. (1964b), "Lateral Resistance of Piles in Cohesionless Soils," Journal of Soil Mechanics and Foundation Division, ASCE, Vol. 90, No. SM3, pp. 1233-1256, May.

Coyle, H. M., and L. C. Reese (1966), "Load Transfer for Axially Loaded Piles in Clay," Journal of the Soil Mechanics and Foundation Division, ASCE, Vol. 92, No. SM2, March.

Czernicak, E., (1957), "Resistance to Overturning of Single, Short Piles," Journal of Structural Division, ASCE, Vol. 83, No. ST2, pp. 1188-1 to 1188-25, March.

Davisson, M. T., and J. R. Salley (1968), "Lateral Load Tests of Drilled Piers," Proceedings, 71st Annual Meeting American Society for Testing and Materials, San Francisco, California, No. STP 444, pp. 68-83, June 23-28.

Det Norske Veritas, DnV, (1977), "Rules for the Design, Construction, and Inspection of Offshore Structures," May 1.

DM-7 (1971), "Design Manual, Soil Mechanics, Foundations, and Earth Structures," NAVFAC DM-7, Department of the Navy, Naval Facilities Engineering Command, Alexandria, VA.

Douglas, D. J., and E. H. Davis (1964), "The Movement of Buried Footings Due to Moment and Horizontal Load and the Movement of Anchor Plates," Geotechnique, Vol. 14, No. 2, June.

Electric Power Research Institute, EPRI (1982), "Laterally Loaded Drilled Pier Research," Final Report, Prepared by GAI Consultants, No. EL-2197, Vol. 2, Research Project 1280-1, January.

Gerwick, B. C., Jr., et al. (1983), "Application of Multiple Spuds (Dowels) to Development of Sliding Resistance of Gravity Base Platforms," Proceedings, 15th Annual Offshore Technology Conference, Houston, Texas, pp. 347-351, May 2-5.

Hetenyi, M. (1946), Beam on Elastic Foundation, The University of Michigan Press, Ann Arbor, Michigan.

Ivey, D. L. (1968), "Theory, Resistance of a Drilled Shaft Footing to Over-Turning Loads," Texas Transportation Institute, Research Report No. 105-1, February.

Kraft, L. M., R. P. Ray and T. Kagawa (1981), "Theoretical t-z Curves," Journal of the Geotechnical Engineering Division, ASCE, Vol. 107, No. GT11, November.

Lubking, P. (1977), "Horizontal Loading Tests on Large Diameter Piles," Proceeding, 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Japan, ISSMFE, Vol. 1, pp. 629-632.

Matlock, H. (1970), "Correlations for Design of Laterally Loaded Piles in Soft Clay," Proceedings, 2nd Annual Offshore Technology Conference, Houston, Texas, pp. 577-594.

Matlock, H., D. Bogard and I. P. Lam (1981), "BMCOL76: A Computer Program for the Analysis of Beam-Columns Under Static Axial and Lateral Loading," Documentation Report, June.

Mindlin, R. D. (1936), "Force at a Point in the Interior of a Semi-Infinite Solid," Physics, Vol. 7, pp. 195-202.

Nolan-Ertec, Ltd. (1981), "Hibernia Gravity-Base Structure, Preliminary Foundation Evaluation," Report to Mobil Oil Canada, Ltd., July.

Nolan-Ertec, Ltd. (1984), "Hibernia Gravity-Base Structure Preliminary Foundation Design and Analysis," Report to NODECO, St. John's, Newfoundland.

Nolan-Ertec, Ltd. (1985), "Hibernia Gravity-Base Structure, Foundation Model Test Study," A report to Mobil Oil Canada, Ltd., February.

Norwegian Geotechnical Institute, and Det Norske Veritas (1982), "Analysis of Full-Scale Measurements on Gravity Platforms," Final Report Foundation Performance During Installation and Operation of North Sea Concrete Gravity Platforms.

O'Neill, Michael W. (1983), "Group Action in Offshore Piles," Proceedings, Conference on Geotechnical Practice in Offshore Engineering, University of Texas at Austin, Austin, Texas, April.

