PILE FOUNDATION DESIGN: INTERRELATION OF SAFETY MEASURES FROM DETERMINISTIC AND RELIABILITY-BASED METHODS

by

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ABSTRACT

PILE FOUNDATION DESIGN: INTERRELATION OF SAFETY MEASURES FROM DETERMINISTIC AND RELIABILITY-BASED METHODS

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An algorithm is developed to interrelate the basic concepts and procedures commonly used in deterministic design of axially-loaded piles in cohesionless soil with reliability-based design methods. The derivation of the algorithm uses the commonly accepted "standard" static formula (Beta-Method) and the in-situ soil exploration data from Standard Penetration Test (SPT).

For the determination of the allowable capacity ($\mathbf{Q_a}$) in the design of a single pile foundation, safety measures are frequently applied to the predicted capacity ($\mathbf{Q_p}$). To determine $\mathbf{Q_a}$, this study presents the interrelationship between the conventional deterministic safety measure (Factor of Safety, **FS**) and reliability-based safety measures (Central Factor of Safety, **CFS**, Reliability Index, $\boldsymbol{\beta}$, and the Probability of Failure, $\mathbf{P_f}$) within a systematic and rational framework. Interrelation equations and design charts are derived and calibrated from 23 pile loading tests, 11 of which are on instrumented piles. Consequently, at the recommended **FS** or $\boldsymbol{\beta}$, the value of $\mathbf{Q_a}$ can be determined and interpreted by either deterministic or reliability-based approaches.

DEDICATION

To *ayah*, Haji Daud Mamat
(may your soul rest in peace under His mercy)

> ... The constant memory of your wisdom and vision to excel (worldly and hereafter) has helped me accomplish this study.

To my parents, "... My Lord! Bestow upon them mercy as they pampered me since childhood." (Quran: 17,24).

To my dearest wife, N. Hasimah; daughter, Siti Nurrasyida; son, Muhammad Firman; daughter, Nurrasyila; and the baby to come.

> ... who patiently endure riding the troughs and crests of the dissertation wave.

May the Almighty make this world
a "greener" place to live in - Amin!

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CHAPTER 1

INTRODUCTION

1.1 Introduction

A considerable number of probability or reliability models have been proposed over the years that can be applied in geotechnical engineering. For example, statistical approaches have been applied for more than two decades to model physical characteristics of soils, to recommend number of soil tests, and to quantify characteristics of soil, earthwork, slope stability, settlements and the bearing capacity of pile foundations (e.g., Zlatarev, 1965; Langejan, 1965; Lumb, 1966; Holtz and Krizek, 1971; Kay, 1976; Kay, 1977; Evangelista et al, 1977; Madhav and Arumugum, 1979; Biarez and Favre, 1981; Jaeger and Bakht, 1983; Sidi and Tang, 1987 Rethati, 1988; Alonso and Krizek, 1975; Wolff, 1989; and Wolff and Wang, 1992).

The determination of the allowable capacity for pile foundations may be done by two approaches, namely; the deterministic (conventional) and reliability-based (probability) approaches. From a practical standpoint, the conventional method produces satisfactory designs; and with few reported failures. Alternatively, design by a reliability-based approach may provide more information; but have its own limitations, and the methods are not generally understood by designers. Consequently, pile foundations are not routinely designed by the reliability-based approach. At present, reliability methods remain largely for research purposes; and today there is no code available to guide design engineers. However, with recent improvement in reliability theories and with an increase in computational efficiency, reliability methods continue to gain acceptance and are expected to be the trend in future.

The algorithm developed in this study could serve as a bridge between these two approaches. It is a combined deterministic-reliability approach that is easily used by design practitioners, while at the same time, is capable of accommodating the conventional principles.

This study will focus on a practical combined application of the deterministic and reliability approaches rather than on the validity of their theories.

1.2 Background

Engineers are faced with the challenge of designing safe but yet economically feasible structures. For the design of pile foundations large uncertainties are involved; such as, soil properties and conditions, applied loads, pile material properties, methods of construction, and including the choice of design approaches. In general, the design of pile foundations should ensure adequate safety measure against potential failure. The appropriate safety measure depends on the importance of the structures, possible losses in case of failure, and the certainty of the information regarding soil and environmental conditions. The evaluation of safety measures is more complex in pile foundations as compared to shallow foundations because of the larger uncertainty in the soil-pile interaction; and the structure is likely to be more expensive when piles are used (Bowles, 1988). For pile foundations, the factor of safety commonly recommended in practice ranges from 2.0 to 4.0, depending on local practice and the level of "designers' uncertainties" (e.g., Peck et al, 1974; Meyerhof, 1976a; Fuller, 1978; Poulos and Davis, 1980; Poulos, 1981; McClelland and Reifel, 1986; Tomlinson, 1986; Bowles, 1988).

Over the years, many deterministic methods have been proposed for calculating or estimating the axial capacity of a single pile foundation. Static formulas were established using the accepted theory of soil mechanics, often verified and calibrated with the "true" or "actual" capacity measured from the full scale loading tests (e.g., Meyerhof, 1951; Berezantsev, 1961; Vesic, 1963; Nordlund, 1966; Coyle and Reese, 1966; Housel, 1966; Olson and Flaate 1967; D'Appolonia, 1968; Meyerhof, 1976a; Coyle and Castello, 1981; Kulhawy, 1984; Dennis and Olson, 1985; Focht and Kraft, 1986; McClelland and Reifel, 1986; API, 1991; and Coyle and Ungaro, 1991). Alternatively, the capacity of a pile may also be estimated from dynamic formulas from the driving data by using concepts from dynamics. Recently, a more rigorous method that is becoming more acceptable with improved computing capabilities is estimating pile capacity using the one dimensional wave equation (e.g., Smith, 1960; Samson et al, 1963; Housel, 1966; Lowery et al, 1969; Rausche and Goble, 1985; Goble and Rausche, 1986; Likins and Rausche, 1988).

Nevertheless, all three approaches described above have varying success (or limitations) in predicting the capacity of a pile (e.g., Cheeks, 1978; Lawton et al, 1986). Despite the many methods and numerous studies on the prediction (or calculation) of pile capacities, for the most part, there is still a **large scatter** between the estimated (analytical) and the measured capacities from loading tests (e.g., Dennis and Olson, 1985; Briaud and Tucker, 1988; Finno et al, 1989b). The large scatter might in fact be justification for the use of a reliability-based method of design; analysis of a large data set of relatively imprecise measurements may yield more and better information than a few very precise measurements in a highly variable material such as soil. As envisioned by Schmertmann (1990), reliability-based methods might be the trend in the future. However, the statistical procedures in reliability-based approach demanded an additional effort over the fundamental design principles of pile foundation (e.g., Sidi, 1986; Sidi and Tang, 1987; Madhav and Arumugam, 1979; Kay, 1976; Kay, 1977). As such, the design of pile foundation appears "too statistical" for a practicing geotechnical engineer. Thus, this study is an attempt to interrelate some basic concepts from these two approaches, within a systematic and rational framework.

In this study, the focus is on developing an algorithm to interrelate the safety measures from the deterministic and reliability-based approaches.

1.3 Objectives

The purpose of this study is to quantify the interrelation of safety measures as defined by the conventional deterministic and the reliability-based design approaches for pile foundation design; i.e., the interrelation of factor of safety (**FS**) and the reliability index (β). Consequently, an algorithm is developed using known design principles from both approaches.

For the development of the algorithm, a unique and consistent procedure for the determination of the predicted axial pile capacity (Q_p) is needed. This is achieved by following these specific procedures:

1. A repeatable, well-defined chain of steps is developed to calculate the axial capacity of a pile

using data from the Standard Penetration Test (SPT) N-values, derived from a typical "standard" analytical formula.

2. The calculated capacity is calibrated against "accurate" loading tests to predict the axial capacity; this systematic procedure also greatly reduces the element of arbitrary judgment with respect to the selection of the appropriate soil parameters to be used in the analytical formula.
3. The First Order Second Moment (FOSM) method is applied from reliability theory for the development of the algorithm; to interrelate the conventional factor of safety (**FS**) with reliability measures, the central factor of safety (**CFS**), the reliability index (β), and the probability of failure (P_f).
4. The developed algorithm (equations and charts) is illustrated with typical examples.

Having obtained the predicted capacity (Q_p), the safety measure used for the determination of allowable capacity (Q_a) in the deterministic approach is the factor of safety (**FS**), and the corresponding parameter in reliability-based approach is the reliability index (β). From the developed algorithm, the allowable capacity can be found and interpreted from either the deterministic or reliability approaches. The recommended safety measures (**FS** and β) in this study are the ones that can also satisfy the minimum deterministic factor of safety (i.e., **FS**=2.00) frequently suggested in practice.

1.4 Scope and Limitation

In this research, the predicted capacity (and consequently allowable capacity) in cohesionless soil only is considered. This is done by using data from the Standard Penetration Test (SPT); and the measured capacity (Q_m) is determined from the load-movement curves from instrumented pile loading tests.

The correlation of SPT N-values with the internal friction angle of soil (ϕ), may be considered to provide a gross approximation as compared to other in-situ soil exploration methods. Nevertheless, the Standard Penetration Test is the most commonly used method of in-situ soil testing, and data are relatively easy to obtain. Since the algorithm is developed from the N-values, it is felt justified to exclude any refinement to the "deterministic" parameters in the algorithm. Therefore, related issues such as the effect of residual stresses, sensitivity, degree of consolidation (normally or overly consolidation state), etc., are not explicitly included. Rather, a random variable (F_b) is introduced to calibrate the predicted capacity. Nevertheless, the specific parameters obtained from the algorithm could easily be modified to address those issues by using data from other more rigorous testing methods and refinements [e.g., data from cone penetration test (CPT), or dilatometer test (DMT)].

1.5 Definition of Capacity

The terms "ultimate capacity" or "ultimate resistance" used in many papers can be confusing, sometimes redundant and useless although they can be understood. In this research, the specific pile terms as suggested by Fellenius (1990) are followed.

The terms such as "load capacity," "design capacity," "carrying capacity," or "failure capacity" are avoided. "**Capacity**" (Q) is used as a **stand alone** term and as a synonym to "ultimate capacity" or "ultimate load" as found in most literature; however, "resistance" is sometimes used in place of the "capacity."

The "measured capacity" (Q_m) is reserved for capacity evaluated from the results of a static loading test of piles as defined by specific criteria (e.g., Davisson's, or Chin's). "Measured shaft" and "measured toe" capacity is then represented by Q_{sm} and Q_{tm}, respectively.

"Calculated capacity" (Q_c) is defined as the capacity calculated from static formula. "Calculated shaft" and "calculated toe" are represented by Q_{sc} and Q_{tc} respectively.

"Predicted capacity" (Q_p) is the capacity after the calibration of the "calculated capacity" (Q_c) from the static formula.

The "allowable capacity" (Q_a) is the value "allowed for design"; i.e., after the "predicted capacity" (Q_p) has been proportioned by the safety measures (**FS** or β)

Other related pile terms are as shown in Fig. 1.1.

1.6 Organization

Chapter 2 describes current methodology for the "standard" procedure for the calculation (or estimation) of axial pile capacity; it provides some background of methods from deterministic approach. The important considerations and critical elements for analysis and design of pile foundations are described.

Chapter 3 describes several methods of interpreting the measured capacity from full scale pile loading tests for calibration with the calculated capacity. The presentation of the methods of interpreting the measured capacity is limited to the ones that are used in this study.

Chapter 4 summarizes historical and theoretical background of engineering reliability theories that are applicable for pile design from reliability-based approach.

Chapters 2 and 4 provide the theoretical basis for the development of the algorithm derived in Chapter 5. Discussion of the data base, calculation using a spreadsheet program, a summary of the recommended parameters, design procedure, design charts, and the limitations associated with the developed algorithm are also included in Chapter 5.

Application of the developed algorithm is illustrated in Chapter 6, and observations are discussed.

Finally, Chapter 7 provides a summary and the conclusions of this research. Potential areas of additional research are also suggested.

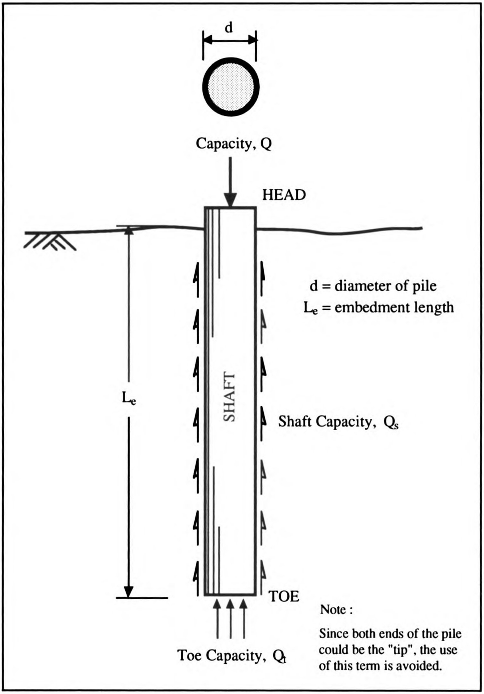


Fig. 1.1: Pile Terms and Definitions.

CHAPTER 2

DETERMINISTIC ANALYSIS AND DESIGN

2.1 Introduction

This chapter presents the basic theory and the general development of static formulas frequently used in design practice for the analysis of axial pile capacity. Attention is focused on the aspects of the theory and limitations that form a general approach for the development of the algorithm in Chapter 5.

Estimating accurate and reliable axial pile capacities has remained a difficult and complex task. Over the years, there has long been a need for a rational, consistent, accurate and yet simple method to predict pile capacity. The term "standard method" referred herein is the commonly accepted procedure rather than a prescription from any standard or code. The "standard" formula presented in Section 2.4, will form the general basis for comparison with loading test (Chapter 3); and the interrelation of safety measures derived in Chapter 5.

The difficulty in the prediction of axial pile capacity is well demonstrated by several studies (Section 2.2) which indicate the scatter of results between designers (and the theoretical methods used). These studies reflect the state-of-practice at this time.

Section 2.6 will briefly describe the principles of in-situ soil exploration by the Standard Penetration Test (SPT).

From previous studies (Section 2.5.1) and due to other various difficulties in predicting axial capacity, in practice, limits are usually imposed on the calculated capacity. In some cases, these limits are then used in codes.

2.2 Capacity Prediction

The soil-pile system is highly indeterminate structure, therefore predicting pile capacity is a complex task. At present, pile capacity is determined by: (i) analytical formula based on statics, (ii) numerical models based on dynamic formulas, or wave equation analysis, and (iii) measurements and interpretations from loading tests.

It is well recognized that capacities calculated using static formulas are approximate. No rational procedure is available to assist the designer in choosing the various formulas and types of testing techniques. As such, local experiences and engineering judgment remain a major component of the process for determining pile capacity.

As a general practice, because various degrees of uncertainty are involved in both the analytical formulas and testing techniques, the analytical predictions are "confirmed" with full scale pile loading tests; especially so for major projects. However, interpretation of the loading test itself involves some degree of uncertainty (see Chapter 3).

A series of pile loading tests were conducted in conjunction with the 1989 Foundation Engineering Congress (Finno, 1989d). Information about the soil, foundation and comparison of the predicted capacity of that study with the developed algorithm in this research will be presented in Chapter 6. Using the same data, 24 geotechnical engineers submitted *a priori* capacity predictions (as plotted in Fig. 6.4 for the pipe and H-pile only). The results of the *a priori* prediction and loading tests were then compared in Finno et al (1989c).

As shown in Table 2.1, there was a wide variation of predicted capacities and methods used, depending on participant's experience and judgment. In that study, predictions of axial pile capacities were made with conventional analytical methods, correlation with in-situ test results, load transfer functions and in one case a finite element simulation.

Table 2.1: Analytical Equations Used By 24 Predictors (After Finno et al, 1989c).

PREDICTOR NUMBER	SHAFT RESISTANCE (in Sand)	TOE RESISTANCE (in Clay)	REFERENCES USED BY PREDICTORS
1	$F_w K_S \tan(\delta) p'_{vo}$	$A_S (A_c S_u + p'_{vb})$	Poulos & Davis, 1980
2	$\alpha_S q_c R_t / N_o$	$\alpha_b q_c$	Bustamente et al, 1986
			Kulhawy, 1983
3	$(K/K_o) K_{oz} \tan[(\delta/\phi) \phi_z] p'_{vo}$	$N_c S_u$	Tomlinson, 1957
			Kulhawy, 1983
4	$K' \tan(\delta) p'_{vo}$	S_u	Meyerhof, 1956
	$2 f_S$	q_c	
5	f - z Curve		Mosher, 1984
6	N / 50		Meyerhof, 1976
	$K \tan(\delta) p'_{vo}$	$N_c S_u$	Kulhawy, 1983
	CPT Correlations		
7	$0.5 f_S$	0	Nottingham, 1975
	FHWA	0	Reese & O'Neill, 1988
8	N / 50	$N_c S_u$	Meyerhof, 1976
9	$K \tan(\delta)\ 1.3\ p'_{vo}$	$N_c S_u$	(not submitted)
10	CPT Correlations	CPT	De Beer, 1971/1972
	SPT	$N_c S_u$	
11	$K \tan(\delta) p'_{vo}$	$N_c S_u$	API, RP2A, 1987
	$\beta\ p'_{vo}$ or limiting f_S	$N_c S_u$	Reese & O'Neill, 1988
12	limiting f_S	$N_c S_u$	Nottingham, 1975
	$K \tan(\delta) p'_{vo}$	$N_c S_u$	Reese & Wright, 1977
13	$K (\sin(w + \delta) / \cos(w))\ p'_{vo}$	$N_c S_u$	Nordlund, 1980
14	$(K/K_o) K_{oz} T \tan(\delta/\phi) \phi_z p'_{vo}$	0	Poulos & Davis, 1980
		0	Stas & Kulhawy, 1983
15	f_S with SPT	$N_c S_u$	Coyle & Castello, 1981
	$\beta\ p'_{vo}$	$N_c S_u$	Reese & O'Neill, 1988
16	LCPC $(q_c / \alpha) < q_l$		Azzouz & Baligh, 1984
17	$K \tan(\delta)\ p'_{vo}$ (pipe pile)	$N_c S_u$	Coyle & Castello, 1981
	f_S (H-Pile)	$N_c S_u$	
18	$K \tan(\phi_i)\ p'_{vo}$	$10 S_u$	NA
19	$K \tan(\delta) p'_{vo}$	$N_c S_u$	NA
20	SPT N / 50		Meyerhof, 1976
	ϕ Correlation	0	Poulos & Davis, 1980
	CPT f_S		Nottingham, 1975
21	$\beta\ p'_{vo}$	$N_t p'_{vo}$	Canadian Fdn Engr Man, 1985
22	CPT f_S	$N_c S_u$	Dennis & Olson, 1983
23	$K \tan(\delta) p'_{vo}$	$N_c S_u$	NA
24	$K \tan(\delta) p'_{vo}$	$N_c S_u$	NA

As summarized by Finno et al (1989c) who analyzed the predictions, "standard" analytical methods of calculating pile capacity were used by 19 out of 24 participants (the number of individuals making predictions is actually more than 24 since "one participant" in some cases is a group of more than one individual). These approaches represent typical practice, and is an indication of the state-of-the-practice. For the most part, *effective stress* methods were used to evaluate shaft resistance in *sand* and *total stress* methods were used to evaluate both shaft and toe resistance in the *clay*. The parameters used by the participants in the analytical methods are summarized in Table 2.2. From Table 2.2, evidently there is a wide range of interface friction angles (ϕ) and lateral stress ratios (**K**) selected from the data provided. The variation of **K** chosen in these studies is mainly attributable to different judgments regarding effect of the construction procedures that could cause different lateral stresses against the pile after installation (uncertainty regarding the stress state). Also, values of the lateral earth pressure at rest (K_o) supplied in the data package were based either on Menard Pressuremeter or Dilatometer results. However, the fact that many participants used empirical relations to evaluate K_o indicates either lack of confidence in those results or a lack of experience with those types of data (Finno et al, 1989c).

Table 2.2: Parameters for Analytical Methods Used By 24 Predictors (After Finno et al, 1989c).

PREDICTOR NUMBER	PILE TYPE	SAND		CLAY	
		K	δ (degrees)	Ns (b. c. factor)	S_u (psf)
1	H-Pile	0.8	30	0.46	900 - 1000
	Others	0.4	30	0.23	
3	H-Pile	K_o PMT	(2/3) φ SPT		670
	Pipe	K_o PMT	(2/3) φ SPT		
	Slurry	0.67 K_o PMT	(0.8) φ SPT		
	Cased	0.67 K_o PMT	(2/3) φ SPT		
4	Driven	1.5 - 2.3	40 - 44.5	1.26 - 2.26	800
	Drilled	1	40 - 46	0.84 - 1.04	
9	H-Pile	0.5	35	0.35	501
	Pipe	0.57		0.4	
	Cased	0.45		0.32	
	Slurry	0.5		0.35	
11	H-Pile	0.8	15 - 35	0.21 - 0.56	600
	Pipe	0.9		0.24 - 0.63	
12	Driven	1	24.7	0.46	750
	Drilled	1	30	0.58	
13	H-Pile	K_o PMT	26		590
	Pipe	K_o PMT	24		
	Drilled	0.4	37	0.3	
14	H-Pile	K_o	(0.6) φ'		780
	Pipe	1.25 K	(0.7) φ'	0.99 - 1.7	
	Drilled			0.34	
17	Pipe	0.7	30.4 - 31.2	0.41 - 0.42	550
18	H-Pile	0.9	28	0.48	0.33 p'_{vo}
	Pipe	K_o	28		
	Drilled	K_o	40		
19	All	0.7	47	0.75	740
21	Driven			0.25 above w.t.	[0.35]
				0.45 above w.t	
23	H-Pile	1.42 - 1.74	27.2 - 30.8	0.73 - 1.04	567
	Pipe	2.13 - 2.61	23.8 - 26.9	0.93 - 1.32	
	Slurry	0.95 - 1.16	30.6 - 34.6	0.56 - 0.8	
	Cased	1.06 - 1.3	30.6 - 34.6	0.63 - 0.9	
24	Driven	1	32	0.62	470 - 620
	Drilled	1	40	0.84	

Approaches to estimate capacity varied from simply using averages of all types of test results to using normalized soil properties based on correlation with published data.

Fifteen participants chose to provide an estimate of capacity for which there would be a 90% probability that the measured values would exceed the predicted value. Nine participants used *judgment* when making their lower bound predictions by incorporating reduced strength parameters or lateral stress coefficients, by using different methods for their estimates, or by reducing their best estimate by an arbitrary amount. Three participants used a First Order Second Moment (FOSM) analyses with various normally distributed random variables. Six participants used probability approaches to predict lower bound capacities. For example, Wolff (1989) used the Point Estimate Method (PEM) and considered shear strength as a random variable and a function of depth. Others used the FOSM method and assumed other parameters normally distributed. As indicated by Finno et al (1989c), the fact that only six out of 24 participants chose a probabilistic approach may indicate the degree, or lack thereof, to which uncertainties are incorporated into design of piles for axial load in practice.

In the Northwestern study, measured capacities for the two driven piles were approximately equal to the most frequently predicted capacity; however, a relatively large difference in the predicted capacities for the drilled piers can be attributed to the effects of installation procedures.

In any case, as pointed out by Finno et al (1989c), if one were to compute capacities of a pile using several different methods and took an average value of the values to represent the predicted capacity, the effects of installation procedures would apparently be masked out. When the measured capacity and the lower bound predictions made by the same predictor are compared, it is found that there is an average reduction of about 25% in the predicted capacities. The range of reductions of the lower bound predictions for all participants varies from 10 to 40%.

For the portion of pile in clay, for practical purposes, the excess pore water pressures were assumed by most investigators to have been fully dissipated by the end of 52 weeks. Some participants assumed that the 2 week capacity was the long term capacity.

In sand, the gain in capacity with respect to time is generally neglected. For toe resistance in the clay, because of the relatively small contribution to the total capacity, it was also generally neglected.

However, other aspect of human factors might also influence the outcomes in that study. The participants might not have devoted enough time with their knowledge realizing that the prediction symposium is a staged event rather than an actual professional practice.

Another major study about predicting axial pile capacity was the one conducted by Briaud et al (1986) and Briaud and Tucker, (1988) at Texas A & M University. The purpose of that

study was to evaluate 13 specific analytical prediction methods using 98 tested piles provided by Mississippi Highway Department. The 13 specific methods as listed by Briaud and Tucker, (1988) are; (1) Coyle, (2) Briaud-Tucker, (3) Meyerhof, (4) API Code, (5) Mississippi Highway Department, (6) Direct Cone Method, (7) deRuiter and Beringen, (8) LPC Cone, (9) Schmertmann, (10) Tumay and Fakroo, (11) Penpile, (12) LPC Pressuremeter, and (13) ENR Formula. The outcome from that study also indicated varied results, especially for piles driven in mixed soil and with toe in sand, where "no best results" was found. Some methods seemed to be suitable only for certain soil conditions. For example, for piles driven in sand, Coyle's method was credited to performed the "best"; and in clay the Mississippi Method was "the best." However, Briaud et al (1986) concluded that combining a method which works well for piles all in sand with one that works for piles all in clay *does not* necessarily produce a method that works well for piles in layered soil. Among other findings by Briaud et al (1986) is that, "It was considered that perhaps an average of several methods would produce better results than the methods individually."

In short, the studies as briefly presented above reflect the complexity of predicting pile capacity. Due to large scatter of various methods used, variation in assumptions and individual judgment have resulted in a range of possible capacities being predicted.

Other previous studies related to statistical analysis and model error will be presented in Section 4.2.

2.3 Deterministic Analysis

This section presents some background information of several empirical methods using static formulas used to calculate capacity of pile foundations. Specific information about the "standard" Beta (β^*) Method used primarily to calculate pile capacity in sand will be presented in section 2.4.

A number of static, dynamic and wave equation methods have been proposed over the years for the prediction of axial capacity of pile foundations. In the effort to better estimate pile capacity, considerable research is ongoing with respect to modelling of soil-pile behavior, soil characterization, reconsolidation and various loading conditions.

When there is no toe bearing layer (e.g., in soft clay), the toe resistance is almost non-existent and is assumed to be negligible as compared to the resistance along the shaft. Therefore, the shaft resistance that can be provided by the soil is of primary importance.

Currently, empirical methods are used to estimate shaft resistance. This is achieved by means of correlations established in *similar* deposits between values of shaft resistance backfigured from loading tests and some assumed *relevant* in-situ characteristics of these deposits (Azzouz, 1990). Existing empirical methods for estimating shaft resistance can be broadly classified into three categories, namely;

(1) **Total Stress Method**: This group of methods is commonly referred to as the Alpha (α) Method; which uses the *total stress* approach. This method is primarily used for calculating capacity in clay soil.

(2) **Effective Stress Method**: Another group of approach is commonly known as the Beta (β^*) Method. This method was advocated by Zeevaert in 1959 which uses the *effective stress* approach (e.g., D'Appolonia, 1968; Coyle and Sulaiman, 1970; Meyerhof, 1976a). This method would be presented in some detail in the next section.

(3) **Mixed Method**: A quasi-effective stress approach is also used for the prediction of pile capacity. This third group is referred to as the Lambda (λ) Method. This method was proposed by Vijayvergia and Focht (1972) based on the general equation of unit shaft resistance presented by D'Appolonia et al (1968) and the Rankine equation for passive pressure. This method has been widely used for computing axial capacities of pile in offshore structures (Vijayvergia, 1977; Kraft et al, 1981; McClelland and Reifel, 1986; API, 1991). Vijayvergia (1977) used the remolded reconsolidated shear strength and the Coulomb equation for passive pressure.

In short, estimating the capacity of pile foundation using static formula may be approached by either Alpha(α), Beta (β^*), or Lambda (λ) Methods. The "standard" Beta (β^*) Method is typically adopted in sand.

2.4 Standard Method in Sand

The "standard" method for calculating pile capacity in sand is the Beta (β^*) Method. In this method, the shaft resistance is correlated to the initial effective overburden stress, p'_{ov}, through the empirical factor β^*. The use of effective stress method was advocated by Zeevaert (1959). In earlier days, this method was used for both sand and clay (e.g., Burland, 1973). However, present understanding indicated that application of the Beta (β^*) Method is limited to sand only [e.g., in Finno et al, (1989d), only one out of 24 predictors used this method in the clay layer].

Parry and Swain in 1977 extended Burland's (1973) concept to include a more realistic assumption for determining the effective overburden pressure (p'_{ov}) at failure. They assumed that the p'_{ov} at failure equals to the initial p'_{ov}.

The unit shaft resistance (q_s) from the effective stress approach may be expressed as;

$$q_s = f_s\,(p'_h) \qquad 2.1$$

where, f_s is the unit shaft resistance; and p'_h is the effective lateral earth pressure.

However, an accurate estimate of effective lateral earth pressure (p_h') is not easy. Since p_{ov}' is relatively easier to evaluate, for example, Randolph (1983), concentrated on finding the relationship between f_s and in-situ p_{ov}'. Equation 2.1 is then modified to include p_{ov}' as;

$$q_s = \frac{f_s\,(p_h')\,p_{ov}'}{p_{ov}'} \qquad 2.2$$

$$q_s = \beta^*\,p_{ov}' \qquad 2.3$$

Consequently, the β^* coefficient reflects not only the friction between pile and soil, but also the ratio of horizontal to vertical stress, K (i.e., $\beta^* = K\tan\delta$). The values of β^* may also be evaluated by simplified drained failure criteria (e.g., Burland, 1973).

There is substantial uncertainty associated with q_s and between various versions of the β^* Method. While there is still significant scatter between calculated and measured capacities, it does not appear as great as in using α-Method. Nevertheless, the formulation of β^* Methods is still based on several assumptions, e.g., drained [cohesion, $c = 0$; soil pile interface angle (δ) is taken at a reduced angle of internal friction of soil (ϕ)], and the final coefficient of earth pressure is assumed to be equal to the initial coefficient (which is not quite the case after pile installation).

In most cases, existing methods of pile design started by estimating the axial capacity of a single pile. In general, the calculated capacity of a single pile, **Q**, can be expressed as (e.g., Winterkorn and Fang, 1975; Meyerhof, 1976a; Fuller, 1978; Coyle and Castello, 1981; Kulhawy, 1984; Focht and Kraft, 1986; Tomlinson, 1986; McClelland and Reifel, 1986; Olson and Long, 1989; Finno et al, 1989d; API, 1991):

$$Q = Q_s + Q_t - w \qquad 2.4$$

where Q_s=shaft resistance; Q_t=toe resistance; and **w**=weight of the pile. The weight of the pile is comparatively small and is generally neglected in the computation of **Q**. The shaft and toe components are further discussed in Sections 2.4.1 and 2.4.2 respectively.

The complete format adopted for the calculation of axial pile capacity in this study is by combining the shaft and toe capacity (Eqs. 2.7 and 2.10), i.e.;

$$Q_c = \sum_{i=1}^{n} \left[\left(p'_{ov}\, K \tan \delta\right)_i (A_s)_i \right] + p'_t\, N_q^*\, A_t \qquad 2.5$$

where,

p'_{ov} = average effective overburden pressure along pile segment in soil layer **i**

p'_t = effective overburden pressure at the toe of the pile

K = coefficient of lateral earth pressure

δ	=	interface angle of friction between soil and pile material
N_q^*	=	modified bearing capacity factor
A_s	=	surface area of pile shaft in soil layers **i**
A_t	=	cross sectional area at the toe of the pile.

The calculated capacity (Q_c) is divided by a factor of safety, **FS**, to get the allowable capacity (Q_a) use for design. In deterministic analysis, the recommended **FS** commonly ranges from 2.0 to 4.0 (Peck et at, 1974; Meyerhof, 1976a; Fuller, 1978; Poulos and Davis, 1980; Tomlinson, 1986; Bowles, 1988), depending on designer's judgment.

The next two sections briefly discuss factors comprising the shaft and toe components of Eq. 2.5.

2.4.1 Shaft Resistance

The soil-pile interaction is very complex and poorly understood. The shaft resistance is commonly estimated based on the laws of mechanics considering friction between solid surfaces. The unit shaft resistance (q_s) can be represented as (e.q., Meyerhof, 1976a; Coyle and Castello, 1981; Olson and Long, 1989; Kulhawy, 1984; Focht and Kraft, 1986; API, 1991);

$$q_s = p'_{ov}\ K \tan\delta \qquad 2.6$$

therefore the total capacity can be obtained by integrating over the pile shaft area;

$$Q_s = \sum_{i=1}^{n} \left[\left(p'_{ov}\ K \tan\delta\right)_i \left(A_s\right)_i \right] \qquad 2.7$$

where the parameters are as defined by Eq. 2.5.

Effective overburden pressure is calculated knowing the location of the ground water table and the unit weight of sand, γ. To determine q_s, factors K and $\tan\delta$ need to be established; these are the most complex terms to evaluate (e.g., Kulhawy, 1984; Meyerhof, 1976a).

2.4.1.1 The value of K

The coefficient of lateral earth pressure, K (which is the ratio of horizontal to vertical effective earth pressure) is a function of the original in-situ horizontal stresses, and the stress changes caused in response to pile installation, loading, and time.

Meyerhof (1959) and Nordlund (1963), e.g., dealt theoretically with the problem of determining K values. In both studies, it was assumed that the pile displaces the sand in the horizontal direction, without inducing any vertical deformation. This displacement compacts the surrounding soil and the compaction is a maximum at the pile-soil interface. Since the shaft laterally compresses the sand and the horizontal movement is large (equal to pile radius), it is theoretically possible that **K** could be as high as the passive earth pressure coefficient (K_p).

Focht and Kraft (1986) took **K**=0.7 for compressive loads and **K**=0.5 for tensile load. Olson and Long (1989) suggested the value of **K**=1.0 for displacement piles (i.e., timber, concrete, closed ended steel pile), and **K**=0.8 for open ended pipe piles and H-piles. Other **K** values suggested by previous investigators are presented in Table 2.3. In Kulhawy (1984), the values have been evaluated using an extensive loading test data from Stas and Kulhawy in 1984, and the agreement is said to be good.

Table 2.3: Typical Coefficient of Lateral Earth Pressure, **K**.

(a) **K** Values (After Kezdi, 1975).

AUTHOR	BASIS OF RELATIONSHIP	SOIL TYPE	VALUES OF K
Brinch Hansen	theory	sand	$Cos^2 \phi$
Lundgren (1960)	pile test	sand	0.8
Henry	theory	sand	K_p
Ireland (1957)	pulling test	sand	1.75 to 3.00
Meyerhof (1951)	analysis of field data	loose sand	0.5
		dense sand	1.0
Mansur-Kaufman (1958)	analysis of field data	silt	0.3 (compression)
			0.6 (tension)
Lambe-Whitman (1969)	guess		2.0
Kezdi (1958)	theory	granular	K_p

Table 2.3: Continued.

(b) **K** Values as Suggested by Kulhawy (1984).

FOUNDATION TYPE AND METHOD OF INSTALLATION	RATIO OF HORIZONTAL SOIL STRESS COEFFICIENT TO IN-SITU VALUE (K/K_o)
Jetted Pile	1/2 to 2/3
Drilled Shaft, Cast-in-place	2/3 to 1.0
Driven Pile, Small Displacement	3/4 to 5/4
Driven Pile, Large Displacement	1.0 to 2.0

As can be seen of **K** values reported in the literature (Table 2.3), the bound approximately ranges from minimum active (K_a) to maximum passive (K_p) stress states. Evidently, it is not an easy task to assign the **K** value. In deterministic designs, given the range of possible values of **K** and without any other soil data available (say, only the SPT N-values), then perhaps the appropriate assumption is to design the pile for a long term capacity. If long term capacity is considered, the best estimate of **K** is the coefficient of lateral earth pressure at rest, K_o. Consequently, for this research K_o is simply adopted (see also the assumptions in Section 5.3). Kaizumi (1971) and Kulhawy (1984) pointed out that **K** may be maximum near the top and decrease to the minimum near the toe of the pile. By assuming **K** equal to K_o, the "modified β^*" factor (equal to

$K_o * \tan\delta$) derived in this study could satisfy this requirement.

Nevertheless, for the development of the algorithm in this study, the "exact" **K** value need not be quantified. From the method employed to develop the algorithm, the "choice" of selecting the different **K** values will show up in the form of correction and bias factors. However, to calculate pile capacity for the developement of the algorithm, a consistent procedure is needed. This is also true for δ values presented below.

2.4.1.2 The value of δ

For this research, the interface angle of friction between soil and pile material (δ) is taken as a reduced angle of internal friction of soil (ϕ). Potyondy (1961) under the supervision of Meyerhof, determined the coefficient of friction (or adhesion) between various materials for cohesionless soils at different densities. The study was made using direct shear tests in the laboratory. Consequently, it is possible to develop a relationship between ϕ and $\tan\delta$, as shown in Table 2.4.

Vesic (1977) proposed a different approach for the determination of $\tan\delta$. The sand located at the interface between the soil and the pile is considered to be at a state of failure for the determination of shaft resistance.

Consequently, δ is independent of the initial soil density and pile material. Therefore δ was considered to be equal to the residual friction angle (ϕ'_{res}). Focht and Kraft (1986) simply adopted $\delta = (\phi - 5°)$; and Kulhawy (1984) recommended value ranges from $\delta = (0.5\,\phi)$ to $\delta = (1.0\,\phi)$, depending on pile type. The selection within the range for any particular pile type is typically dependent upon personal judgment.

The δ values as suggested by Potyondi (1961), Vesic (1977), Focht and Kraft (1986), and Kulhawy (1984) suggestions do not seem to be significantly different. However, a rather rigorous approach as suggested by Potyondy (1961) is adopted for this study (Chapter 5); as opposed to merely an intuitive and subjective approach as suggested by Bowles (1988), Kulhawy (1984), and Focht and Kraft (1986). References to Potyondy's values were also made in earlier investigators (e.g., Thurman and D'Appolonia, 1965; Koerner, 1970; Coyle and Tucker, 1989).

Table 2.4: Typical Soil Pile Interface Angle , δ.

(a) δ Values as Suggested by Kulhawy (1984).

INTERFACE MATERIALS	f_ϕ	TYPICAL FIELD ANALOGY
Sand/Rough Concrete	1.0	Cast-in-place
Sand/Smooth Concrete	0.8 to 1.0	Precast
Sand/Rough Steel	0.7 to 0.9	Corrugated
Sand/Smooth Steel	0.5 to 0.7	Coated
Sand/Timber	0.8 to 0.9	Pressure-treated

Table 2.4: Continued

(b) δ Values as Suggested by Potyondy (1961).

CONSTRUCTION MATERIAL			SAND		COHESIONLESS SILT			COHESION GRANULAR	
	Surface of Construction Material		0.06<D<2.0 mm		0.002<D<0.06 mm			50% Sand + 50% Clay	
			f_ϕ		f_ϕ			f_ϕ	f_c
			Dry	Sat	Dry	Sat	Sat	Consistency Index	
			Dense		Dense	Loose	Dense	= 1.0 to 0.5	
STEEL	Smooth	Polished	0.54	0.64	0.79	0.40	0.68	0.40	-
	Rough	Rusted	0.76	0.80	0.95	0.48	0.75	0.65	0.35
WOOD	Parallel to grain		0.76	0.85	0.92	0.55	0.87	0.80	0.02
	At right angle to grain		0.88	0.89	0.98	0.63	0.95	0.90	0.40
CONCRETE	Smooth	Made of iron form	0.76	0.80	0.92	0.50	0.87	0.64	0.42
	Grained	Made of wood form	0.88	0.88	0.98	0.62	0.96	0.90	0.58
	Rough	Made of adj't ground	0.98	0.90	1.00	0.79	1.00	0.95	0.30

Note: f_ϕ is the Ratio of Soil-Pile Interface Angle (δ) to Angle of Friction of Soil (ϕ); $f_\phi = (\delta/\phi)$; $f_c = (c_a/c)$

2.4.2 Toe Resistance

According to Vesic (1968), the theoretical formulation of the toe resistance was initiated by Caquot and Buisman in the mid 1930's (from an extended work on punching failure done by Prandtl and Reissner approximately 15 years earlier).

Among the later contributors were Terzaghi, DeBeer, and Jaky in the 1940's; Meyerhof, Caquot, and Kerisel in the 1950's.

A somewhat different approach from punching failure was taken by Skempton, Yassin, and Gibson in the early 1950's when they dealt with the failure induced by the pile toe as a special case of the expansion of a cavity inside a solid. The same concept was used again by Vesic (1977). This time the approach involves the use of complex soil parameters (e.g., rigidity index).

In all of the theoretical solutions, the unit toe resistance, q_t, can be represented as (e.g., Coyle and Castello, 1981);

$$q_t = C\, N_c\, s_c + \gamma'\, B\, N_\gamma\, s_\gamma + p_t'\, N_q\, s_q \qquad 2.8$$

where γ'=effective unit weight of soil at the toe; B=smallest foundation dimension; p_t'=effective overburden pressure at the toe; C=cohesion; s_c, s_γ, s_q=shape factors; and N_c, N_γ, N_q=bearing capacity factors (usually dependent upon the soil friction angle and the assumed pattern or mechanism of failure). This is essentially the formula for the bearing capacity of shallow foundations, but with modified bearing capacity factors.

In practice, Eq. 2.8 is usually simplified. Since this study is for sand, the first term can be eliminated; the second term is relatively small and this can also be neglected. Since most piles have circular or square cross sections and the shape factor is the same (Kezdi, 1975; Vesic, 1967), it is reasonable to use a modified bearing capacity factor, N_q^*, that incorporates this constant shape factor. Therefore, the commonly used form (Coyle and Sulaiman, 1970) of Eq. 2.8 for unit toe capacity is reduced to:

$$q_t = p_t' N_q^* \qquad 2.9$$

And therefore the total toe resistance is equal to:

$$Q_t = (p_t' N_q^*) A_t \qquad 2.10$$

The range of modified bearing capacity factor (N_q^*) varies widely among the previous investigators. As presented by Vesic (1963), typical N_q^* values are such as the one proposed by Vesic, Meyerhof, Hansen, Coquot and Kerisel, Berezantsev, Terzaghi, Skempton et al, Janbu, and others; and the "best" N_q^* chosen are much depended on local practice or designer's preference.

In addition to the pile geometry, in most theories the basic parameters are the effective soil friction angle (ϕ'), which is used to find N_q^*, and the effective confining pressure. As mentioned by Coyle and Castello (1981), "*no theory" considers the shaft resistance of the soil along the pile shaft, or a possible interdependence between shaft and toe resistances.* Some of the theories assume soil to be compressible (Bishop et al, 1948; and Fruco and Associates, 1964) while others (Terzaghi, 1943; Meyerhof, 1951; and

Berezantzev et al, 1961) assume soil as a rigid-plastic system.

Under normal circumstances, there is no clear evidence that one bearing capacity factor is superior than the other to provide a more reliable prediction. Therefore in this study, the Vesic (1963) modified bearing capacity factor (N_q^*), is simply chosen (see also Section 5.3). After all, from the algorithm developed in this study, it is not of utmost importance which bearing capacity factor is adopted; the difference would show up in the form of other correction and bias factors in the developed algorithm as described in Chapter 5.

In short, the "standard" static formula for the calculation of axial capacity is presented; and the specific parameters have been identified. This static formula is the one most frequently used format by practitioners rather than a standard. The next section will present the limits imposed on the calculated capacity of shaft and toe resistances use by some authorities for the design of pile foundation.

2.5 Design Practice: Limits and Corrections

Studies by Whitaker and Cooke (1966), and Coyle and Reese (1966) indicated that the slip to develop maximum shaft resistance is about 5 to 10 mm and is relatively independent of pile diameter and embedment length, but may depend upon soil parameters. Mobilization of the ultimate toe resistance requires a toe displacement about 10 % of its diameter for driven piles and up to 30 % of the base diameter for bored piles and caissons.

As pointed out by Coyle and Castello (1981), Kerisel in 1961 observed that different size piles in the same sand attain a *maximum value* of unit load resistance (shaft and toe) which appears to remain constant with increasing penetration. The depth where the constant value is attained varies with pile diameter. The larger the pile diameter the deeper is the required penetration. Vesic (1970) confirmed Kerisel's finding with field tests; apparent unit resistances increased with depth to some *limiting value*. Vesic (1970) noted, however, that even though the rates of increase sharply decrease at some *critical* depth, there was an additional increase with further penetration. This critical depth was identified as being between 10 pile diameter for loose sands and 20 diameter for denser sands. In another study, Hanna and Tan (1973) performed laboratory model test using relatively long piles and observed that the critical depth approached 40 pile diameters. Meyerhof and Valsangkar (1977) obtained additional laboratory evidence of a limiting value for unit toe resistance. The study also showed that the critical depth for submerged sands is 1.6 times greater than that for dry sands. This increased critical depth is probably caused by buoyancy effects. Coyle and Castello (1981) also pointed out the study of Biarez and Gresillon in 1972 (France); again limiting values were obtained.

For practical purposes, limiting values are placed on shaft and toe resistances (e.g., API, 1991), in accord with the above empirical data. The limiting values recommended by API (1991) are as shown in Table 2.5.

However, as pointed out by McClelland and Reifel (1986) specifications by API are oriented towards predicting capacities of large offshore piles. The upper limits recommended for shaft and toe resistances are intended to ensure that capacities of "long" piles (say more than 50 ft.) are not overpredicted; and this method tends to underpredict the capacities of "short" piles.

All the above studies (Kerisel, 1961; Vesic, 1970; Meyerhof and Valsangkar, 1977) indicated some limiting values for shaft and toe resistances. However, other studies (e.g., Kaizumi, 1971; Kulhawy, 1984) argued that shaft and toe resistances *do not reach a limit* at a critical depth; they in fact *increase* with depth. For the shaft, the rate is a function of increasing overburden pressure and decreasing $\mathbf{K_o}$ with depth. For the toe, the rate of increase decreases with depth primarily because of decreasing rigidity with depth.

Therefore from previous studies, evidently the behavior of soil-pile system (or the failure mechanism) is not generally well understood. For the most part, Vesic's (1977) saying is still a fact in practice at this point in time. Vesic (1977) said, "...*computation of the ultimate load is quite difficult and a general solution is not yet available ... in view of the many uncertainties involved in analysis of* *pile foundations, it has become customary, and in many cases mandatory, to perform a certain number of full-scale pile load tests at the site of more important projects."* Loading tests may provide better estimates of pile capacity compared to computed values. However, it is also a fact that the loading tests themselves are subjected to variations (see Chapter 3).