O'Neill, M. W., and J. M. Murchison (1983), "An Evaluation of p-y Relationships in Sands," A Report to the American Petroleum Institute (PRAC 82-41-1), May.

O'Neill, Michael W., and Sal M. Gazioglu (1984), "An Evaluation of p-y Relationships in Clays," A report to the American Petroleum Institute, University of Houston, Houston, Texas, April.

Parker, F., Jr., and L. C. Reese (1970), "Experimental and Analytical Studies of Behavior of Single Piles in Sand Under Lateral and Axial Loading," Research Report 117-2, Center for Highway Research, The University of Texas at Austin, November.

Poulos, H. G. (1971), "Behavior of Laterally Loaded Piles: I-Single Piles," Journal of Soil Mechanics and Foundation Engineering Division, ASCE, Vol. 97, No. SM5, May.

Prevost, J. H. (1981), "DYNA-FLOW: A Nonlinear Transient Finite Element Analysis Program," Dept. Civil Eng., Princeton University.

Prevost, J. H. (1979), "Mathematical Modeling of Soil Stress-Strain Strength Behavior," Proceedings, 3rd Int. Conf. Num. Methods in Geomechanics, Aachen, W. Wittke (Ed.), Balkema Press, Vol. 1, pp. 347-361.

Prevost, J. H., B. Cuny, and R. F. Scott (1981), "Offshore Gravity Structures: Centrifugal Modeling," Journal of the Geotechnical Engineering Division, ASCE, Vol. 107, No. GT2, February.

Rahman, M. S., H. Bolton Seed, and John R. Booker (1976), "Liquefaction Analysis for Offshore Oil Tanks," A Report on Research Sponsored by The National Science Foundation, Report No. TE76-2, University of California, Berkeley, September.

Reese, L. C. (1962), "Ultimate Resistance Against a Rigid Cylinder Moving Laterally in a Cohesionless Soil," Journal of Society of Petroleum Engineers, December.

Reese, L. C., et al. (1974), "Analysis of Laterally Loaded Piles in Sand," Preprints, 6th Annual Offshore Technology Conference, Houston, Texas, pp. 473-483.

Reese, L. C., and J. D. Allen (1977), "Structural Analysis and Design for Lateral Loading," published in Drilled Shaft Design and Construction Guidelines Manual, U.S. Department of Transportation, Vol. 2, July.

Reese, L. C., and R. Welch (1975), "Lateral Loading of Deep Foundations in Stiff Clay," Journal of Geotechnical Engineering Division, ASCE, Vol. 101, No. GT7, pp. 633-649, July.

Reese, L. C., and S. J. Wright (1977), "Drilled Shaft Design and Construction Guidelines Manual, Vol. One," Submitted to U.S. Department of Transportation, FHWA, Washington, D.C., July.

Rowe, P. W., and W.H. Craig (1979), "Applications of Models to Predictions of Offshore Gravity Platform Foundation Performance," Proceedings, International Conference Offshore Site Investigation, London, England, March.

Terzaghi, K. (1955), "Evaluation of Coefficients of Subgrade Reaction," Geotechnique, Vol. 5.

The Earth Technology Corporation (1983a), "Supplementary Geotechnical Evaluations for SAMS - Spud Foundation," Report to SOHIO Construction Company, San Francisco, California.

The Earth Technology Corporation (1983b), "Static and Cyclic Lateral Load Tests on Instrumented Piles in Sand," A Joint-Industry Research Report, October.

The Earth Technology Corporation (1985a), "Draft Report, Highway Bridge Foundation Design and Analysis to Resist Earthquake Motion," Submitted to FHWA, September.

The Earth Technology Corporation (1985b), "Volume I. Input Guide for Program DIRTMOD," a documentation report to Mobil Research and Development Corporation, March.

Thompson, G. R., (1960), "Application of the Finite Element Method to the Development of p-y Curves for Saturated Clays," M.S. Thesis, University of Texas at Austin, May.

Vijayvergiya, V. N. (1977), "Load-Movement Characteristics of Piles," Paper presented at the PORT 77 Conference, Long Beach, California, March.