Dennis and Olson (1983, 1985) recommended empirical correction factors in lieu of simply placing limits on shaft and toe capacity. These capacity correction factors are as represented by Eqs. 2.11 and 2.12 for shaft and toe respectively:

$$F_s = \frac{1.0}{0.6 \ \mathrm{Exp}^{(L_e/60\,d)}} \qquad 2.11$$

$$F_t = \frac{1.0}{0.15 + 0.008\,L_e} \qquad 2.12$$

where F_s=shaft capacity correction factor; F_t=toe capacity correction factor; L_e=distance from ground surface to pile toe in ft.; d=diameter of pile; (L_e/d) is dimensionless. These correction factors are multiplied to the shaft and toe components in the static formula (e.g., Eq. 2.5).

In that study, by using the proposed corrections factors, conformance of calculated to measured capacity is improved; they may also provide a correction for the fact

Table 2.5: Recommendation of Limiting Shaft and Toe Resistances by API (1991).

DENSITY	SOIL DESCRIPTION	δ	LIMITING SHAFT RESISTANCE, kips/sq.ft (kPa)	N_q^*	LIMITING UNIT TOE BEARING, kips/sq.ft (MPa)
Very Loose Loose Medium	Sand Sand-Silt Silt	15	1.0 (47.8)	8	40 (1.9)
Loose Medium Dense	Sand Sand-Silt Silt	20	1.4 (67.0)	12	60 (2.9)
Medium Dense	Sand Sand-Silt	25	1.7 (81.3)	20	100 (4.8)
Dense Very Dense	Sand Sand-Silt	30	2.0 (95.7)	40	200 (9.6)
Dense Very Dense	Gravel Sand	35	2.4 (114.8)	50	250 (12.0)

Note: δ is the Soil-Pile Friction Angle in Degrees; N_q^* is the Modified Bearing Capacity Factor.

that frictional resistance (consequently δ) decreases as the stress increases. However, the derived F_t is *not* dimensionless; which from an analytical standpoint might be undesirable.

For the algorithm developed in this research (Chapter 5), a similar approach for deriving the correction factors was used. It will be shown that the dimensionless factors ($\mathbf{F}_s$ and $\mathbf{F}_t$) derived in Chapter 5 are also found to give a good prediction of capacities as do those factors used by Dennis and Olson (1985).

2.6 Field Testing

Because of the specific requirements of a given structure and the spatial variability of soil properties both in breadth and depth, there is no single procedure for soil exploration (Lee et al, 1983). One of the most informative methods of soil exploration is a trench cut through the soil deposits, although in loose sands or below the water table such a method is inapplicable. Trenching is generally cost effective and appropriate for shallow soil deposits. Equipment is usually readily available from local sources. For deep foundations, the most commonly used soil exploration is perhaps the Standard Penetration Test (SPT).

This section presents a general review of the principles, application and limitations of the SPT. Other tests that have the potential to replace the SPT in the future are only briefly mentioned as they are out of the scope of this research.

2.6.1 Standard Penetration Test

The Standard Penetration Test is one of the earliest, most economical and most widely used tests for soil exploration. The test has been "standardized" by ASTM D 1586-67 (1976); and the basic elements of SPT test can be summarized as follows;

1. sampling with the standard spoon sampler (1 3/8 in. inside diameter, 2 in. outside diameter, 24 in. long),
2. driving the standard spoon sampler with a 140 lbs hammer; dropped 30 in.
3. driving the spoon 18 in. into the soil, and recording the number of blows at three 6 in. increments,
4. the N-value is defined as the number of blows to drive the last 12 in. of the standard spoon sampler.

As stated by Lee at al (1983), the SPT is "good" in terms of *quality* and *quantity* (quality refers to meaningfulness, i.e., the accuracy and precision with which the measurements can be made; and quantity refers to whether a large amount of data can be collected in a reasonably short time).

The SPT is a crude but useful test because of the enormous empirical data base obtained over the years. The SPT may be used in both sand and clay, but the correlation between N-values and the shear strength of clay is poor (e.g., Schmertmann, 1979; Lee at al, 1983; and Peck et al, 1974). Therefore, it should be treated only as a quantitative guide. On the other hand, a carefully conducted SPT in cemented sands should give a reliable guide to the in-situ density and therefore the ϕ-value.

As reported by Gibbs and Holtz (1957), normally consolidated sands under small overburden pressures tend to have N-values "too small" for their relative density, therefore, correction factors were suggested. Other studies by Bazaraa (1967) and Peck and Bazaraa (1969) also suggested that the N-values should be corrected for the overburden pressure. Over the years, there are many formulas and correlation suggested for the correction of the N-values. For this research, a simple format as proposed by Liao and Whitman (1986) has been adopted.

The SPT may require careful interpretation in soils other than sand. In particular, in gravelly sand as large pieces of gravel or rock may be trapped at the bottom of the spoon, thus affecting the N-value (Palmer and Stuart, 1957; Fruco and Associates, 1973). However, for the development of the algorithm in this research, only SPT data from sandy sites (and according to the selection criteria as described in Chapter 5) is considered.

The "standardized" SPT is subject to many sources of variability; and most of the time in practice, it deviates from the recommended procedure. This is especially true for older data sources. For example, Fletcher (1965) discussed the difficulties associated with the SPT. Earlier effort to standardize the SPT has resulted in little success (e.g., De Mello, 1971). Many studies done by Schmertmann (1975) on various types of penetration tests, for instance, found that "reproducing N-values is difficult." There are many factors attributed to the difficulty in reproducing the N-values, even in adjacent borings. Kovacs et al (1975) indicated that using 3 instead of 2 turns of rope around the cathead to hoist the hammer could increase N-values by as much as 40 %. Bowles (1988) states that McLean et al in 1975 pointed out that by using a longer drill rod may increase the N-values by as much as 14 blows/ft.

The sources of errors (mainly related to poor practice) and the difficulties associated with the SPT, can be listed as follows (Bowles, 1988);

1. variations in drop height; most often, this is done by eye,
2. interference with the free fall of the hammer; various guides and rope arrangement,
3. use of a drive shoe at the end of the standard spoon sampler,
4. failure to seat the sampler on undisturbed material; i.e., on the second 6 inches,
5. failure to maintain sufficient hydrostatic pressure in the boring; producing a "quick" zone at the borehole,
6. use of too light or too long a string of drill rods,

7. driving gravel ahead of the sampler,
8. use or non usage of liner trap; i.e., a retainer in the spoon to trap soil samples,
9. human error; careless drill crew.

In spite of the crude nature of the SPT and its high degree of uncertainties, the test is not easily phased out. Among the reasons for continued use of the SPT are that it is economical, cost effective and soil samples can be recovered for laboratory analysis. It is used routinely as the test is simple; therefore, enormous experience with the test, and a huge accumulation of data.

Using the corrected N-values according to Peck et al (1974), Meyerhof (1976a) presented a "direct" determination of the bearing capacity from the SPT N-values;

$$Q_p = \left[\overline{N}' * A_s \right] + \left[\left(\frac{0.4\,\tilde{N}'\,L_e}{d} \leq 4\,\tilde{N}' \right) * A_t \right] \qquad \text{(tons)} \qquad 2.13$$

where, $\overline{N}'$ is the *average* corrected N-values within embedment length of pile, L_e (ft.); $\tilde{N}'$ is the *average* corrected N-values within a distance of three to three quarter pile diameter, **d** (ft.) above the pile toe and a distance of one pile diameter below the pile toe; and A_s and A_t are the surface area of the shaft and cross sectional area of the toe (sq. ft.), respectively. The corrected N-values chosen to be used with Eq. 2.13 is highly subjected, and is much depended on designer's interpretation of the "average".

Briaud and his group in the early 1980's also directly correlated the SPT N-values with pile capacity. Using the *uncorrected* N-values, the predicted capacity (Q_p) is expressed as (Briaud et al, 1986);

$$Q_p = \left(0.448\, \overline{N}^{\,0.29} * A_s \right) + \left(39.5\, N_t^{\,0.36} * A_t \right) \quad \text{(kips)} \qquad 2.14$$

where, $\overline{N}$ is the weighted average of the N-values within embedment length of pile; N_t is the average N-values at four pile diameter above and four pile diameter below the toe; and A_s and A_t are the surface area of the shaft and cross sectional area of the toe (sq. ft.), respectively.

However, even by using the same data base that was used to derived the equation (Eq. 2.14), the prediction resulted in quite a large variation. The standard deviation of the predicted over measured capacity (Q_p/Q_m) was found to be 0.364. As reported by Briaud and Tucker (1984), the error was mainly attributed to two reasons; namely, (i) the error due to the natural variability of the soil (this is tied to the fact that the pile loading test and the soil test are not performed at the same location), (ii) the error in testing of the soil (this is for example the error on the N-values associated with the SPT). In another study, an advanced application of the SPT for the determination of soil parameters is currently being pursued, by incorporating the concept of wave equation; e.g., the research led by Goble at The University of Colorado (personal conversation, 1989).

In addition to the difficulties as discussed above, another issue that contributes to the variability and could affect the calculation, say pile capacity, is the *interpretation* of the N-values; and consequently, its correlation with the angle of internal friction of soil, ϕ. A common practice is to adopt the "average" N-value over the soil profile. Nevertheless, there are various ways to arrive at this "average" value. In short, the interpretation of the N-value remains largely a judgmental procedure.

In this study, the "actual" N-values from the boring logs are used; and a microcomputer spreadsheet is used to linearly interpolate and correct the N-values at every foot of the soil profile. There are many soil parameters that could approximately be correlated to the N-values; e.g., relative density, angle of internal friction, and unit weight of soil. In this research, the only correlation needed and used is the correlation with angle of internal friction, ϕ, as presented by Peck et al (1974). This relation is adopted as the friction angle of soil can be obtained as a function of N-values. A similar procedure has been used earlier by Wolff (1989); whereby a convenient calculation can be set up on a computer program. This procedure will also eliminate the element of arbitrary judgment in choosing the appropriate value of ϕ.

The use of N-values, correction and procedure adopted for this research are further discussed in Chapter 5.

2.6.2 Other Methods

To determine the capacity of pile foundations, other tests are sometimes employed [e.g., the cone penetration test (CPT), and dilatometers (or pressuremeters)]. The CPT is popular in Europe and is gaining ground in the United States. The CPT is equally good in sand and in clay, and provided a more reliable information as compared to the SPT, e.g., to quantify shaft resistance or settlement of pile foundation.

The Flat Dilatometer (DMT), e.g., has become one of geotechnical engineering's more versatile tools (Hryciw, 1990). It has an advantage of being able to measure the coefficient of lateral earth pressure (**K**) "directly" on the site (and consequently the bearing capacity), without any correlation from an "inferior" laboratory analysis. Using the DMT, other information about soil type, density, strength, compressibility, and the coefficient of consolidation can also be empirically derived. Analysis of data and current design recommendations based on DMT test procedure can be found, e.g., in Marchetti (1980), Baldi et al (1986), and Schmertmann and Crapps (1988).

It is well recognized that SPT is inferior to the tests briefly mentioned above. However, in the statistical sense, the analysis of a large data of relatively imprecise measurements may yield better information than a far more precise measurement in a highly variable material such as soil.

2.7 Summary

This chapter has described the "standard,"; i.e., the most frequently used static formula for calculating (or estimating) the axial pile capacity of pile foundation. The important considerations for a typical deterministic design of pile foundations were described. With the background information presented, a typical procedure to estimate parameter values and design pile foundations from Standard Penetration Test data is presented. Critical parameters and the "standard" formula described in this chapter will form the basis for the development of the algorithm in Chapter 5.

CHAPTER 3

INTERPRETATION OF PILE LOADING TEST

3.1 Introduction

This chapter describes techniques selected to interpret the measured axial pile capacity (Q_m) from a loading test. The emphasis is limited to the failure criteria that are used to derive the design charts in Chapter 5; specifically failure defined by, (i) 2 in. pile head movement, (ii) Davisson's criteria, and (iii) Chin's Criteria.

3.2 Description of Axial Pile Loading Test

The most reliable method (as compared to static formulas, dynamic formulas or the wave equation analysis) to determine the pile capacity is to perform a loading test. The test has been standardized as ASTM D 1143-81 (1987). However, local building codes may stipulate the load increments and time sequence. The pile can be loaded by using a hydraulic jack and jacking against reaction piles, or a weighted platform. Typical load-movement curves from pile loading tests are as found in Appendix A.

Piles in granular soil are often tested 24 to 48 hours after driving when load test equipment has been made. If the loading test is to assess long-term capacity, the test is generally done after transient effects due to installation or driving have dissipated. In non-cohesive soil, the waiting period could be, for example, a few days to weeks. For friction piles in soft to medium clay, a waiting period of three to six months may be required. For practical purposes, some contracts may permit loading tests seven days after driving. As indicated by Leonards and Lovell (1978), Sherman in 1969 adopted a minimum waiting period of two weeks; and Roth in 1972 stated that there were no additional effects after five to six weeks.

In any case, "sufficient time" should elapse before testing to allow partial dissipation of residual compression stresses in the lower shaft (and toe) from negative shaft resistance on the upper shaft caused by shaft expansion upward as the hammer energy is released. Residual stresses and/or forces have been observed from previous studies (e.g., Hunter and Davisson, 1969; Williams, 1960; Vesic, 1977).

3.3 Interpretation of Load Test

In the 1930's, "failure" load in a pile test was defined by Terzaghi (1942) as the load that corresponded to movement of 10% of the pile diameter. Since that time, many definitions have subsequently been proposed. At present, for example "failure" can be viewed as a rapid progressive movement at a constant load, or as a load satisfying a particular criteria, such as a movement of one inch; or twice the load at which the net movement is 0.25 or 0.50 in; or the load at which the movement per unit load is 0.01 in/ton, or when the incremental movement per unit load is 0.03 or 0.05 in/ton; or some similar combination of load and component of movement (Leonards and Lovell, 1978).

Each criteria has its own practical purpose. Thus, the interpretation of failure load varies greatly upon local practice, and local building codes.

It is common in practice that many piles are not tested to "failure." Testing may be stopped at say twice the "design load" according to the quick testing method (Leonards and Lovell, 1978; Fellenius, 1980). It may also be found that the pile continues to sustain increasing loads even at relatively large movements. In non-cohesive soil, this may be attributed to the continued dilation and/or the crushing of sand particles beneath the toe of the pile.

Fellenius (1975, 1980) compared nine different failure criteria using the same load-movement curve to interpret loading test. The measured capacity (Q_m) was found to be 181 and 235 tons for the Davisson's and the Chin's criteria respectively. Comparison of the nine different criteria is as shown in Fig. 3.1.

All the other seven criteria yield capacity values in between the Davisson and the Chin values. A similar indication was obtained from the study of Leonards and Lovell

(1978). This gives a notion that the Davisson's criteria is probably the lower limit and Chin's criteria can probably be taken an upper limit. The "best" Q_m is still uncertain to the profession at this time. Perhaps the "best single measured" capacity lies somewhere in between the Davisson's and Chin's criteria.

With present knowledge and the complexity of the soil-pile system, it is difficult to make a rational choice of the best criteria to use. It is very much dependent on one's experience. Fellenius (1975, 1978 and 1980) compiled several methods of interpreting loading tests (i.e., those outlined by Davisson, Butler and Hoy, De Beer, Fuller and Hoy, Hansen 90%, VanDer Veen, Mazurkiewicz, Hansen 80% and Chin). Of course, for the same loading test data, the interpreted capacity would not be the same as different methods are used. Based on those two earlier studies, Fellenius (1980) preferred using four criteria in his practice (namely; Davisson, Chin, Hansen 80% and Butler and Hoy).

With regards to the loading test program, from the experience of others (e.g., Fuller, 1978; Bourguard, 1987; Fellenius, 1980), besides other factors (such as pile depth, soil type or installation procedures), the leaks in hydraulic jacks (most commonly used for loading) may also introduce one of the largest variation in obtaining the load-movement curves. Consequently, the interpreted measured capacity would be affected.

A detailed comparison of the various methods or testing systems is beyond the scope of this study. However, to develop the design chart and the calibration of the predicted capacity in Chapter 5, there is a need to precisely define and consistently interpret the measured capacity. Consequently, the 2 in. movement, Davisson's, and Chin's criteria are considered. The fourth criteria is simply the *average of the measured capacity* as found by the Davisson's and Chin's criteria.

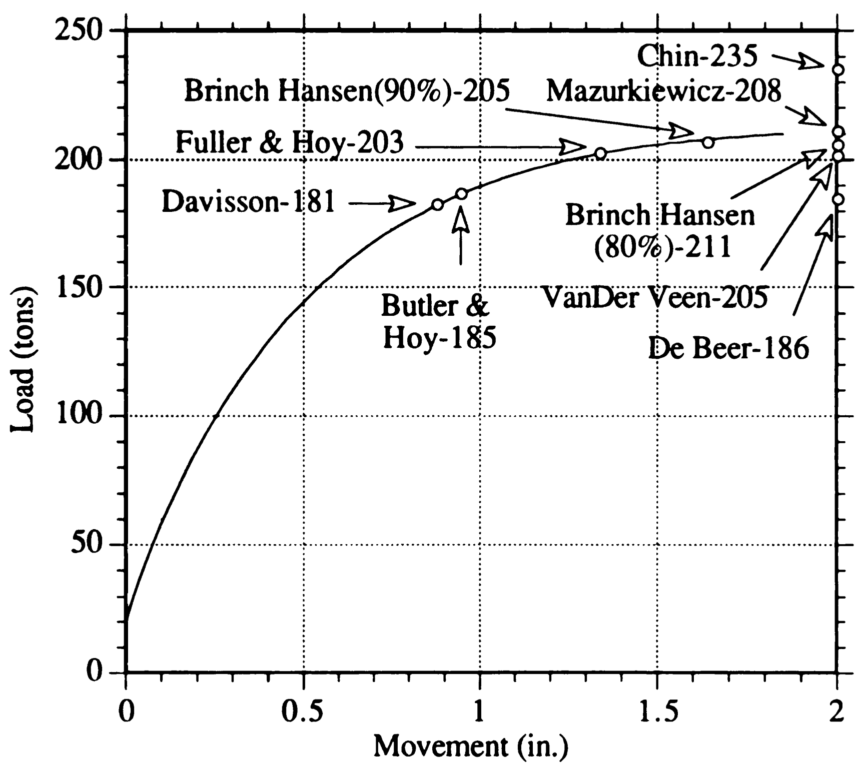


Fig. 3.1: Interpretation of Pile Loading Test by Nine Failure Criteria (After Fellenius, 1980).

3.3.1. 2 inch Movement Failure Criterion

The 2 inch movement failure criteria is simply defined as the axial load applied to the pile that causes the head of the pile to move down 2 in. Coyle and Ungaro (1991) and Ungaro (1988) indicated that 2 in. movement of the pile head is needed to mobilize the "maximum" or "limiting" shaft and toe capacities for H-pile. Consequently, for this research, the 2 in. movement criteria is arbitrarily considered.

3.3.2 Davisson's Failure Criteria

Davisson developed a method of interpreting pile load capacity by comparing the results of wave equation analysis with results of static loading test (Davisson, 1975). Consequently, the method has an advantage of being compatible with the wave equation analysis.

From loading test, a load-movement curve can typically be represented by either curve A, B or C in Fig. 3.2 for piles that primarily develop their capacity from shaft, shaft and toe, and toe resistances, respectively.

Davisson (1972, 1975) used an offset criteria similar to that used for some metals for determining the ultimate capacity. The criteria has been developed for the toe bearing pile by considering the deformation required to cause elastic compression (or quake) of the soil at the toe of the pile.

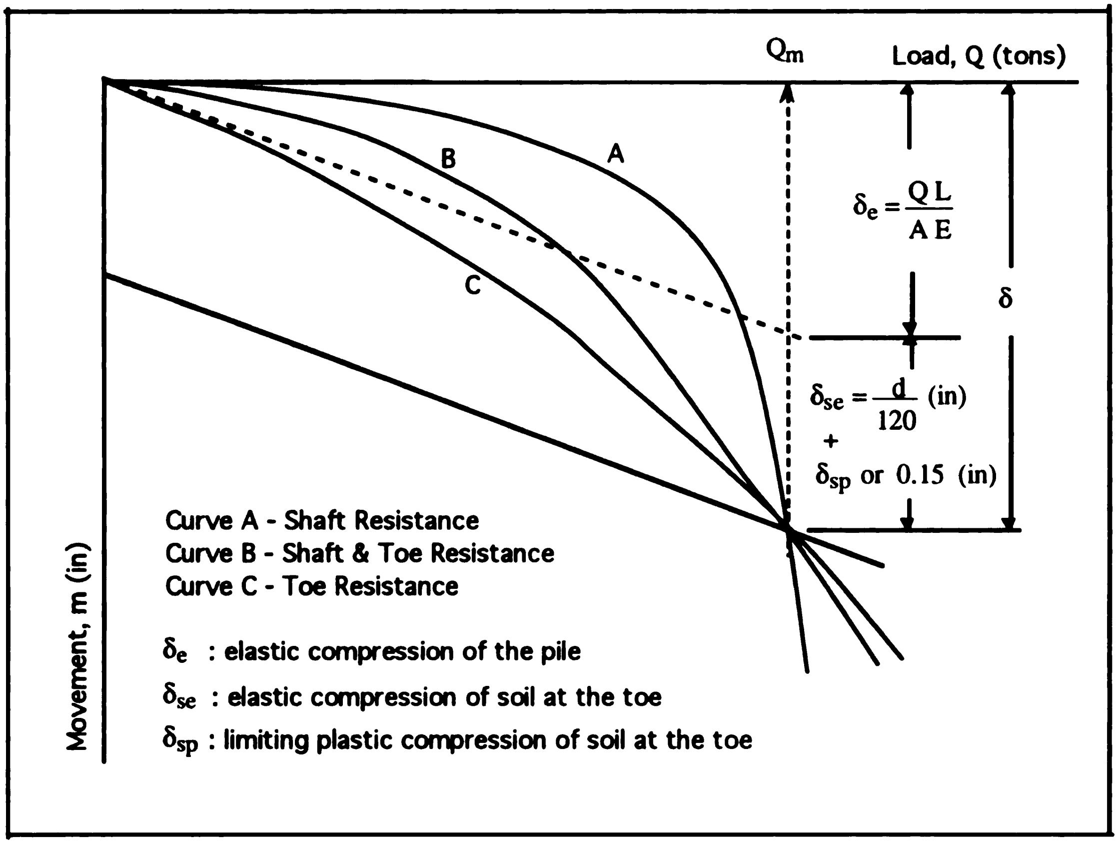


Fig. 3.2: Interpretation of Pile Loading Test, Davisson's Failure Criteria.

Davisson found that when movement of the toe is 0.15 in. (limiting plastic compression of the soil at the toe of the pile) plus quake (0.1 in. for a 12 in. diameter pile - could directly be measured by mounting a tell-tale device at the toe of the pile) below the elastic compression line (QL/AE), the ultimate load is presumed to have been reached. Therefore, from the load-movement plot of the head, the measured capacity, Q_m (or commonly known as "ultimate capacity") is interpreted when the head movement is equal to δ:

$$\delta = \delta_e + \delta_{se} + \delta_{sp} \qquad 3.1$$

$$\delta = \frac{QL}{AE} + \frac{d}{120} + 0.15 \quad \text{(in.)} \qquad 3.2$$

where,

δ_e = elastic compression of the pile

δ_{se} = elastic compression of soil at the toe of the pile

δ_{sp} = limiting plastic compression of soil at the toe of the pile.

3.3.3 Chin's Failure Criteria

From the pioneering work of Kodner in 1963, Chin (1970) proposed that the load(**Q**)-movement(**m**) curve could be expressed by two main assumptions;

(i) a load vs. movement curve is assumed to follow a rectangular hyperbola:

$$Q = \frac{m}{\Delta m + c} \qquad 3.3$$

where Δ and **c** are constants, defining the interaction between the pile and the soil,

(ii) on application of load, shaft resistance is mobilized progressively from the ground surface downwards, and **maximum** shaft resistance is achieved **before** any toe resistance is developed.

Equation 3.3 can be rewritten in the form:

$$Q = \frac{1}{\Delta + (c/m)} \qquad 3.4$$

which shows that Q approaches $(1/\Delta)$ as m approaches a high value.

This asymptotic behavior is a fundamental property of a rectangular hyperbola. Consequently, from the plot of (m/Q) vs. m, measured capacity can be determined, i.e., $Q_m = 1/\Delta$. To determine Δ, transposing terms in Eq. 3.4, resulted in:

$$\frac{m}{Q} = \Delta m + c \qquad 3.5$$

Equation 3.5 has the form $(y = \Delta x + c)$, which is an equation of a straight line, of slope Δ. Thus, Q_m is simply the reciprocal of the slope in the plot of (m/Q) vs. m.

When the result of a pile loading test is plotted in this manner (Eq. 3.5), two straight lines **ab** and **bd** may be obtained (Fig. 3.3). To explain this, Chin's second assumption applies, i.e., maximum shaft resistance is achieved before toe capacity is developed. The implication of this assumption is that two separate phases of capacity

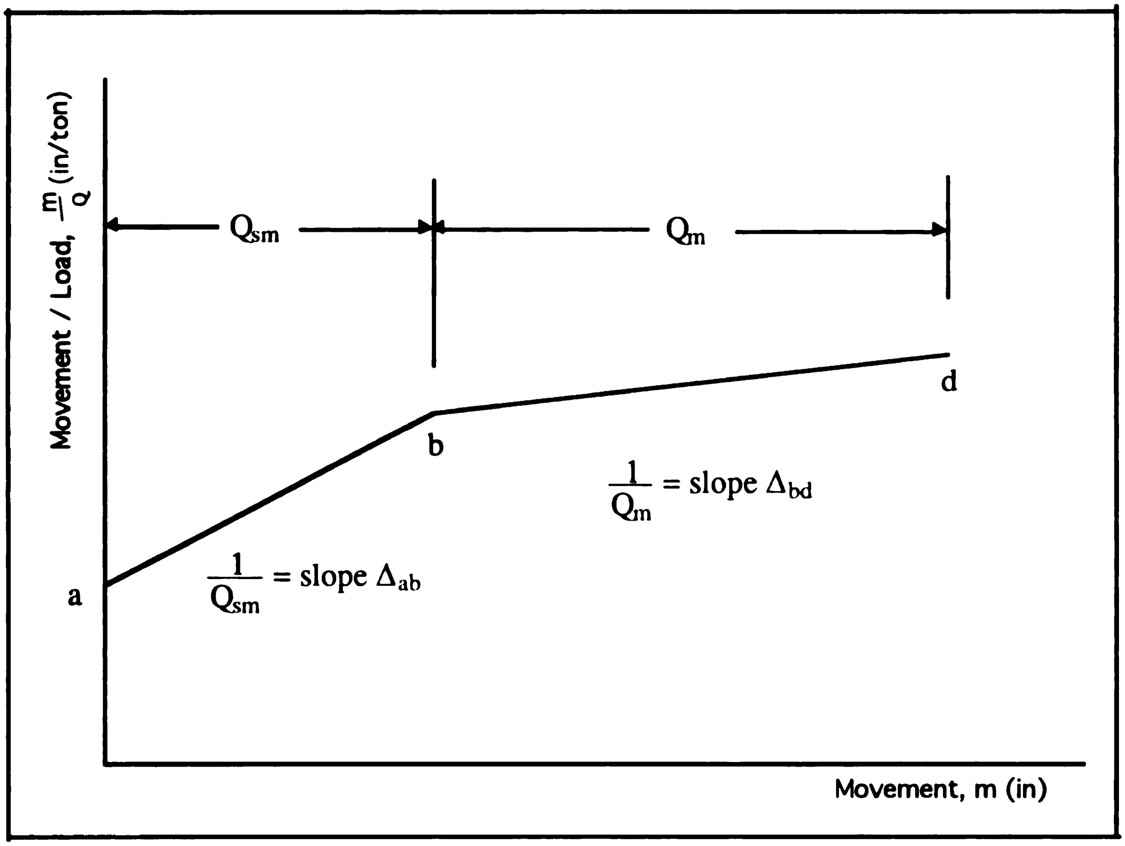


Fig. 3.3: Interpretation of Pile Loading Test, Chin's Failure Criteria.

generation governed by two different soil-pile interaction mechanisms occur; first, the build up of shaft capacity (Q_{sm}) developed only by elastic compression of the pile; second, the build up to ultimate capacity as toe capacity (Q_{tm}) is developed by penetration of the toe of the pile into the soil. These two interaction mechanisms possess different **Δ** and **c** values resulting in two different straight lines on the plot of **(m/Q)** vs. **m**. This leads to the conclusion that, line **ab** represents the build up of shaft capacity, Q_{sm}, with maximum shaft capacity being $Q_{sm} = 1/\Delta_{ab}$ and line **bd** represents the build up to ultimate capacity, i.e., $Q_m = 1/\Delta_{bd}$.

In routine pile loading test, a pile is typically loaded up to 1.5 or 2.0 times the allowable capacity invariably reaches a pile head movement in excess of that required to draw line **ab** with sufficient additional points to define the slope of line **bd**. Therefore, by using Chin's method, an estimate of Q_m can be made even though the test may not have been loaded to failure.

3.4 Limitations of Davisson's and Chin's Criteria

With an increased usage of the wave equation method of analysis, Davisson's method has gained widespread use (since Davisson's method was developed from the analysis using wave equation - see also Section 3.3.2). Its simple application for static analysis and straight-forward procedure made it popular and it can be used on routine loading test by relatively inexperience practitioners. It allows an engineer, when proof testing a pile for a certain allowable load, to determine in advance the maximum allowable movement for this load with consideration of the length and size of the pile.

The Chin's method is founded on an ingenious and convincing theoretical approach; and it could offer the advantage of:

(i) a possible separation of ultimate capacity into two components of shaft and toe capacities (However, from the data considered for this study, as indicated by the plots in Appendix A, only few plots are found to have two straight lines).

(ii) estimation of ultimate capacity without loading to failure.

(iii) identifying possible structural damaged of the pile.

From the plot of **(m/Q)** vs **m**, a damage pile can be identified by a non linear plot, and this is generally not highlighted from the conventional plot of **(Q** vs **m)**. This check can be done even when the test is in progress, a sudden kink or slope changes in the line could prompt the tester of problems of either the pile or the test instruments used. Therefore, the presence of damaged in piles or instrumental errors is consistently revealed by using Chin's method. However, in using Chin's method, the correct straight line does not start to materialize until the load has passed the Davisson's criteria. In many cases, a few points from the start of the test need to be neglected to avoid getting a false straight line (see also Appendix A). Another additional advantage is that the method is less sensitive to imprecisions of the load and movement values.

The Chin's method is applicable to both quick and slow tests, provided constant time increments (CRP) are used.

According to Fellenius (1980), the ASTM "standard" method of testing is therefore usually not applicable. The number of points in the standard testing are too few; an interesting development could well appear between load increment number 7th and 8th and be lost.

As with other methods, the accuracy of Chin's criteria is also dependent upon the validity of the assumptions made in formulating the hypothesis. Most piles do exhibit a hyperbolic (Q vs m) plot under loading test, but not all piles tested actually reach the limiting value. The maximum mobilization of the shaft capacity before any mobilization of toe capacity is also questionable. Evidence that toe capacity develops from the beginning of loading test or that shaft capacity continues to increase at some additional load can be found in many of the previous studies and reported tests of instrumented piles (e.q., Vesic, 1970; Leonards and Lovell, 1978; Fellenius, 1980). These and other limitations indicated that Chin's hypothesis cannot be completely correct.

Fellenius (1980) indicated a variation of about 30 % between the largest (Chin's criteria) and the lowest (Davisson's criteria) of interpreting failure load (in Fig. 3.1). From instrumented piles studied earlier by Vesic (1970), Leonards and Lovell (1978) pointed out that in some cases, an additional axial load applied on pile can be supported solely by shaft resistance without an increase in toe resistance. Therefore in general, the notion of early mobilization of shaft resistance or maximum shaft resistance

before toe resistance is mobilized could not be true. The reason as pointed out by Leonards and Lovell (1978) could be due to a redistribution of effective lateral earth pressure on the pile shaft. However, there are cases when shaft resistance is mobilized first, but this *cannot* be assumed a priori.

As a rule (Fellenius, 1980), interpretation of measured capacity using Chin's criteria is about 20 to 40% greater than the measured capacity using Davisson's criteria; when this is not the case, a closer look at all the test data was recommended.

3.5 Separating Shaft and Toe Capacity

In the 1950's, it was uncommon to use a tell-tale device when conducting routine pile loading test. A method of separating shaft and toe capacity from loading test was introduced by Van Weele (1957). This required knowledge of the magnitude and distribution of the unit shaft capacity, which Van Weele (1957) proposed be obtained by correlation from Dutch Cone penetration test. Analytically the method by Van Weele provide a consistent separation of the total capacity into shaft and toe capacity even though in some cases showed varying results. However, some investigators have lately begun to use it (Leonards and Lovell, 1979; Brierley et al, 1979; and Fellenius, 1980). Nevertheless, in

Van Weele's method, the knowledge of toe movement is needed; consequently this method was not considered in this study (load-toe movement curves are not available from the data considered).

However, the separation of shaft and toe capacity can be estimated according to the interpreted criteria as briefly described below.

The distribution of axial load in the pile can be measured if the pile is instrumented with strain gages at a few locations along the pile (e.g., Williams, 1960; Vesic, 1970; Bustamente, 1982b). With proper calibration, the load along the pile shaft can be plotted as represented by curve **abc** in Fig. 3.4. The load (**Q**) on the curve **abc** in Fig. 3.4 is not typically the measured capacity (Q_m) as interpreted by the chosen criteria. Therefore for this research, the proportion of the toe capacity, (Q_{tm}/Q), is linearly interpolated according to the interpreted Q_m as dictated by the function of (Q_{tm}/Q) for the respective criteria (see also Step 3 in Section 5.3.2). Consequently, the shaft capacity, Q_{sm}, is then the difference of the interpreted Q_m and Q_{tm}. This method provided a consistent method of separating Q_{sm} and Q_{tm} from loading test. (see Chapter 5).

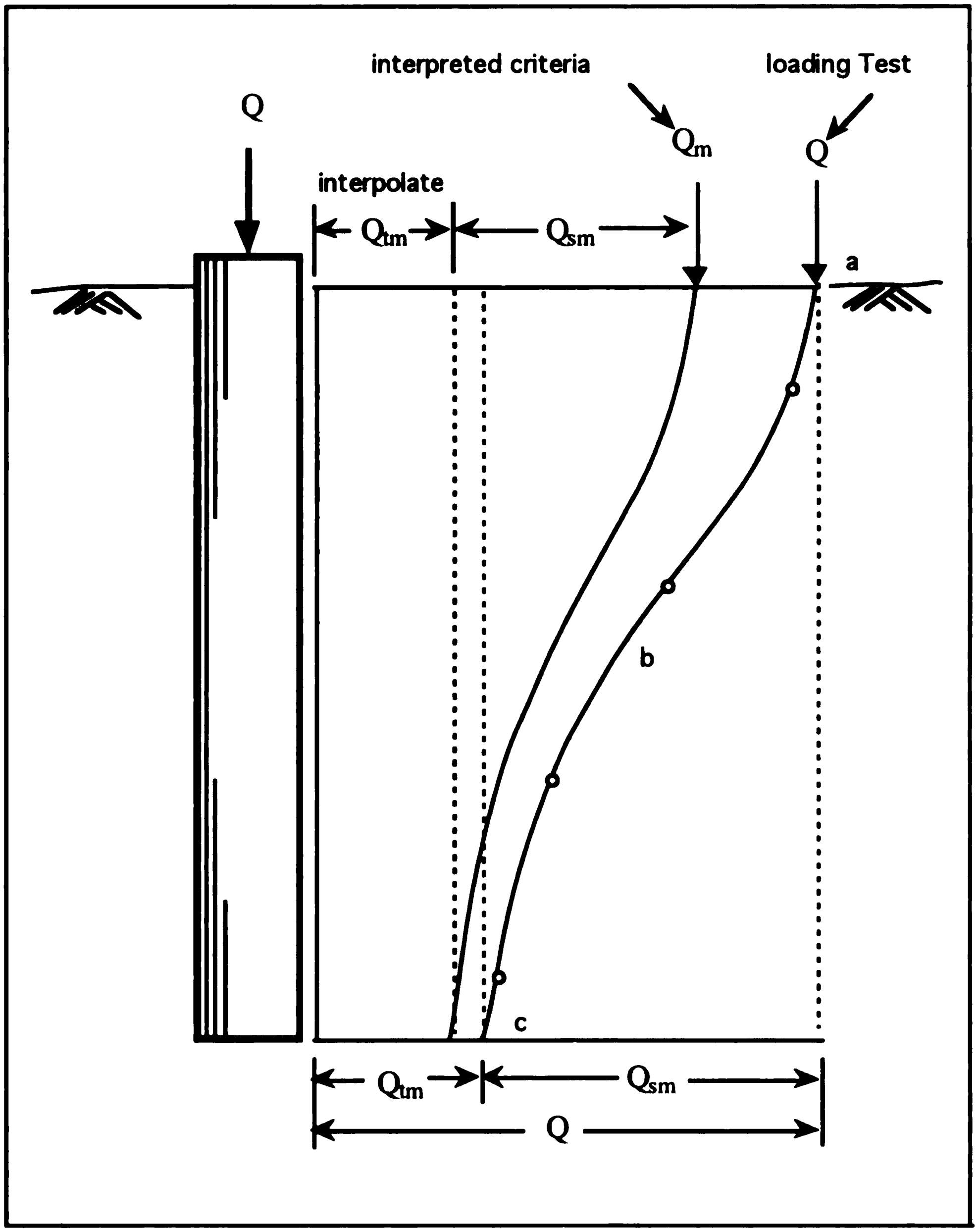


Fig. 3.4: Distribution of Resistances Along Pile Shaft.

3.6 Summary

Even though the capacity of pile as determined by loading test is considered to be the most accurate, the interpreted measured capacity is highly dependent upon how the failure criteria is defined. Evidently from previous studies, the measured capacity could vary in the order of about 20 to 40 % if different criteria are used.

The four criteria as presented in this Chapter were used extensively in Chapter 5 to derive the design charts in the developed algorithm.

CHAPTER 4

RELIABILITY THEORY IN ENGINEERING

4.1 Introduction

In the presence of uncertainties, absolute reliability is not possible. However, probability theory and reliability-based design techniques do provide a formal framework for developing criteria for design which insure that the probability of unfavorable performance is acceptably small.

There have been no standards adopted in the US which synthesize all the available information for purposes of developing reliability-based criteria for design (Ellingwood, 1980). Prior to implementation of Load Resistance Factor Design (LRFD), the use of statistical methodologies had stopped at the point where the nominal strength or load was specified. Additional load and resistance factors, or allowable stresses, were then selected subjectively to account for unforeseen unfavorable deviations from the nominal values. At present however, probability theory and structural reliability methods make it possible to select safety factors to be consistent with a desired level of performance, i.e., acceptably low probability of unsatisfactory performance.

Some background and findings from previous studies related to the prediction of pile capacity have been presented in Section 2.2.2. This section will present previous studies that are related to reliability analysis and draw from it concepts for the development of the algorithm in this research.

4.2 Previous Work

To estimate pile capacity, it is necessary to assess the individual pile uncertainties as well as the overall bias and error associated with design according to a specified recommended procedure. It is only through a complete uncertainty analysis that the reliability of a pile or pile system could be estimated. Among previous work in this area are the studies by Dennis and Olson (1983, 1985), and Olson (1984). Dennis and Olson developed a computerized data base of 1004 pile load tests to evaluate the bias and error associated with specific pile capacity prediction methods. From that study, it was shown that there is a large scatter between measured and predicted capacities especially in the earlier API recommended design practice. Their emphasis have been on the statistical assessment of the prediction model error and providing suggestions for improved design procedures. The correction factors as proposed by Dennis and Olson (1983, 1985) for the shaft and toe capacities are the one as presented in Section 2.5.1. However, some other factors, e.g., loading rate, reconsolidation and nominal strength were not explicitly accounted for.

Bea (1983) identified various factors affecting offshore pile capacity prediction, and suggested preliminary estimates of the bias and error associated with each factor. An extension of the work by Bea (1983) was done by Tang (1989) and Sidi (1986).

For drilled shafts, in another study by Kulhawy and his group (Kulhawy, 1984), it was shown that there is a significant bias and error between predicted and measured pile capacities.

Kay (1976, 1977) proposed a rational procedure for consistent design of single pile, correlated from the results of loading tests. To optimize the testing procedure and to improve final design of piles, the Bayesian probability theory was used. In those studies, the First Order Second Moment (FOSM) method as proposed by Cornell in 1969 was used. On comparing the results of static pile loading tests on driven piles and the prediction of pile capacity (using published data from Whitaker and Cooke in 1966), Kay (1976, 1977) assumed that the capacity values followed a *lognormal* distribution. However, it must be emphasized that in Kay (1976) the *number of the examined piles was only five.* The mean of the ratios of log of measured to predicted pile capacities ranged from 0.92 to 1.42 and the standard deviation of this ratio ranged from 0.14 to 0.35. The standard deviation of the ratio of the log in those studies has a *large impact* on the reliability index, and consequently the allowable capacity. Also, the measured capacities from the loading tests that were used in that study (also adopted from reported literature) were not likely interpreted by the same method.

In short, it can be said that Kay's findings were derived from a sound theoretical basis, but the accuracy of the end result is somewhat "of low precision." The research herein follows Kay's work with regards to the assumption of scatter of model uncertainty at the site; but the capacities are determined from a well defined and consistent methods.

4.3 Structural Reliability Theory

4.3.1 Historical Background

Reliability theory has been used by engineers for some times in the determination of the reliability of electronic and mechanical systems in the manufacturing process. For example, probability methods are commonly used in circuit problems or determining the life expectancy of a machine. Using reliability theory, structures can also be designed to function for an expected and reasonably predictable time.

Over the lifetime of most structures, they are subjected to various kinds of loads that are not always constant, predictable or predetermined. The material used for a support system, for example is dependent on many uncertainties in the manufacturing process, material strengths and modelling methods.

Until recently, structural design has been primarily a deterministic procedure using what were perceived the upper bounds of loads effects and lower bounds of material strengths. There was an understanding that there existed a continuum of probable values. As reported by Corotis (1985), structural reliability research had not gained much attention for advancement mainly for three reasons, namely; first, the continuum of probable values could not readily be defined due to lack of sufficient data bases; second, the mathematical methods had not yet been developed to define rational safety margins; and finally, slow change in structural codes. A comprehensive review on early attempts at introducing reliability methods and practices into structural design can be found in, e.g., Madsen et al (1986).

It was not until the middle of 1960's that reliability or probability based structural design began to gain some support. Although by this time quite a number of new concepts had been developed, much of them were in the class of academic research rather than being actually used by practitioners; after all, deterministic design is fairly straight forward and had served the profession well with very few failures (De Mello, 1971; Corotis, 1985).

A probability-based structural code based on a second moment approach was proposed by Cornell (1969). His work gained much attention and attracted others to research on reliability methods because his approach had the promising ability to produce a set of safety factors on loads and resistances, something that code writing committees specified. Cornell's work opened the door to the possibility and acceptance of probability-based structural standards; even though problems were encountered in attempting to implement this approach.

Some examples of usage of system reliability are such as Frudenthal, (1961, 1966); Kay, (1976, 1977); Ellingwood, (1978); Madhav and Arumugam, (1979); Ellingwood et al, (1980); Grigoriu, (1982/1983); Moses and Rashedi, (1983); Wolff and Harr, (1987); Sidi and Tang, (1987); and Wolff and Wang, (1992).

4.3.2 Formats of Reliability Theory

The general format of reliability theories was categorized by Corotis (1985) as Stochastic, Full Distribution, First Order Second Moment (FOSM), and Load Resistance Factored Design (LRFD). General definitions of these formats are briefly presented below.

4.3.2.1 Stochastic

A stochastic reliability format can be represented as:

$$P_s = P[\, D(s,t) < C(s,t) \,] \qquad 4.1$$

where, P_s = probability of safe performance; s = space throughout the structure; t = time over the economic lifetime of the structure; $D(s,t)$ = various components of *loading* (or Demand); $C(s,t)$ = various components of *resistance* (or Capacity).

Equation 4.1 indicates that the probability of safe performance is the probability that each component of the load effect vector is less than the corresponding component of the resistance vector, throughout the space of the structure and the design life. This approach is simple in concept but complex and untraceable in application due to the difficulty of handling of multi-dimensional stochastic variables. Thus, this format remains limited to research status only (but provide an important conceptual understanding).

4.3.2.2 Full Distribution

This format is a simplified version of the stochastic format. By assuming that structural resistances do not vary over time, the time component in Eq. 4.1 is restricted. Then some approximate approaches to load combination theory and stochastic process are used to replace the time-varying loads by a few critical load combinations involving *random variables* rather than *random process*.

A simple and relatively accurate form is such as the one developed by Turkstra et al (1980). It considers N random load processes through time on a structure. At some point in time during the design lifetime of the structure (actually N different times in general) each of these processes will experience its lifetime maximum load, $\mathbf{D_m}$. The assumption is made such that as each process experiences its maximum, the other processes assume average (so called arbitrary-point-in-time) values, $\mathbf{D_a}$. For N different load process, this means there are N different load combinations to check, each corresponding to a different process experiencing its maximum.

It has been shown that this method works well (Turkstra et al, 1980), although there are other load combination approaches available.

A typical equation for a Full Distribution Format might take the following form:

$$P_s = P\left\{ \max_{j=1,N}\left[D_{jm}(s) + \sum_{\substack{i=1,N \\ i\neq j}} D_{ia}(s) \right] \langle\ C(s) \right\} \qquad 4.2$$

Equation 4.2 is much simpler than Eq. 4.1 although it looks complicated since stochastic variables (in time) have been replaced by random variables. In theory, Eq. 4.2 represents a check for structural performance throughout the structure. In actual practice, however, the vector aspects of loads (or Demands) and resistances (or Capacity) are implemented by considering a few key load effects (e.g., bending moment and mid-span shear). Also, the spatial aspects, **S**, are considered by studying only the critical points in the structure. This reduces the problem to one of repeated scalar comparisons at single locations. One such check would take on the following form:

$$P_s = P\left[D \langle C\right] \qquad 4.3$$

in which

$$D = \max_{j=1,N}\left(D_{jm} + \sum_{\substack{i=1,N \\ i\neq j}} D_{ia}\right) \qquad 4.4$$

Since both **C** and **D** in Eq. 4.3 are random variables, the evaluation of that probability generally involves a convolution integral. Two forms of convolution integrals may be used to evaluate Eq. 4.3. One form is:

$$P_s = \int_0^{\infty} f_D(\ell)\left[1 - F_C(\ell)\right] d\ell \qquad 4.5$$

This equation represents the probability of safe performance as the likelihood the load assumes any particular value $f_D(\ell)$ multiplied by the probability that the resistances exceeds this value $[1 - F_C(\ell)]$, and then this product is integrated over all possible particular load values. Another convolution form is:

$$P_s = \int_0^{\infty} f_C(r)\,[F_D(r)]\,dr \qquad 4.6$$

This convolution equation can be said to represent the likelihood that the resistance assumes a particular value $f_C(r)$ multiplied by the probability that the load is less than the value $[F_D(r)]$, and then this product is integrated over all possible resistance values.

4.3.2.3 First Order Second Moment

This format was first proposed by Cornell (1969). The basic premise of this format is that it simplifies the full distribution format for practical applications. Within a deterministic sense, each "variable" assumes a single design value. Cornell suggested the expression of random variables using two parameters, rather than a single value, to introduce random aspects without causing the problem to become untraceable. The two quantities are the mean (or the expected value) and the other is a measure of uncertainty or variability (commonly accepted is the standard deviation or variance).

This approach of *using the mean and variance*, is therefore referred to as a *first order* uncertainty analysis (Cornell, 1971) or *second moment* approximation of the Taylor Series (Harr, 1987).

The real advantage of the FOSM approach is that it introduces consideration of variability without requiring full distribution analysis. As an example, consider a single load (or "Demand") effect quantity, **D**, with mean $\overline{D}$ and standard deviation σ_D, and a single corresponding resistance (or "Capacity," **C**) with mean $\overline{C}$ and standard deviation σ_C. To design a structure with a high likelihood of satisfactory performance, one might pick a design value for the Demand that is a few standard deviations above its mean and a resistance (i.e., Capacity) design value a few standard deviations below its mean. The design equation for this format would then be the design load that does not exceed the design resistance:

$$D_d = \overline{D} + k_D \sigma_D \leq C_d = \overline{C} - k_C \sigma_C \qquad 4.7$$

The reliability implied by Eq. 4.7 depends on k_D and k_C, and also on the probability distribution for **C** and **D**. Without knowledge of the actual distributions, only approximate probabilities or very wide Chebyshev bounds can be found. However, this approach is straight forward, and it does include direct consideration of variabilities.

If both Capacity and Demand are assumed to be independent and normally distributed, the safety margin, **SM**, is also normally distributed.

The reliability is the probability that **SM** is positive:

$$P_s = P[\,SM > 0] = 1 - F_{SM}(0) \qquad 4.8$$

Since **SM** is normally distributed, Eq. 4.8 may be expressed as:

$$P_s = 1 - F_U\left(\frac{0 - \overline{SM}}{\sigma_{SM}}\right) = F_U\left(\frac{\overline{SM}}{\sigma_{SM}}\right) = F_U(\beta) \qquad 4.9$$

where, $F_U(u)$ is the unit, e.g., normal cumulative distribution function (unit c.d.f). The reliability is uniquely determined by β, which may be thought of as the number of standard deviations by which the mean Capacity exceeds the mean Demand; i.e., the standard deviation of the **SM**. The quantity is often referred to as a reliability index, β.

The extension of Eq. 4.9 to multiple loads or resistance (non-linear functions) causes a problem known as lack of invariance. To circumvent this, using the small variance approximation, Hasofer and Lind (1974) developed an approximate procedure by using the concept of performance function (Ellingwood et al, 1980; Corotis, 1985) as shown in Fig. 4.1.

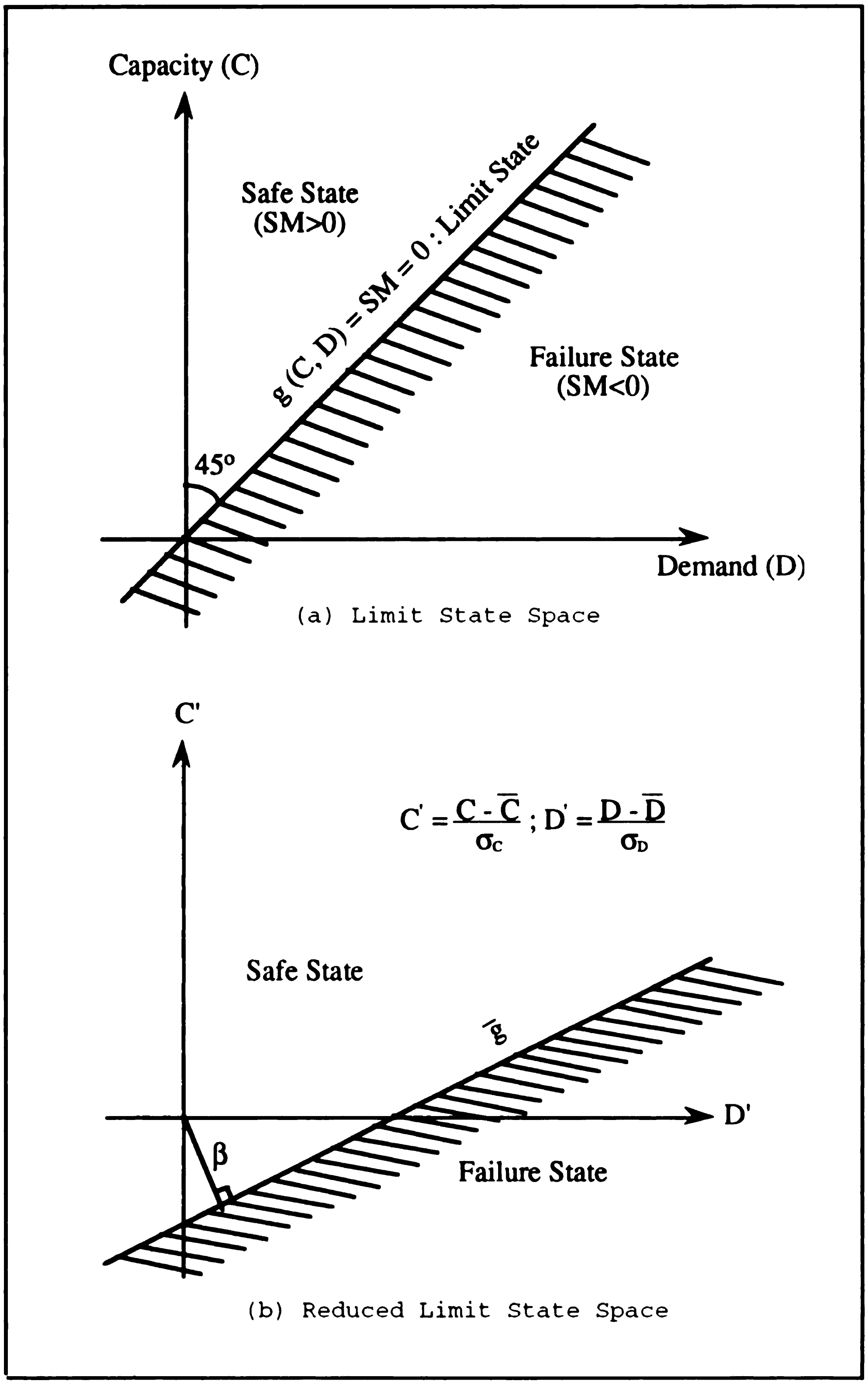


Fig. 4.1: The Model of Performance Function.

The general form of *performance function*, or *limit state* equation in terms of Capacity and Demand may be expressed as (Ellingwood et al, 1980; Ang and Tang, 1984; Harr, 1987);

$$g = g_C(C_1, C_2 \ldots) - g_D(D_1, D_2 \ldots) \qquad 4.10$$

The surface in capacity and demand space, i.e., at ($g=0$) is termed as the limit state surface. Failure is defined as any condition when ($g<0$) or when ($g_R<g_S$). By simple substitution, the limit state surface may be expressed in terms of reduced variables (with mean equal to zero and unit variance as shown in Fig. 4.1(b)). After a transformation to independent random variables if correlation exists, the limit state surface is then in terms of so called basic variables in reduced space.

The shortest distance from the origin to the limit state surface in this space is defined as the measure of reliability, i.e., β, which may be approximately expressed as;

$$\beta = \frac{\bar{g}}{\sigma_g} \qquad 4.11$$

where, $\bar{g}$ = mean value of g; σ_g = standard deviation of g.

It is assumed that capacity and demand are random independent variables, otherwise the shortest distance to the limit state does not represent the reliability.

In terms of **SM**, Eq. 4.10 can be rewritten as;

$$g(C, D) = SM = 0 \qquad 4.12$$

Safe state is represented by (**SM>0**) and failure state by (**SM<0**). It can be shown that (e.g., Ellingwood, 1980; Harr, 1987) for normal uncorrelated distributions, the shortest distance from the origin, thus β value can be expressed as:

$$\beta = \frac{\overline{C} - \overline{D}}{\sqrt{\sigma_C^2 + \sigma_D^2}} \qquad 4.13$$

where, $\overline{C}$ = mean value of resistance (or Capacity); and $\overline{D}$ = mean value of load (or Demand).

A detailed assessment about the subject can be found in Ang and Tang (1984), Ditlevsen (1981) and Harr (1987).

4.3.2.4 Load and Resistance Factor Design

A convenient simplification of the FOSM format is possible when structural analysis is based on a member analysis rather than a system analysis. This is the basic premise of FOSM format. The approximate degrees of safety or reliability are provided on an element or component basis. This is accomplished through the use of partial safety factors applied to nominal values of loads and resistances.

Examples of Load Resistance Factor Design (LRFD) can be found in ACI (1991), or PCA (1989) Code since 1963, AISC (1980) specification, and Canadian Steel Design (1978); where the nominal capacity of an element is multiplied by a specific strength reduction factor (value less than 1.0). The assumed demands (i.e., moments or forces) are multiplied by specific load factors (values generally more than 1.0). The nominal values employed are not necessarily the mean values for strength and are in most cases somewhat conservative. A typical equation can be expressed as;

$$f_r R = f_{DL} DL + f_{LL} LL \qquad 4.14$$

where, f_r, f_{DL} and f_{LL} are the factors for resistance, dead load and live load, respectively.

The left hand side of Eq. 4.14 is referred to as the required strength based on factored loads, and the right hand side is the design strength. Although the nominal load effects and strength as used in ACI (1984) Code are not necessarily mean values, they are reasonably close approximations (Ellingwood, 1980).

The LRFD format (e.g., Eq. 4.14) is simple to use and is not strictly method of reliability analysis. It is simply a method of safety checking and provides the basis on which other formats discussed above can be imposed; with various degrees of probabilistic sophistication (Ellingwood, 1978; MacGregor et al, 1983).

As additional data become available, and with an increases in computer usage and efficiency, the probabilistic sophistication of design engineers continues to grow, it is expected that design codes of FOSM or higher formats may eventually appear. For the next generation of codes, however, it is clear that LRFD format with factors determined from probabilistic considerations will be a compromise that is acceptable to design engineers, while at the same time capable to accommodate updated probability information (Corotis, 1985). The algorithm developed in this research is perhaps a step towards this direction.

4.4 Pile Design: Safety Measures

In the design of pile foundation, one of the major considerations is to ensure that the load "working" on the pile (or the allowable capacity) is not to exceed the "actual" capacity (the pile can sustain) before "failure." This assurance of safe performance is quantified by the value of safety measures; which are typically defined in the proceeding sections.

In deterministic design, the risk of failure is depicted by the allowable factor of safety, **FS**. From Harr (1987), **FS** can be represented as;

$$FS = \frac{\tilde{C}}{\tilde{D}} \qquad 4.15$$

where, $\tilde{C}$ and $\tilde{D}$ = nominal values of capacity (i.e., pile resistance) and demand (i.e., allowable load), respectively. It is also a common practice that the value of resistance is assigned lower than the mean. This can be expressed as;

$$\tilde{C} = \overline{C} - h_C\sigma_C \qquad 4.16$$

In pile design, this generally occurs by the designer using some conservatism in assigning the value of resistance. Likewise, $\tilde{D}$ is assigned higher than the mean. This generally occurs because the allowable load is generally assigned higher than actual; and can be expressed as;

$$\tilde{D} = \overline{D} + h_D\sigma_D \qquad 4.17$$

Substituting Eqs. 4.16 and 4.17 into 4.15 would give;

$$FS = \frac{\overline{C} - h_C\sigma_C}{\overline{D} + h_D\sigma_D} \qquad 4.18$$

where, $\overline{C}$ and $\overline{D}$ are the mean values of Capacity and Demand, respectively; h_C and h_D are h sigma units of their respective functions.

In deterministic design, if the calculated **FS** is greater than the specified minimum value (from code) or obtained from past experience, it is considered satisfactory. In reliability-based (FOSM) design, $\tilde{C}$ and $\tilde{D}$ can be treated as random variables as they cannot be determined with certainty. The ratio of the expected values, known as the central factor of safety, **CFS**, can be represented as (Harr, 1979; Ellingwood et al, 1980; and Harr, 1987);

$$CFS = \frac{E[C]}{E[D]} = \frac{\overline{C}}{\overline{D}} \qquad 4.19$$

where, $E[C]$ and $E[D]$ are the expected values of the Capacity and Demand, respectively.

As indicated by Fig. 4.2, the probability distributions of **C** and **D** will overlap if the maximum demand, D_{max}, exceeds the minimum capacity, C_{min}; and there will be a nonzero probability of failure.

The probability of failure is the shaded area in Fig. 4.2 (b), i.e., when ($SM \leq 0$), it can be expressed as;

$$P_f = P[(C - D) \leq 0] = P[SM \leq 0] \qquad 4.20$$

$$P_f = \int_0^{\infty} F_C(x)\; f_D(x)\, dx \qquad 4.21$$

where, F_C = cumulative probability distribution function (c.d.f) in **C** and f_D = probability density function (p.d.f) for **D**. For a normal distribution, e.g., then;

$$P_f = \phi\left[-\frac{\overline{C} - \overline{D}}{\sqrt{\sigma_C^2 + \sigma_D^2}}\right] \qquad 4.22$$

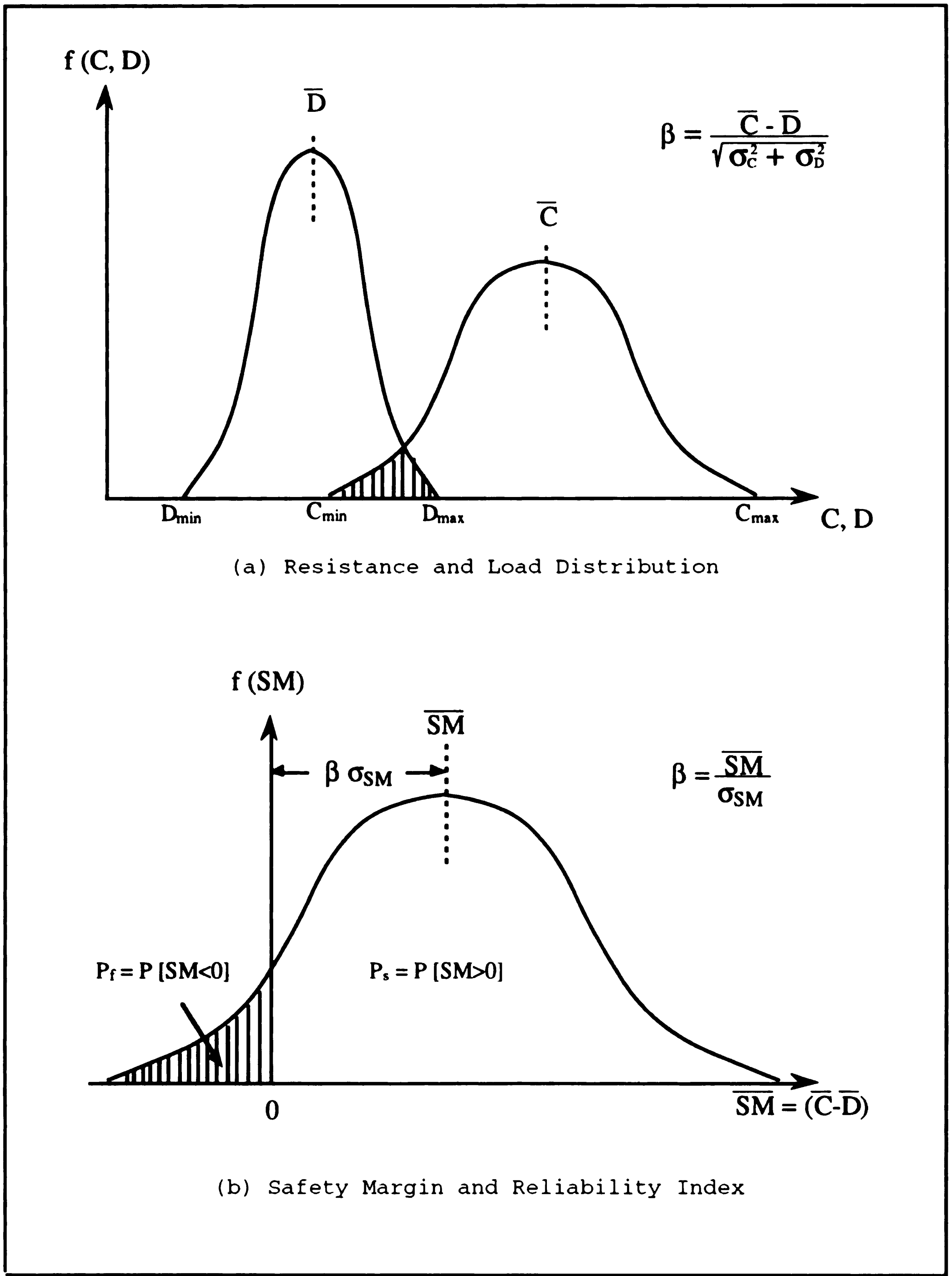


Fig. 4.2: Safety Measures - Normally Distributed FOSM Design.

where, $\phi[]$ = standard normal cumulative probability distribution (c.p.d).

Comparing Eqs. 4.22 and 4.13, the reliability index is related to the percent point function of the standard normal distribution according to;

$$\beta = \phi^{-1}(1 - P_f) \qquad 4.23$$

$$P_f = \phi(-\beta) \qquad 4.24$$

As a rule of thumb (Harr, 1987), when ($1<\beta<4$) the probability of failure, P_f, can be approximated as;

$$P_f \approx 1 * 10^{-\beta} \qquad 4.25$$

4.5 Summary

In summary, five safety measures have been defined, namely; the conventional factor of safety, **FS**; the central factor of safety, **CFS**; the safety margin, **SM**; the reliability index, β; and the probability of failure, P_f. Adaptation of these safety measures for the development of an algorithm in this research will be presented in the next chapter.

CHAPTER 5

DEVELOPMENT OF THE ALGORITHM

5.1 Introduction

This chapter presents the derivation and development of the algorithm to interrelate the safety measures for the deterministic and reliability-based design approaches.

The uncertainty associated with pertinent variables for the determination of allowable pile capacity can be incorporated in a rational design process without requiring a full and complex probabilistic analysis. The use of only nominal values of resistances and loads (as in the case of working stress deterministic design) may work well but provides us no information regarding the dispersion of variables. With the inclusion of probability theory, more information is incorporated in the analysis.

One scheme to determine the expected values of resistance and load (or capacity and demand) in the complete formula (e.g., Taylor series) requires the evaluation of derivatives. The procedure can be simple but often is cumbersome for complex functions such as shaft resistance; and certainly is not appealing for use in routine designs. To circumvent the analytical evaluation of derivatives, a First Order uncertainty analysis (e.g., Cornell, 1971) can be used. Some application of these methods can be found in Kay (1976, 1977), Wolff and Harr (1987), Sidi and Tang (1987), Harr

(1987), Wolff (1989), Wolff and Wang (1992). For example, Kay (1976, 1977) used the First Order Second Moment (FOSM) format as presented by Cornell in 1969. It is assumed that any uncertainties inherent in either the capacity (i.e., resistance) or demand (i.e., load) can be represented by the expected values (or mean) and the standard deviation of the pertinent random variables.

This chapter will also present the calibration of the calculated capacity from static formula for the determination of the predicted capacity for pile foundations in sand. Data from in-situ Standard Penetration Test are used. In general, the assumptions of FOSM method by Kay (1976, 1977), and the procedure for integration of unit shaft resistance by Wolff (1987) is adopted.

The derived algorithm will, (i) use the actual Standard Penetration Test resistance as recorded from the soil boring logs, and interpolated linearly at small discrete intervals (herein one foot), (ii) introduce calibration factors into the "standard" static pile formula by using the measured capacity from the loading tests, and (iii) interrelate deterministic safety measure with reliability-based safety measures.

From the developed algorithm, more consistent designs are possible. Consequently, engineering judgment is more explicitly quantified as compared to the present procedures in conventional design (arbitrary judgment in choosing specific parameters for the standard formula is not required).

5.2 Data Base

A total of 23 pile tests (Table 5.1) were used for the calibration of the algorithm. Eleven out of the 23 tests involved are instrumented piles from Vesic (1970) and Fruco and Associates (1964). The remaining 12 piles are from Peck (1961) and were not instrumented. The criteria for choosing these piles are, (i) SPT N-values were available in the vicinity of the pile, (ii) piles were predominantly driven into cohesionless soil, (iii) piles were not end bearing (and/or N-values of more than 100 at the toe, or piles with hard bearing layer near the toe are excluded).

All of the 23 piles were steel pipe piles. A brief summary of pile types, pile testing, pile installation, and soil profile is presented below.

5.2.1 Site at Ogeechee River

The first five tested piles (Pile No.1 through No.5) used in this study are from Test Pile 1 as reported by Vesic (1970), at the site of Ogeechee River bridge on Interstate Highway 16; approximately 18 miles west of Savannah, Georgia. This site is characterized by deep deposits of medium dense to dense well graded sand, typical for this area of the Atlantic Coastal Plain.

Standard penetration records are available from three adjacent borings; B-1, B-2 and B-69 located at about 17, 15, and 25 ft. from the test pile respectively. For the developed algorithm, the N-values are interpolated at every foot of depth for each boring and the average values from the three borings are used.

The pile was an instrumented steel pipe of 18 in. diameter and 1.5 in. wall thickness. The pile consisted of five sections each 10 ft. long allowing driving and successive testing in five stages at depths of approximately 10, 20, 30, 40 and 50 ft. The bottom section was plugged at the base with a 2 in. thick flat steel plate.

The piles were driven by a McKiernan-Terry diesel hammer with a ram weighing 2 tons. The tests were loaded by a 500 ton hydraulic jack against a water filled box girder supported at its end by reaction piles, after a waiting period of 12 hours after driving. The loading was done according to the constant rate of penetration (CRP) method as described by Whitaker and Cooke (1961).

The interpreted measured capacities, Q_m, by the 2 in. movement [2"], Davisson's [D], and Chin's [C] criteria are as presented in Appendix A. For each of these three criteria, five data points are available from this data set.

5.2.2 Locks and Dam No.4

Piles No.6 through No.11 used in this study, are from a pile testing program reported by Fruco and Associates, (1964). The test pile program was done the Locks and Dam No.4, on the lower Arkansas River below Pine Bluffs, Arkansas. The site is on east bank of the Arkansas River.

From the boring logs reported, three major soil strata exist; a surface of silt and fine sand (about 13 ft.), a deep stratum of relatively dense medium to fine sand (about 93 ft.), and a basal stratum of Tertiary clay of unknown thickness. Dry unit weights ranged from 90 - 109 pcf and did not show any significant trend with depth.

All the piles were instrumented and driven unplugged and ranged from 12 - 20 in. diameter. The steel piles were driven by a 7 ton Vulcan 140C steam hammer. The pile testing was performed using constant load (CL) increments; it was loaded by a hydraulic jack reacting against a test frame with concrete blocks. The loading was done in ten equal increments, applied and released at a rate of two tons/min. Each load was maintained for at least one hour, and a new load was applied after pile movement of 0.01 in./hr. The test was terminated when the movement was 0.005 in./hr.

5.2.3 Peck's Collection

The third set of pile tests (Pile No.12 through No.23) is from the collection of Peck (1961). Peck compiled pile tests data from all over the country for the Highway Research Board. Only the very basic information about pile and pile testing are available, such as hammer type, pile type, embedded length, and load-movement curves. No detail of the soil profile at the various sites is available except for the location of tested pile and SPT N-values. There is no mention about water table or any instrumentation. Therefore, it is assumed that the test sites were free of groundwater and that the piles were not instrumented and driven unplugged. From the characteristics of the load-movement curves, apparently the piles were tested according to the constant load (CL) procedure.

The locations of the selected piles are from several different sites as indicated from Table 5.1. The load-movement curves for several of these piles had to be extrapolated to determine the measured capacity using the 2 in. movement criteria [2"]. For the interpretation by the Davisson's [D] and Chin Criteria [C], no extrapolation is needed.

Table 5.1: Data Base For The Development of The Algorithm.

Pile No.	Pile	Length L_e (ft)	Diameter d (in)	Date Tested	Site Location
1	Ogeechee.10'	10	18	[4]NA	[1]Ogeechee River
2	Ogeechee.20'	20	18	[4]NA	[1]Ogeechee River
3	Ogeechee.30'	30	18	[4]NA	[1]Ogeechee River
4	Ogeechee.40'	40	18	[4]NA	[1]Ogeechee River
5	Ogeechee.50'	50	18	[4]NA	[1]Ogeechee River
6	L&D4.TP1	53	12	[4]NA	[2]Locks & Dam No.4
7	L&D4.TP2-1	53	16	[4]NA	[2]Locks & Dam No.4
8	L&D4.TP2-2	53	16	[4]NA	[2]Locks & Dam No.4
9	L&D4.TP3	53	20	[4]NA	[2]Locks & Dam No.4
10	L&D4.TP10	53	16	[4]NA	[2]Locks & Dam No.4
11	L&D4.TP16	39	16	[4]NA	[2]Locks & Dam No.4
12	Peck.15	74	14	3.17.54	[3]New Orleans, Louisiana
13	Peck.22	60	14	3.17.54	[3]New Orleans, Louisiana
14	Peck.51	64	14	3.7.53	[3]Jefferson City, Texas
15	Peck.55	39	16	2.17.53	[3]Brazoria County, Texas
16	Peck.206	45	12	5.3.56	[3]Richland County, Ohio
17	Peck.208	65	12	5.21.56	[3]Richland County, Ohio
18	Peck.272	33	10	10.26.56	[3]Noxon Rapids, Montana
19	Peck.274	68	10	9.21.56	[3]Noxon Rapids, Montana
20	Peck.358	49	12	12.27.55	[3]Indiana
21	Peck.359	48	12	12.1.55	[3]Indiana
22	Peck.360	50	12	12.27.55	[3]Indiana
23	Peck.361	50	12	12.8.55	[3]Indiana

[1]from Vesic (1970)
[2]from Fruco & Associates (1964)
[3]from Peck (1961)
[4]Exact date of test Not Available from the References
Pile No.11 is jetted

5.3 The Algorithm

The theoretical concepts used as the bases for the development of the algorithm are described in detail in

Chapters 2 and 4 for the deterministic and the reliability-based components of the method respectively. Limitations and difficulties associated with the specific equations and parameters have also been discussed.

Presented below are the various formulas from both methods adopted for the derivation of the algorithm.

5.3.1 Basic Concepts and Assumptions

5.3.1.1 Deterministic Formula: Q_{sc} & Q_{tc}

The analytical formula (Eq. 2.5, Section 2.4) for calculating pile capacity is expressed as the sum of the calculated shaft and calculated toe capacities, i.e;

$$Q_c = Q_{sc} + Q_{tc} \qquad 5.1$$

From Eqs. 2.7 and 2.10, the calculated shaft and calculated toe capacities are respectively defined as:

$$Q_{sc} = \sum_{i=1}^{n} \left[\left(K_s \, p'_{ov} \tan\delta \right)_i (A_s)_i \right] \qquad 5.2$$

$$Q_{tc} = \left(p'_t \, N_q^* \right) A_t \qquad 5.3$$

where other parameters are as previously defined in Section 2.4. For the calculation of the area of the pile at the toe, A_t, it is assumed that the pile is fully plugged (i.e., A_t is the cross sectional area of the pile). For piles driven without a steel cover plate at the toe, this assumes a soil plug is formed by soil arching.

5.3.1.2 Coefficient of Lateral Earth Pressure: K

The coefficient of lateral earth pressure at the site, K_s, is assumed to be equal to the empirical coefficient of lateral earth pressure at rest, K_o. Assuming that the piles are designed for **long term capacity** (and not the capacity immediately after the pile is driven), then K_o may better represent the coefficient of lateral earth pressure of the soil at the site [as opposed to adopting ($K_s > K_o$), i.e., an increased K_s immediately after pile driving, or ($K_s < K_o$) a reduction in K_s due to soil sensitivity or a loose soil condition after driving]. Therefore, as presented by Bowles (1988), Jaky's empirical correlation in 1948 for K_o is used for K_s in this study; whereby $K_o = 1 - \sin\phi$. As mentioned by Meyerhof (1976a), K_o was also an assumption adopted by Burland in his previous study.

With the inclusion of calibration factors (i.e., F_s and F_t which will be shown in Section 5.4.2), the "calibrated" β^* coefficient in the standard formula can be statistically determined to be at a maximum near the surface of the soil and decreased to a minimum near the toe of the pile, as suggested by Kaizumi (1971) and Kulhawy (1984).

5.3.1.3 Soil-Pile Interface Angle: δ

The uncertainty associated with the soil-pile interface angle (δ) has been discussed in Section 2.4.1.2. In this study, δ is taken as a reduced angle of internal friction of the soil (ϕ); where ($\delta = f_\phi \phi$). The reduction factor, f_ϕ, is adopted from a rigorous study by Potyondi (1961). Consequently, with the combination of the above assumption (i.e., $K_s = K_o$), therefore ($K_s = 1 - \sin\delta$).

5.3.1.4 Corrected N-Value: N'

The N-value is corrected for overburden pressure as proposed by Liao and Whitman (1986), which is represented as:

$$N' = N\sqrt{\frac{1}{p_t'}} \qquad (\ p_t' \ \text{in tsf}\ ) \qquad 5.4$$

5.3.1.5 Friction Angle of Soil: ϕ

Using a similar method by Wolff (1989), the friction angle of soil, ϕ, is correlated from the corrected SPT N-value, N', using the relationship as presented by Peck et al (1974), which can be approximated as:

$$\phi = 26.70 + 0.36\,N' - 0.0014\left(N'\right)^2 \qquad 5.5$$

5.3.1.6 Modified Bearing Capacity Factor: N_q^*

The Vesic (1963) modified bearing capacity factor, N_q^* is adopted; which is equal to:

$$N_q^* = e^{3.8\,\phi \tan\phi} \tan^2(45 + \frac{\phi}{2}) \qquad 5.6$$

5.3.1.7 Effective Overburden Earth Pressure: p'_{ov}

Unit weight values for sand are not available for all the collected data base. The calculated capacity is not greatly sensitive to the unit weight (about less than one ton increase for every one pcf increase for a 50 ft length of pile). Consequently, in all cases the saturated unit weight of sand, γ_{sat}, and wet unit weights of sand, γ_{wet}, *are assumed* to be 130 pcf and 120 pcf respectively. The surcharge or the overburden pressure, p_{ov}, is equal to the unit weight multiplied by the height of the soil, h (i.e., $p_{ov}=\gamma_{wet}\,h$). For the soil below water table, the effective overburden pressure, p'_{ov}, is used.

5.3.1.8 Capacity from Loading Test: Q_m

In practice, perhaps more than one method is preferable to interpret the loading tests (i.e., measured capacity, Q_m). The interpreted values should be counter checked by other criteria. For this study, therefore the measured capacity from loading test is interpreted by four criteria:

i. Criteria [2"] - Q_m at a settlement of 2 in.
ii. Criteria [D] - Q_m determined using Davisson's Method
iii. Criteria [C] - Q_m determined using Chin's Method
iv. Criteria [DC] - the **average** Q_m of criteria [D] & [C].

5.3.1.9 Safety Measures: FS & β

From reliability theory as defined by Eq. 4.13, the reliability index (β) for normally distributed random variables is;

$$\beta = \frac{\overline{C} - \overline{D}}{\sqrt{\sigma_C^2 + \sigma_D^2}} \qquad 5.7$$

The deterministic factor of safety (**FS**) as defined by Eq. 4.15 is;

$$FS = \frac{\tilde{C}}{\tilde{D}} \qquad 5.8$$

and the reliability-based central factor of safety (**CFS**) as defined by Eq. 4.19 is;

$$CFS = \frac{\overline{C}}{\overline{D}} \qquad 5.9$$

where all the respective parameters are as defined in Chapter 4.

In this study, $\overline{C}$ and $\overline{D}$ are substituted and defined as the *predicted* axial pile capacity, $\mathbf{Q_p}$, and the *allowable* capacity, $\mathbf{Q_a}$, respectively as indicated in Fig. 5.1. In slope stability studies by Wolff (1985) and Wolff and Harr (1987), a compromised reliability method was used, i.e., the Demand ($\mathbf{D}$) was assumed as a nominal value (deterministic) instead of a random variable. In this study, a similar assumption is adopted, whereby the allowable capacity is assumed as deterministic value; as it is desired to define a single "comfortable" load be allowed on the pile before "failure." Therefore, Eq. 5.9 can be represented as:

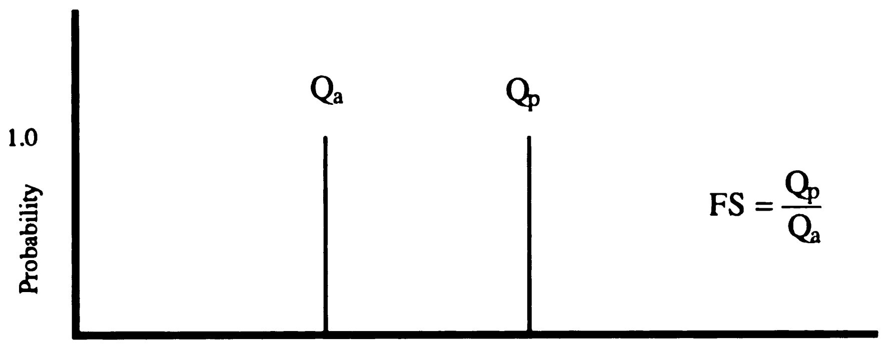
$$\mathrm{CFS} = \frac{\overline{Q}_p}{Q_a} \qquad 5.10$$

Consequently, β can be redefined as:

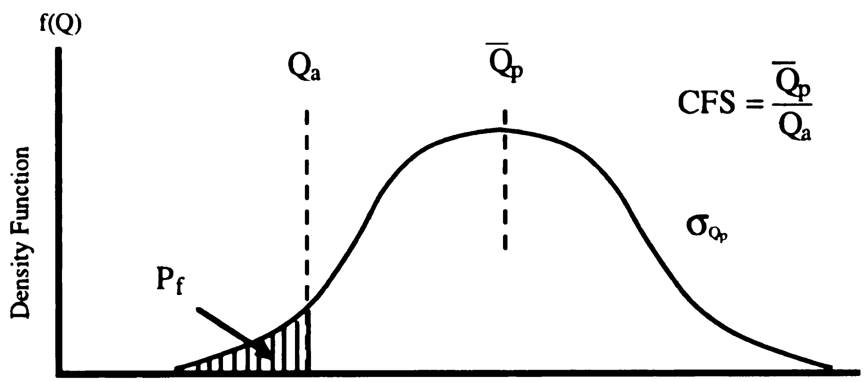
$$\beta = \frac{\overline{Q}_p - Q_a}{\sigma_{Q_p}} \qquad 5.11$$

or the number of standard deviations of the predicted capacity by which the predicted capacity exceeds the allowable capacity.

If it is desired to express reliability as a probability of failure ($\mathbf{P_f}$), it can be approximated by Eq. 4.25; however, many agencies and code writers prefer to use only β.



(a) Deterministic Model:
Q_p and Q_a - Nominal Values.



(b) The Assumed Reliability Model:
Q_p - Random Variable; Q_a - Nominal Value.

Fig. 5.1: Deterministic and Reliability Models for Safety Measures.

5.3.2 The Developed Procedure

The developed algorithm can be most easily described by considering the prediction of pile capacity and its calibration as a sequence of ten steps. Each step is fully described in the proceeding sections. After the calibration of the "standard" formula (Eq. 2.5) with the measured capacity from loading tests (Eq. 5.12), the uncertainty of the predicted capacity will be lumped in global bias factor (F_b) to be discussed.

5.3.2.1 Predicted Capacity: Q_p

The 'calculated' capacity (Q_c) for a given pile in Eq. 2.5 can be corrected to bring it into conformance with the 'measured' capacity (Q_m) as defined by one of the four criteria (see also STEP 1 in the proceeding sections). This can be done by introducing correction factors for shaft and toe capacities as developed in Steps 1 through 5 presented below. Using these factors, a consistent predicted pile capacity, Q_p, can be achieved (STEP 5). Remaining uncertainties or scatter of Q_p about Q_m can then be expressed as a "statistical bias factor" (F_b in STEP 6). The "bias-corrected" for the predicted capacity (after STEP 6) will be used for the determination of the allowable capacity (Q_a). The correlation of the safety measures as determined from the deterministic and reliability-based approach are then developed as in Steps 7 through 10.

STEP 1

Measured capacity (Q_m): The measured capacity (Q_m) from a loading test can be divided into two components of measured shaft capacity, Q_{sm}, and measured toe capacity, Q_{tm}, using loading test data from instrumented piles. Therefore Q_m can be represented as:

$$Q_m = Q_{sm} + Q_{tm} \qquad 5.12$$

[Note: For the determination of R_t in STEP 2, only data from instrumented piles were used. For the determination of F_s, F_t in STEP 4 and F_b in STEP 6, the inclusion of non instrumented piles are also considered (see STEP 3)].

STEP 2

Toe Capacity Equation (R_t): The measured toe capacity (Q_{tm}) can be represented as a portion of the total measured capacity, Q_m. This portion or percentage typically decreases as pile length increases. Therefore, the percentage of measured toe capacity (Q_{tm}/Q_m) for the instrumented and non instrumented piles, was plotted against the embedded length of pile, L_e, normalized by pile diameter, d (as in Fig. 5.3). A series of exponential curves fit through these plotted points allows the development of mathematical expressions to estimate the ratio of measured toe capacity, $(Q_{tm}/Q_m)=R_t$, as a function of dimensionless (L_e/d);

$$R_t = f\left(\frac{L_e}{d}\right) \qquad 5.13$$

Consequently, the ratio of the measured shaft resistance (Q_{sm}/Q_m) is simply $(1 - R_t)$. And these ratios can be represented as:

$$Q_{tm} / Q_m = R_t = f\left(\frac{L_e}{d}\right) \qquad 5.14$$

$$Q_{sm} / Q_m = 1 - R_t = 1 - f\left(\frac{L_e}{d}\right) \qquad 5.15$$

Substituting the ratios from Eqs. 5.14 and 5.15 into Eq. 5.12 gives:

$$Q_m = (1 - R_t)\, Q_m + (R_t)\, Q_m \qquad 5.16$$

$$Q_m = Q_{sm} + Q_{tm} \qquad 5.17$$

STEP 3

Interpretation of Measured Toe Capacity (Q_{tm}): By using the exponential functions of R_t, the measured capacity (Q_m) from instrumented and non-instrumented loading tests can now be consistently and reasonably allocated into two components, and **interpreted by Eq. 5.17** for the determination of other factors (i.e., F_s, F_t, and F_b in STEP 4 and STEP 6). This assumes that the division of shaft and toe capacities can be approximated by the function (Fig. 5.3) rather than the actual individual points from instrumented piles. The advantage of the interpretation of Q_{tm} by Eq. 5.17 is that the proportion of the shaft and toe capacities (Eq. 5.12) need to be interpolated (linearly) for each of the respective criteria considered in Section 5.3.1.8 (and see also Fig. 3.4). If the actual individual points from the loading tests are used, the interpretation of Q_{tm} according to the respective criteria could not be made. This is true even for the instrumented piles, but the assumption is especially important if data from non instrumented piles are to be included for the determination of F_s, F_t, and F_b (see also the analysis of the different Cases in Section 5.3.3).

STEP 4

Shaft and Toe Correction Factors (F_s & F_t): In practice, a limiting upper limit on shaft and toe resistances are often assumed (e.g., as found in methods by API, 1991; Meyerhof, 1976a; and findings by Vesic, 1970). However, others such as Kaizumi (1971) and Kulhawy (1984) have argued a gradually decreasing model of unit shaft and toe resistances. In this study, the depth-decreasing model arises by the introduction of correction factors as presented below. This portion of work done for this study is analogous to the previous work by Dennis and Olson (1983), but the F_t factor by Dennis and Olson is not dimensionless (see also Section 2.5).

The measured shaft and toe components obtained from Eq. 5.17 are divided with the respective components from Eqs. 5.2

and 5.3 and plotted against the dimensionless (L_e/d). Again by exponential regression (Q_{sm}/Q_{sc}) and (Q_{tm}/Q_{tc}) can be expressed as functions of (L_e/d).

$$Q_{sm} / Q_{sc} = f\left(\frac{L_e}{d}\right) \qquad 5.18$$

$$Q_{tm} / Q_{tc} = f\left(\frac{L_e}{d}\right) \qquad 5.19$$

Therefore the correction factors for shaft, F_s, and toe, F_t, is the functions in Eq. 5.18 and Eq. 5.19 respectively:

$$F_s = f\left(\frac{L_e}{d}\right) \qquad 5.20$$

$$F_t = f\left(\frac{L_e}{d}\right) \qquad 5.21$$

The fitted exponential functions for the determination of F_s and F_t by Eqs. 5.20 and 5.21 can be found in Fig. 5.4.

STEP 5

Predicted capacity (Q_p): The predicted capacity can now be obtained by modifying the calculated shaft and toe capacities from Eqs. 5.2 and 5.3 by multiplying them with the respective F_s, and F_t, to bring into conformance with capacities from the respective loading test, Q_m, resulting;

$$Q_p = F_s \; Q_{sc} + F_t \; Q_{tc} \qquad 5.22$$

$$Q_p = \sum_{i=1}^{n} \left[\left(F_s \, p' \, K_s \tan \delta\right)_i (A_s)_i \right] + F_t \left(p_t' \, N_q^* \, A_t\right) \qquad 5.23$$

Because of its form as a summation, the calculation of Q_p can be conveniently set up on a spreadsheet program with a small vertical increment (say, one foot). This procedure is adopted from Wolff (1989), but extended to include the correction factors.

STEP 6

Statistical Bias Factor (F_b): After the calibration of the calculated capacity by the F_s, and F_t functions for the individual data points (23 data points) in STEP 5, a statistical evaluation of the match was examined. It was found that the predicted capacity, Q_p, as determined by Eq. 5.23 for the whole 23 data points showed some bias. Specifically, it tended to "over predict" the capacity as compared to the respective measured capacity (see Table 5.4). To further correct this "remaining" difference or bias, the "statistical bias factor" (F_b), was introduced. It is simply the mean of the ratio of (Q_m/Q_p):

$$F_b = \overline{\left(\frac{Q_m}{Q_p}\right)} \qquad 5.24$$

[Note: As F_b is the measured divided by the predicted quantity, consequently if **(F_b>1.0)**, the pile capacity is **under predicted**, and if **(F_b<1.0)**, the pile capacity is **over predicted**.]

From the studies of Kay (1976), Kay (1977), Madhav (1979) and Dennis & Olson (1983), Jaeger and Bakht (1983) the ratio of measured to predicted (Q_m/Q_p) capacities follow a **lognormal** distribution. For design purposes, even though soil conditions over the site may be defined as "uniform," a variability in the site capacity will occur. Therefore, the capacity over a common site is treated as a random variable, and the assumption of *normality* for the distribution of *log* (Q_m/Q_p) as used by Kay (1976, 1977) is herein adopted (i.e., if a random variable X is lognormally distributed, then the log of X is normally distributed).

5.3.2.2 Interrelation of Safety Measures: FS & β

STEP 7

Reliability Index (β): Predicted capacity can be normalized by the measured capacity to remove the influence of variable geometry. Recalling Eq. 5.11, Q_p is taken as a random variable because it contains inherent uncertainty or error. Rather than trying to define uncertainty in a specific pile or specific variables, the reliability index can then be expressed in terms of the normalized **mean of log** (Q_p/Q_m) and the normalized (Q_a/Q_m), which is also one of the main assumptions in Kay (1976, 1977) and adopted in this study. Therefore, by taking the normal distribution of the logarithm to the base 10 of the ratios as used by Kay, a simpler reliability index of the lognormal distribution as in Eq. 5.25 can be applied (as opposed to other formats), see also Table 5.4.

The expected value of the log (Q_p/Q_m) [herein referred to as $\overline{\log(Q_p/Q_m)}$], and the standard deviation of the distribution of log (Q_p/Q_m) [herein referred to as S] are measures of the average and scatter associated with the model uncertainty or bias with the use of the prediction equation (as found by Eq. 5.23).

Therefore, substituting the respective parameters into Eq. 5.11, reliability index can be represented as:

$$\beta = \frac{\overline{\log(Q_p/Q_m)} - \log(Q_a/Q_m)}{S} \qquad 5.25$$

or

$$\log(Q_a/Q_m) = \overline{\log(Q_p/Q_m)} - \beta S \qquad 5.26$$

therefore

$$(Q_a/Q_m) = 10^{(\overline{\log(Q_p/Q_m)} - \beta S)} \qquad 5.27$$

In the above expressions, S is a measure of uncertainty within which the predicted and measured capacity can be matched at a given site. The value of S is expected to be larger for "non-uniform" sites and smaller for "uniform" sites. For practical purpose, the "non-uniform" site can be represented by a larger S values, which is reflected by a larger variability of the soil properties at the site (e.g., the site with a combination of sand and clay). Similarly, the "uniform" site can represent by a tighter s values at the site (e.g., site that is predominantly in sand). The values of s for the "non-uniform" and "uniform" site can be quantified from the statistical "population" (23 data points) and "sample" (individual site) considered in this study respectively.

The mean of the normal distribution associated with the ratio log, (Q_p/Q_m), as presented by Kay (1976, 1977) from Ang and Tang in 1975 is:

$$\overline{\log\left(Q_p/Q_m\right)} = \log\left(\overline{Q}_p/Q_m\right) - \frac{\ln 10}{2}\, s^2 \qquad 5.28$$

Therefore the estimate of normalized capacity, Q, i.e., $(\overline{Q}_p/Q_m)$, is now

$$(\overline{Q}_p/Q_m) = 10^{\left(\overline{\log\left(Q_p/Q_m\right)} + \frac{\ln 10}{2} s^2\right)} \qquad 5.29$$

Substituting Eqs. 5.27 and 5.29 into Eq. 5.10, **CFS** can be represented as:

$$\mathrm{CFS} = 10^{\left(\beta\, s + \frac{\ln 10}{2} s^2\right)} \qquad 5.30$$

or

$$Q_a = \frac{\overline{Q}_p}{\mathrm{CFS}} \qquad 5.31$$

Alternatively, from Eq. 5.30 reliability index can be represented as:

$$\beta = \frac{\log \mathrm{CFS} - \frac{\ln 10}{2} s^2}{s} \qquad 5.32$$

From Eq. 5.32, for any quantity of s values, the reliability index, β, can be directly related to **CFS**.

STEP 8

<u>Allowable Capacity</u> (Q_a): After the correction for statistical bias factor, F_b from Eq. 5.24, and using Q_p as found by Eq. 5.23, the final allowable axial pile capacity, Q_a, can be expressed as;

$$Q_a = \frac{F_b}{10^{(\beta s + \frac{\ln 10}{2} s^2)}} * Q_p \qquad 5.33$$

The denominator of the first term of Eq. 5.33 is the **CFS**; and knowing the quantity of s, therefore at any quantity of β values, a "unit allowable capacity," q_a, can be found; whereby ($q_a = \frac{F_b}{CFS}$).

STEP 9

<u>Design Chart</u>: Consequently, the relationship between q_a and the safety measures (**FS**, β, **CFS** and P_f) can be represented graphically (as shown in Fig. 5.5). Thus, Eq. 5.33 could be presented in graphical format which might be used as a quick design chart. The allowable pile capacity, Q_a, at a desired safety measure can be determined by multiplying q_a as determined from Fig. 5.5 (or using Eq. 5.33), and Q_p as determined from the spreadsheet program (i.e., Eq. 5.23).

STEP 10

Other Safety Measures (**FS, CFS & P_f**): The conventional factor of safety (**FS**) can be back-calculated by substituting Q_p as determined by Eq. 5.23 and Q_a as determined by Eq. 5.33 as the nominal resistance and load respectively into Eq. 5.8, consequently;

$$FS = \frac{F_b\, Q_p}{Q_a} = CFS \qquad 5.34$$

Therefore, given the axial predicted capacity and applying the recommended factor of safety frequently used in practice, the allowable pile capacities determined from SPT N-values can be found. From the derivation above, not only the **FS** is available (as in typical deterministic design procedures), but that **FS** can also be interrelated to **CFS**, β, and approximately related to P_f.

In this study, from the analysis of some "minimum required" **FS** (see Table 5.7, Section 5.3.4.3), values of **FS** and β are recommended for design purposes. The schematic representation of interrelation model developed in this study between the deterministic and reliability-based design approach is as shown in Fig. 5.2.

In general, if one can predict the axial pile capacity (as in Eq. 5.23), apply the recommended safety measure (e.g., **FS** from codes) and have a reasonable estimate of the **site** S

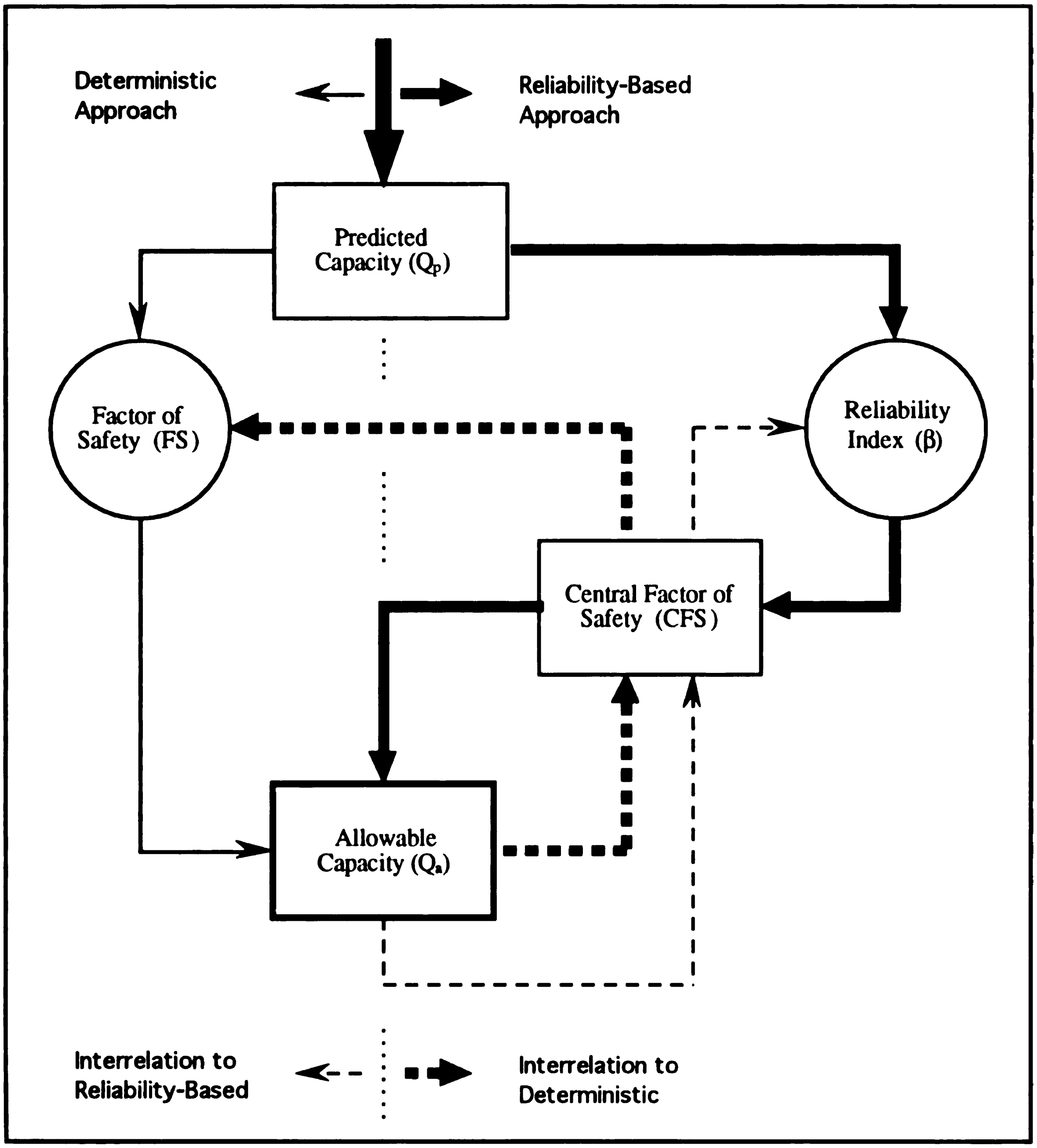


Fig. 5.2: The Schematic of Interrelation Model, Deterministic and Reliability-Based Approach.

values, a similar design chart (as in Fig. 5.5) as developed in this study, can easily be constructed. Such design chart can be considered as a "direct interrelation" of the safety measures as determined from the deterministic and reliability-based approach of design (see also Design Using The Chart, Section 6.2.1.7).

5.3.3 Determination of Calibration Parameters

All the factors subsequently derived (i.e., F_s and F_t, and consequently F_b, and S) relate back to the accuracy of the equation of R_t (see STEP 2 and STEP 3). However, a rigorous parametric analysis was not considered in this study for determination of these factors. Instead, simple curve fitting using a computer program was employed. They are found to best fit exponential functions in all cases (other simple curve fitting capabilities in the program are such as linear, polynomial, logarithmic and power with the options of smoothing, weighted, cubic spline and interpolation).

The proceeding sub-sections will present the determination and analysis of R_t, F_s, F_t, F_b, and S. Ultimately, the recommended values for the determination of the allowable capacity (Q_a) using the algorithm will be summarized in Section 5.4.2.

5.3.3.1 Equations of Toe Capacity Ratio: R_t

The measured capacity (Q_m) for the 11 instrumented piles according to the three criteria ([2"], [D] and [C]) respective criteria are presented in Appendix A. For the determination of R_t in STEP 2, the curves of (Q_{tm}/Q_m) vs (L_e/d) are plotted as shown in Fig. 5.3 for the respective criteria (i.e., [2"], [D], [C], and [DC]). The curves in Fig. 5.3 use data from the 11 instrumented piles only.

In Fig. 5.3, for each criteria it seemed that two curves of R_t could be plotted independently for the five piles from the Ogeechee site and six piles from Locks and Dam No.4 site. However, assuming that the represented population is many more pile tests, this tendency would be "masked out" as more points are added and scattered around the curves. Therefore, "an average of the two sites" is considered and only one curve of R_t is plotted for each criteria. What is important in Fig. 5.3 is the general trend and the coefficients of the functions.

The form of the exponential functions in Fig. 5.3 is:

$$R_t = a * \mathrm{Exp}\,(-b * L_e / d)$$

where **a** and **b** are constant coefficients.

As indicated in Fig. 5.3, the coefficients **a** for all of these equations are found to be very close to unity, and it seemed appropriate to adjust the coefficient **a** to unity.

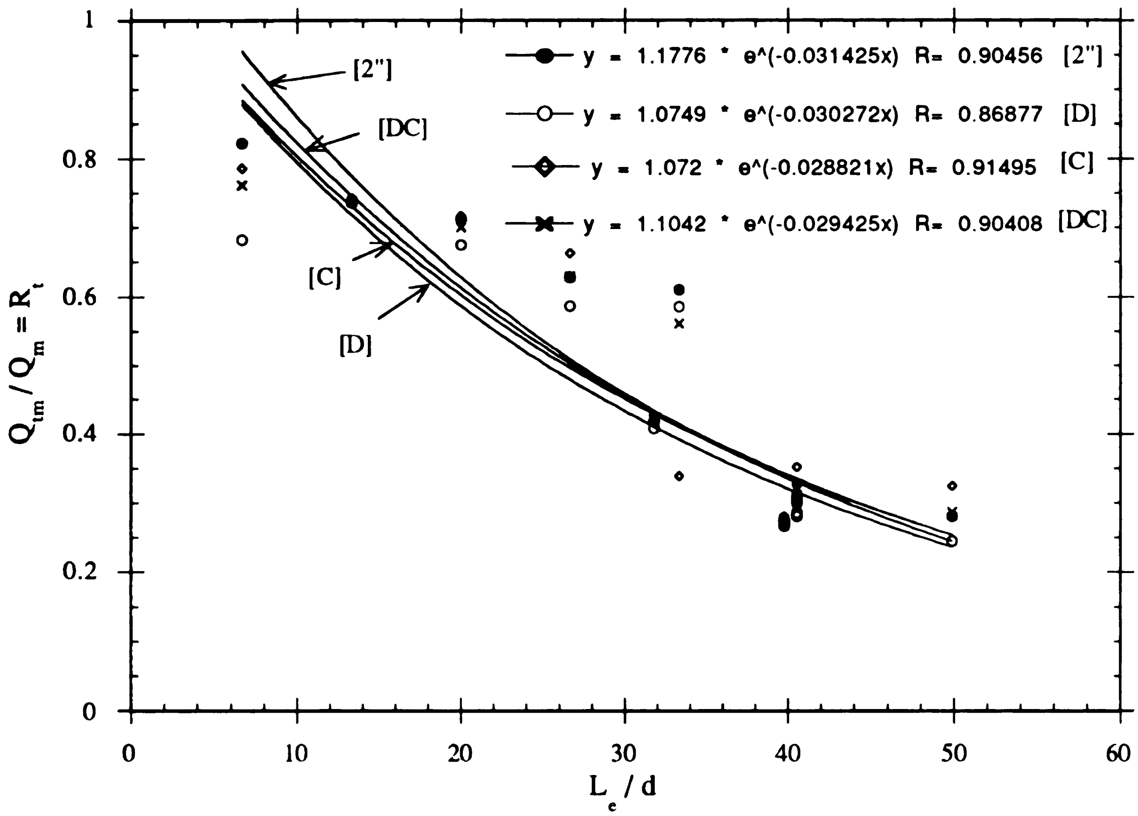


Fig. 5.3: Development of R_t Functions Using 11 Instrumented Piles.

Another practical reason for this adjustment is that, at the surface of the soil, the shaft capacity is zero, and capacity is only attributed to the toe. The toe capacity [consequently the ratio (Q_{tm}/Q_m)] is a maximum for a pile placed at the soil surface and this ratio decreases with depth. Conversely, the shaft capacity increases gradually from zero at the soil surface. However, when **a** is adjusted to unity, in all cases the predicted shaft capacity tended to be "over estimated" quite substantially, especially for piles of more than 50 ft.

of embedded length. Consequently, the equations of $\mathbf{R_t}$ used in subsequent steps are unadjusted, but interpreted by equations as found in Fig. 5.3 (see STEP 2 and STEP 3). Nevertheless, in practice, this is generally not a concern as piles of [$(L_e/d)<10$] are seldom used.

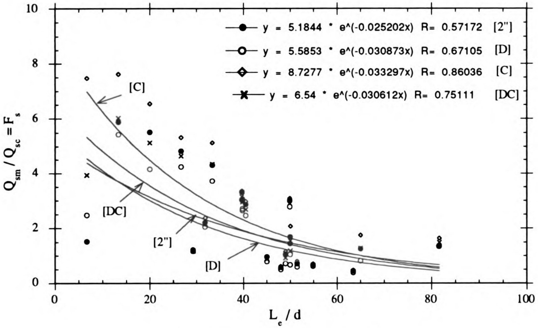
The ratio of the toe capacity, Q_{tm}, to the total measured capacity, Q_m, does not show a significant variation across the four criteria.

5.3.3.2 Shaft and Toe Factors: F_s & F_t

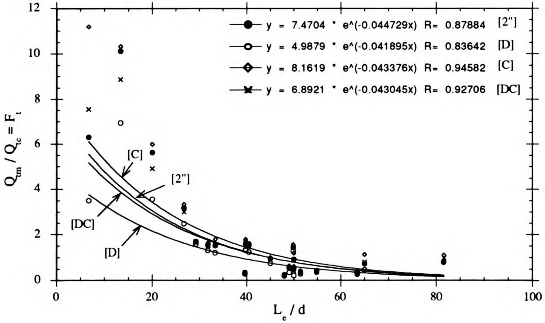
The interpreted measured shaft capacity is simply the difference between the total measured capacity, Q_m, and the interpreted measured toe capacity, Q_{tm}.
For the determination of correction factors in STEP 4, the calculated shaft capacities, Q_{sc}, and the calculated toe capacities, Q_{tc}, are obtained from Eqs. 5.2 and 5.3 respectively. Following STEP 4, the ratios of estimated measured to calculated shaft and toe capacities (Q_{sm}/Q_{sc}) and (Q_{tm}/Q_{tc}), (or F_s and F_t) were respectively plotted against (L_e/d) as shown in Fig. 5.4. All points show a substantial scatter within each of the methods. However, a trend is clearly observed for all the curves, i.e., they decrease with an increase in depth. All curves are found to best fit to exponential functions as shown in Fig. 5.4.

A similar study by Dennis and Olson (1983) for the determination of F_s and F_t in a similar manner but by plotting (Q_{sc}/Q_{sm}) vs. (L_e/d) and (Q_{tc}/Q_{tm}) vs. (L_e) respectively. That study was done on a large collection of tested piles (over 1004), but only 21 data points from instrumented piles were use for the determination of F_s and F_t (i.e., piles with similar consistency in terms of properties, types, etc.). In their study, substantial scatter was also observed. From the points plotted in that study, the general trend (an increase in their case) of F_s and F_t is not clearly seen. In their study, even though the F_s was found to be dimensionless, the F_t obtained was *not* dimensionless. From an analytical standpoint, dimensionless factors seem preferable (and was used herein).

Table 5.2 presents the equations for R_t, F_s and F_t factors as found from Figs. 5.3 and 5.4. The recommended equation for the determination of Q_p by Eq. 5.23 (STEP 5) will be presented Table 5.5.

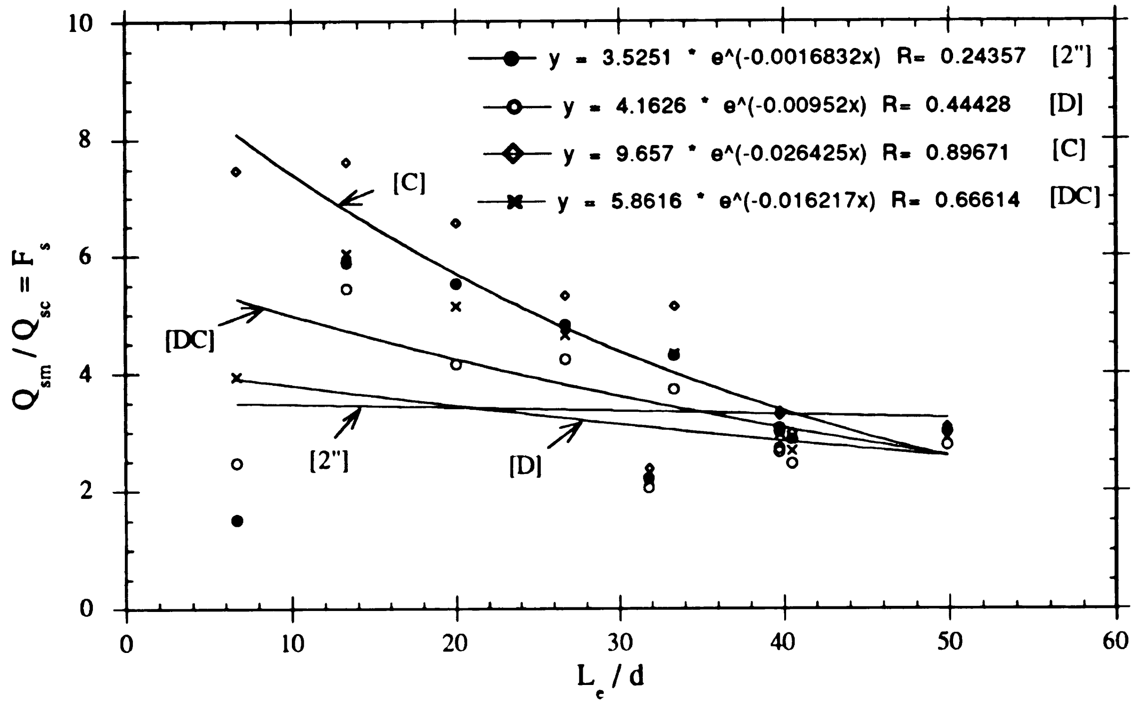


(a) F_s - 23 Data Points

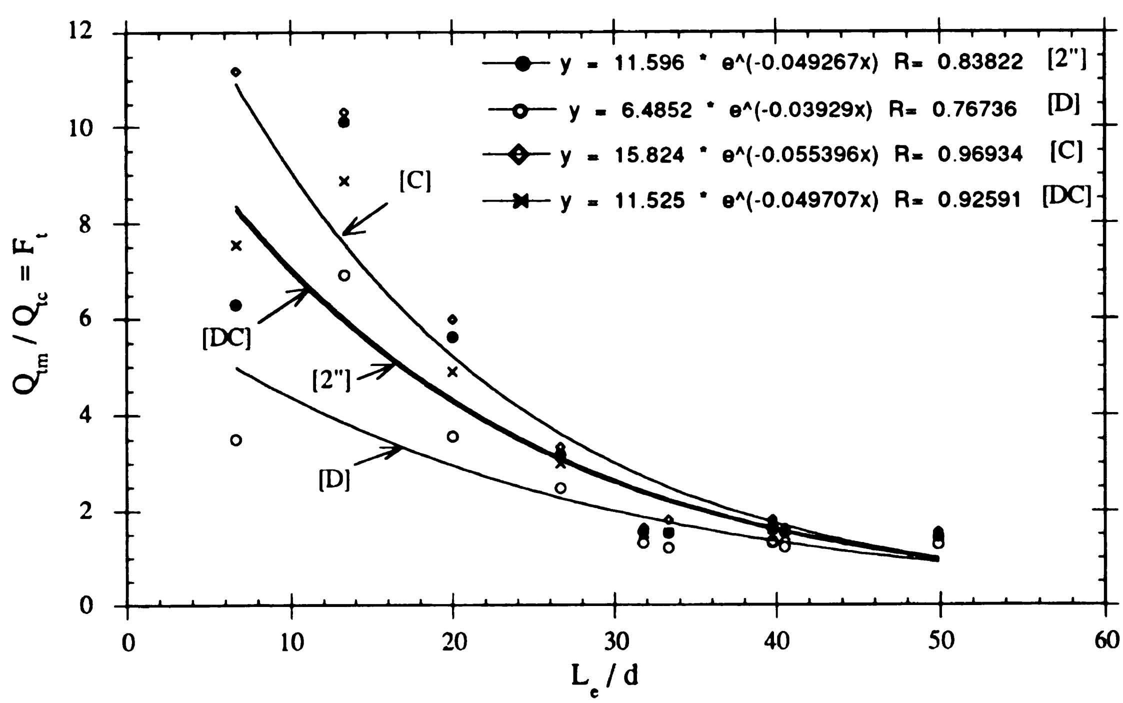


(b) F_t - 23 Data Points

Fig. 5.4: Correction Factors For Shaft and Toe - F_s & F_t.



(c) F_s - 11 Data Points



(d) F_t - 11 Data Points

Fig. 5.4: Continued.

Table 5.2: Equations for R_t, F_s and F_t Factors.

FACTOR	CRITERIA	EQUATION	CORRELATION COEFFICIENT
R_t (11 pts)	[2"]	1.1775 * Exp(-0.0314*L_e/d)	0.90
	[D]	1.0748 * Exp(-0.0327*L_e/d)	0.87
	[C]	1.0720 * Exp(-0.0288*L_e/d)	0.91
	[DC]	1.1042 * Exp(-0.0294*L_e/d)	0.90
F_s (11 pts)	[2"]	3.5251 * Exp(-0.00168*L_e/d)	0.24
	[D]	4.1626 * Exp(-0.00952*L_e/d)	0.44
	[C]	9.6570 * Exp(-0.02643*L_e/d)	0.90
	[DC]	5.8616 * Exp(-0.01622*L_e/d)	0.67
F_t (11 pts)	[2"]	11.596 * Exp(-0.04927*L_e/d)	0.84
	[D]	6.4852 * Exp(-0.03929*L_e/d)	0.77
	[C]	15.824 * Exp(-0.05540*L_e/d)	0.97
	[DC]	11.525 * Exp(-0.04971*L_e/d)	0.93
F_s (23 pts)	[2"]	5.1844 * Exp(-0.02520*L_e/d)	0.57
	[D]	5.5853 * Exp(-0.03087*L_e/d)	0.67
	[C]	8.7277 * Exp(-0.03330*L_e/d)	0.86
	[DC]	6.5400 * Exp(-0.03061*L_e/d)	0.75
F_t (23 pts)	[2"]	7.4704 * Exp(-0.04473*L_e/d)	0.88
	[D]	1.3438 * Exp(-0.04190*L_e/d)	0.84
	[C]	2.1907 * Exp(-0.04338*L_e/d)	0.95
	[DC]	2.5659 * Exp(-0.04305*L_e/d)	0.93

5.3.3.3 Bias Factor and Site Variability: F_b & S

For the determination of F_b and S, three different cases were examined as listed in Table 5.3. The "best" parameters are compared and recommended in Section 5.4.2.2.

Case (i) represents the viewpoint that a greater number of points with less precision will provide the most well-found information; Case (ii) represents precision is more important than quantity; and Case (iii) represents the viewpoint that precision should be examined for F_s and F_t, but quantity is important for the determination of F_b and S.

To quantify the "best" values of F_b and S, within the three cases, the values from 23 data points could be taken as the "population" representing the "non-uniform" site. Similarly, the values from the individual sites could be taken as the "sample" representing the "uniform" site (see also the assumptions in STEP 7).

In Table 5.3, 11 data points are from instrumented piles and 12 are from non instrumented piles. Table 5.4 are the results of F_b and S for the three cases examined.

Recalling STEP 6, F_b is the statistical bias factor associated with the predicted capacity, Q_p, as determined from Eq. 5.23. As shown in Table 5.4, the bias factor is simply the mean of (Q_m/Q_p). Consequently, if $(F_b<1.0)$ the capacity is over predicted, and if $(F_b>1.0)$ the pile capacity is under predicted. If the predicted capacity, Q_p, as determined from Eq. 5.23 is "perfect," than F_b should be unity. However, it was found in this study is that most calculated F_b factors are less than 1.0 (Table 5.4). This indicates that the derived prediction equation will somewhat "over predict" pile capacities with respect to the "true" measured capacities. An exception is the criteria [2"] from Case (ii) which showed under prediction. Another exception is the Ogeechee site when considered on its own, for all Cases (i), (ii) and (iii). With known quantities of F_b factors, the allowable capacity, Q_a, can easily be determined from Eq. 5.33.

Comparing among the *cases*, the "best" F_b to be suggested appears to be from Case (ii), at F_b equals to 1.136, 0.977, 0.863, and 0.952 for criteria [2"], [D], [C], and [DC] respectively. This is also true for S which ranges from 0.10 to 0.12. For Cases (i) and (iii), the S values are at about 0.27; which matches well with the implied larger uncertainty of S of the "population."

Table 5.3: The Three Schemes Examined For The Determination of Calibration Factors - F_b, and S.

FACTOR	NO. OF DATA POINTS USED FOR CALIBRATION		
R_t	11		
	Case (i)	Case (ii)	Case (iii)
F_s	23	11	11
F_t	23	11	11
F_b	23	11	23
S	23	11	23

Table 5.4: Bias Factor and Site Variability.

(a) F_b & S: Case (i).

	PILE	[2"] Q_m/Q_p	[D] Q_m/Q_p	[C] Q_m/Q_p	[DC] Q_m/Q_p	[2"] Log Q_m/Q_p	[D] Log Q_m/Q_p	[C] Log Q_m/Q_p	[DC] Log Q_m/Q_p
1	Ogeechee.10'	1.024	0.844	1.672	1.325	0.01	-0.07	0.22	0.12
2	Ogeechee.20'	2.113	1.943	1.810	1.869	0.32	0.29	0.26	0.27
3	Ogeechee.30'	1.692	1.381	1.461	1.446	0.23	0.14	0.16	0.16
4	Ogeechee.40'	1.425	1.398	1.186	1.288	0.15	0.15	0.07	0.11
5	Ogeechee.50'	1.157	1.150	1.078	1.117	0.06	0.06	0.03	0.05
6	L&D4.TP1	1.392	1.443	1.116	1.283	0.14	0.16	0.05	0.11
7	L&D4.TP2-1	1.180	1.135	0.985	1.072	0.07	0.06	-0.01	0.03
8	L&D4.TP2-2	1.157	1.161	0.968	1.072	0.06	0.06	-0.01	0.03
9	L&D4.TP3	0.776	0.792	0.626	0.709	-0.11	-0.10	-0.20	-0.15
10	L&D4.TP10	1.127	1.075	0.896	0.993	0.05	0.03	-0.05	0.00
11	L&D4.TP16	0.883	0.821	0.685	0.759	-0.05	-0.09	-0.16	-0.12
12	Peck.15	0.266	0.260	0.230	0.249	-0.58	-0.59	-0.64	-0.60
13	Peck.22	0.346	0.321	0.303	0.319	-0.46	-0.49	-0.52	-0.50
14	Peck.51	0.342	0.354	0.285	0.322	-0.47	-0.45	-0.55	-0.49
15	Peck.55	0.497	0.546	0.368	0.454	-0.30	-0.26	-0.43	-0.34
16	Peck.206	0.464	0.420	0.361	0.398	-0.33	-0.38	-0.44	-0.40
17	Peck.208	0.744	0.551	0.840	0.740	-0.13	-0.26	-0.08	-0.13
18	Peck.272	0.558	0.621	0.464	0.522	-0.25	-0.21	-0.33	-0.28
19	Peck.274	0.967	1.115	0.982	1.058	-0.01	0.05	-0.01	0.02
20	Peck.358	0.485	0.365	0.414	0.405	-0.31	-0.44	-0.38	-0.39
21	Peck.359	0.232	0.177	0.157	0.230	-0.63	-0.75	-0.80	-0.64
22	Peck.360	0.806	0.552	0.771	0.702	-0.09	-0.26	-0.11	-0.15
23	Peck.361	0.623	0.326	0.576	0.486	-0.21	-0.49	-0.24	-0.31
		Mean (F_b)				Standard Deviation (S)			
	23 Data pts	**0.881**	**0.815**	**0.793**	**0.818**	**0.26**	**0.28**	**0.29**	**0.26**
	Ogeechee Site	1.482	1.343	1.441	1.409	*0.13*	*0.13*	*0.10*	*0.08*
	L&D4 Site	1.086	1.071	0.880	0.981	*0.09*	*0.10*	*0.10*	*0.10*
	Peck's Data	0.527	0.467	0.479	0.490	*0.19*	*0.21*	*0.24*	*0.20*

	[2"]	[D]	[C]	[DC]
F_b (23 pts)	0.881	0.815	0.793	0.818
S (23 pts)	0.26	0.28	0.29	0.26
S (Ogeechee Site)	0.13	0.13	0.10	0.08
S (L&D4 Site)	0.09	0.10	0.10	0.10
S (Peck's col'n)	0.19	0.21	0.24	0.20

Table 5.4: Continued

(b) F_b & S : Case (ii).

	PILE	[2"] Q_m/Q_p	[D] Q_m/Q_p	[C] Q_m/Q_p	[DC] Q_m/Q_p	[2"] Log Q_m/Q_p	[D] Log Q_m/Q_p	[C] Log Q_m/Q_p	[DC] Log Q_m/Q_p
1	Ogeechee.10'	0.733	0.689	1.004	0.896	-0.14	-0.16	0.00	-0.05
2	Ogeechee.20'	1.715	1.678	1.244	1.425	0.23	0.22	0.09	0.15
3	Ogeechee.30'	1.450	1.175	1.067	1.150	0.16	0.07	0.03	0.06
4	Ogeechee.40'	1.256	1.148	0.901	1.042	0.10	0.06	-0.05	0.02
5	Ogeechee.50'	1.012	0.893	0.838	0.898	0.01	-0.05	-0.08	-0.05
6	L&D4.TP1	1.264	0.997	0.856	0.990	0.10	0.00	-0.07	0.00
7	L&D4.TP2-1	1.093	0.862	0.771	0.868	0.04	-0.06	-0.11	-0.06
8	L&D4.TP2-2	1.072	0.881	0.758	0.868	0.03	-0.05	-0.12	-0.06
9	L&D4.TP3	0.717	0.641	0.487	0.584	-0.14	-0.19	-0.31	-0.23
10	L&D4.TP10	1.046	0.811	0.701	0.803	0.02	-0.09	-0.15	-0.10
11	L&D4.TP16	0.697	0.621	0.536	0.591	-0.16	-0.21	-0.27	-0.23
		Mean (F_b)				Standard Deviation (S)			
	11 Data pts	**1.136**	**0.977**	**0.863**	**0.952**	**0.12**	**0.12**	**0.11**	**0.10**
	Ogeechee Site	1.233	1.117	1.011	1.082	*0.14*	*0.14*	*0.07*	*0.08*
	L&D4 Site	0.982	0.802	0.685	0.784	*0.11*	*0.08*	*0.10*	*0.10*

	[2"]	[D]	[C]	[DC]
F_b (11 pts)	1.136	0.977	0.863	0.952
S (11 pts)	0.12	0.12	0.11	0.10
S (Ogeechee Site)	0.14	0.14	0.07	0.08
S (L&D4 Site)	0.11	0.08	0.10	0.10

Table 5.4: Continued

(c) F_b & S : Case (iii).

	PILE	[2"] Q_m/Q_p	[D] Q_m/Q_p	[C] Q_m/Q_p	[DC] Q_m/Q_p	[2"] Log Q_m/Q_p	[D] Log Q_m/Q_p	[C] Log Q_m/Q_p	[DC] Log Q_m/Q_p
1	Ogeechee.10'	0.733	0.689	1.004	0.896	-0.14	-0.16	0.00	-0.05
2	Ogeechee.20'	1.715	1.678	1.244	1.425	0.23	0.22	0.09	0.15
3	Ogeechee.30'	1.450	1.175	1.067	1.150	0.16	0.07	0.03	0.06
4	Ogeechee.40'	1.256	1.148	0.901	1.042	0.10	0.06	-0.05	0.02
5	Ogeechee.50'	1.012	0.893	0.838	0.898	0.01	-0.05	-0.08	-0.05
6	L&D4.TP1	1.264	0.997	0.856	0.990	0.10	0.00	-0.07	0.00
7	L&D4.TP2-1	1.093	0.862	0.771	0.868	0.04	-0.06	-0.11	-0.06
8	L&D4.TP2-2	1.072	0.881	0.758	0.868	0.03	-0.05	-0.12	-0.06
9	L&D4.TP3	0.717	0.641	0.487	0.584	-0.14	-0.19	-0.31	-0.23
10	L&D4.TP10	1.046	0.811	0.701	0.803	0.02	-0.09	-0.15	-0.10
11	L&D4.TP16	0.697	0.621	0.536	0.591	-0.16	-0.21	-0.27	-0.23
12	Peck.15	0.223	0.155	0.166	0.173	-0.65	-0.81	-0.78	-0.76
13	Peck.22	0.312	0.218	0.231	0.243	-0.51	-0.66	-0.64	-0.61
14	Peck.51	0.306	0.234	0.214	0.241	-0.51	-0.63	-0.67	-0.62
15	Peck.55	0.485	0.462	0.287	0.386	-0.31	-0.34	-0.54	-0.41
16	Peck.206	0.437	0.306	0.276	0.315	-0.36	-0.51	-0.56	-0.50
17	Peck.208	0.621	0.326	0.606	0.514	-0.21	-0.49	-0.22	-0.29
18	Peck.272	0.463	0.449	0.374	0.413	-0.33	-0.35	-0.43	-0.38
19	Peck.274	0.727	0.569	0.675	0.665	-0.14	-0.25	-0.17	-0.18
20	Peck.358	0.445	0.256	0.318	0.316	-0.35	-0.59	-0.50	-0.50
21	Peck.359	0.220	0.165	0.155	0.171	-0.66	-0.78	-0.81	-0.77
22	Peck.360	0.733	0.381	0.589	0.540	-0.14	-0.42	-0.23	-0.27
23	Peck.361	0.562	0.225	0.447	0.378	-0.25	-0.65	-0.35	-0.42
		Mean (F_b)				Standard Deviation (S)			
	23 Data pts	**0.765**	**0.615**	**0.587**	**0.629**	**0.25**	**0.30**	**0.27**	**0.27**
	Ogeechee Site	*1.233*	*1.117*	*1.011*	*1.082*	*0.14*	*0.14*	*0.07*	*0.08*
	L&D4 Site	0.982	0.802	0.685	0.784	*0.11*	*0.08*	*0.10*	*0.10*
	Peck's Data	*0.461*	*0.312*	*0.361*	*0.363*	*0.18*	*0.18*	*0.22*	*0.19*

	[2"]	[D]	[C]	[DC]
F_b (23 pts)	0.765	0.615	0.587	0.629
S (23 pts)	0.25	0.30	0.27	0.27
S (Ogeechee Site)	0.14	0.14	0.07	0.08
S (L&D4 Site)	0.11	0.08	0.10	0.10
S (Peck's col'n)	0.18	0.18	0.22	0.19

5.3.4 Importance of F_b and S

The values of F_b and S are important, especially the S values as a small change in S could affect Q_a by a substantially amount. Similarly, the safety measures are also sensitive to the S values (see Design Charts in Figs. 5.5, 5.6 and 5.7).

Nevertheless, low S values were observed for **each site,** i.e., when they are considered for individual sites only (for all the Cases). For Case (i), S ranges from 0.09 to 0.10 and 0.13 to 0.08 for Locks and Dam No.4 and Ogeechee sites respectively (Peck's collection cannot be considered as a "site" since the piles were collected from several locations as shown in Table 5.1. For Case (ii), S ranges from 0.11 to 0.10 and 0.14 to 0.08 for Locks and Dam No.4 and Ogeechee sites respectively. Overall, the site S values are remarkably low at about 0.08 to 0.14. These values coincide very well with site S values of 0.12 as previously proposed by Kay (1976), who used published values of Q_m, and Q_p as determined from dynamic formula. In a similar study by Kay (1977) in clay, a site S value of 0.08 was used. In another study (Bourquard, 1987), the proposed value of site S was 0.09. In Bourquard (1987) study, however the methods of determining Q_m, and Q_p were not mentioned.

Therefore, the S values obtained from this study compare vary well with values found in previous studies (Kay, 1976; Kay, 1977; and Bourquard, 1987). However, the algorithm developed in this study has the advantage of being consistent and with repeatable calculation; consequently, no "arbitrary" judgment is required in choosing the specific parameters in the standard static equation is required. What is required is only the SPT N-values from the boring log.

Another trend that is observed from the values of (Q_m/Q_p) in Table 5.4 is that the derived algorithm tends to "over estimate" the predicted capacity, especially for Locks and Dam No.4 and Peck's collections) and the algorithm tends to "under estimate" the predicted capacities from the Ogeechee site. This gives a notion that at least two set of F_b should be suggested according to loading test procedure, i.e., Constant Rate of Penetration (CRP) or Constant Load, CL (recall from the data base in Section 5.2: Ogeechee site is CRP type of loading test, and Locks and Dam No.4 and Peck's collections are CL type of loading test). Consequently, three values of F_b are recommended in Table 5.6. The "unknown loading test" condition is where the determination of measured capacity from loading test *to be run has not been decided* with respect to *type*; or the condition where *no loading test is anticipated* (NLT) at the site. (Note: the loading test of piles could be run according to CRP, CL, or some other types of loading schemes).

The F_b factors and **s** values suggested in Table 5.6 are from the results of Ogeechee site for CRP type of loading test [i.e., values from Case (ii)] and values from Peck's collection for CL type of loading test [i.e., values from

Case (iii)]. In cases of predicting pile capacity where type of loading tests is unknown (NLT), only relying on the available SPT N-values, the F_b factor from the average of Ogeechee and Locks and Dam 4 from Case (ii) is recommended.

For the purpose of determining the allowable capacity, the **"uniform"** site may now be quantified from the tighter S values from each of the individual sites where uniform soil properties at the site is expected. Another words, it can be said that the site is predominantly in **sand with no clay**. The recommended S values for the "uniform" site are recommended from the average of Ogeechee and Locks & Dam No.4 sites from Case (ii). Likewise, the **"non-uniform"** site may now be quantified from the larger S values of the "population", where non-uniform site properties are stipulated (the site of predominantly in **sand with some clay**. The S values for the "non-uniform" site in Table 5.6 are recommended from the "population" S values from Case (iii).

5.3.5 Safety Measures: FS and β

For design purposes, the determination of the "best" safety measures recommended in Table 5.7, the following examination was made.

The bias factors, F_b recommended for the design charts, are from the 11 instrumented piles from Case (iii), therefore for the determination of the recommended safety measures, Q_m from Case (iii) of the "population" were examined.

The "minimum required" **FS** is defined as the average value of **FS** when the predicted capacity is divided by the measured capacity [$FS=(Q_p/Q_m)$] for the 23 data points, instead of the normal definition of $FS=(Q_p/Q_a)$. This is the condition when the load allowed on the pile (Q_a) **"is not greater than"** the measured capacity from loading test (Q_m); i.e., the condition of safe state at the lower bound. The minimum required **FS** typically ranges from 1.79 to 2.25; and the values are the *same* for the "uniform" site and "non-uniform" site. This indicates that if the design is done according to the deterministic approach, the **FS** applied could be the same for the "uniform" and "non-uniform" sites, and it is not possible to quantify the uncertainty at the unknown site. What is commonly done in practice is that, knowing that the site has a "non-uniform" soil properties, a higher **FS** value is typically "assigned."

Similarly as in the case of deterministic **FS**, a "minimum required" β is determined by using the relation as given by Eqs. 5.32 and 5.33, and by assuming that (**FS=CFS**). Typical values for the "minimum required" β ranges from 1.43 to 2.93 and 0.46 to 0.88 for the "uniform" and "non-uniform" site respectively. The lower β values for the "unknown site" indicate that the reliability *is reduced* at the "non-uniform" site due to inherent "uncertainties of the non-uniform site." This is perhaps one of the main advantage of reliability method as compared to deterministic approach, whereby the "non-uniform" or the variability at a particular site can rationally be *quantified*.

In practice for *deterministic designs*, a minimum **FS** of 2.0 is frequently provided (e.g., Fuller, 1978). Consequently, in this study **FS**=2.0 or greater than the "minimum required" (and rounded to one decimal place) are recommended in Table 5.7 for the "uniform" sites. For a "non-uniform" site the **FS** is recommended at 3.0, at which the equivalent β is not less than 1.0.

For a "non-uniform" site where S is relatively larger than the "uniform" site, the equivalent β at the "minimum required" **FS** is found to be 0.46, 0.88, 0.75, and 0.79 for criteria [2"], [D], [C], and [DC] respectively. In other words, to provide **FS** at the "minimum required," the "minimum required" β could be very low. From deterministic perspective, considering that the "non-uniform" is more uncertain, intuitively it should be given a higher **FS**, say twice the **FS** value at the "uniform" site, i.e., **FS**=4.0).

When (**FS**=4.0) is assigned to the "non-uniform" site, the respective β was found to be 1.86, 1.59, 1.56, and 1.71 for criteria [2"], [D], [C], and [DC] respectively; which is substantially higher than the "minimum required" β. This higher value is about the same value as proposed by Meyerhof (1970), i.e., at β=1.7. Therefore, the **FS**=4.0 seems to be an appropriate value for the "non-uniform" site if the design is done from deterministic approach.

If a design is done from *reliability-based approach*, the recommended **β** are 2.00, 3.00, 2.50, and 3.00 for criteria [2"], [D], [C], and [DC] respectively for the "uniform" site (values larger than "minimum required" **β**, not less than an equivalent **FS**=2.0 and rounded to one decimal place) . At the recommended **β** values, the equivalent values of **FS** are 2.10, 2.51, 2.43, and 2.33 for the respective criteria. Therefore, these **β** values are recommended in Table 5.7. For the "non-uniform" site, all criteria at **β**=1.00 are found to be adequate (against the "minimum required" **β**), which is also equivalent to **FS** of 2.10, 2.67, 2.82, and 2.57 for criteria [2"], [D], [C], and [DC] respectively. At these **FS**, **β** is higher than the "minimum required" and also adequate to satisfy the minimum (**FS**=2.0). In other words, for the "non-uniform" site, a **β** of only 1.00 could have been recommended, at which the equivalent **FS** value of at least 2.00 could be met.

Nevertheless, the **β** values for the "non-uniform" site recommended in Table 5.7 for the reliability-based approach are the ones *that could also satisfy deterministic* **FS** *of at least* 3.0, considering arguments below.

For the **FS** of the "non-uniform" site, comparing the higher **FS** of the first assumed at **FS**=4.00 from the deterministic approach, this is perhaps on the conservative side. Comparing the equivalent **FS** from reliability approach, which could in fact be smaller than 4.00, the **FS** is revised to 3.00. At **FS**=3.0, the equivalent **β** are found to be 1.36, 1.17, 1.10, and 1.25 criteria [2"], [D], [C], and [DC] respectively. As these values are also higher than the "minimum required" these values are finally recommended in Table 5.7 for deterministic approach.

Therefore from this analysis, it can be said that the *same* **FS** could result in a *different* β (and consequently the probability of failure), and that the deterministic **FS** alone is insufficient to assure acceptable reliability (thus, also an economic design) at the applied factor safety. The design by deterministic approach could also be overly conservative. It is clear that the reliability approach does tell more about the uncertainty (which can also be quantified) as compared to the deterministic approach. In general, from the analysis done using the reliability approach in this study, the recommended deterministic **FS** higher than 3.0 frequently applied in practice for pile design should be examined closely as it could be on the high side.

Table 5.7 presents the recommended safety measures for design purposes either from deterministic or reliability-based approaches; and their respective equivalence is presented in italics. When these recommended values are used to check the 11 instrumented piles, in all cases, it was found that (Q_a<Q_m); which is necessary for safe design.

5.4 Summary of The Algorithm

From the developed algorithm in this study, the determination of the allowable pile capacity from the SPT N-values is summarized in four steps as presented in Section 5.4.3. Section 5.4.1 will present the assumptions adopted; and section 5.4.2 will present the recommended calibration parameter to be used with the "standard" static pile formula.

5.4.1 Implied Assumptions

For the determination of predicted capacity, Q_p, the following assumptions are implied:

(i) the correlation of N-values and ϕ is adopted from Peck's approximation; i.e., Eq. 5.5,

(ii) the soil-pile interface angle, δ, is a reduced ϕ using factors adopted from Potyondy (1961); i.e., 0.76 and 0.80 for above and below water table respectively (Table 2.4),

(iii) the SPT N-values are corrected for overburden pressure using correlation from Liao and Whitman (1986): i.e., Eq. 5.4,

(iv) the wet unit weights (γ_{wet}) and saturated unit weights (γ_{sat}) are assumed to be 120 and 130 pcf respectively,

(v) the coefficient of lateral earth pressure at the site, K_s, is taken as equal to the coefficient of lateral earth pressure at rest K_o [i.e., $K_s = K_o = (1 - \sin\delta)$].

5.4.2 Recommended Calibration

From the previous sections, this section summarizes the recommended calibration parameters to "modify" the standard static formula.

5.4.2.1 Values of R_t, F_s and F_t: for Q_p

Table 5.5 presents the recommended equation of the percentage of the toe over the total capacity (R_t), the reduction factors for shaft (F_s) and toe (F_t) for the determination of predicted capacity (Q_p) by Eq. 5.23. The factors are the values as derived from the 11 instrumented piles from Case (ii).

5.4.2.2 Values of F_b and S: for Q_a

Table 5.6 presents the bias factor and the standard deviation of the predicted capacity for the "uniform" and "non-uniform" sites to be used for determination of allowable capacity, Q_a, by Eq. 5.33.

5.4.2.3 Values of FS and β: for Safety Measures

Table 5.7 presents the recommended safety measures. The allowable capacity can be determined either from deterministic or reliability-based approaches. Their respective equivalent is presented in italics. The lower reliability index (**β**) of about 1.25 to 1.50 recommended for the "non-uniform" site are the ones *implied in practice*, which is *equivalent to deterministic factor of safety* (**FS**) of about 3.00 to 3.50. Although these **β** values appear low compared to other application, e.g., in structural engineering, they are practical values in this study. Recommending higher **β** values for the "non-uniform" site is impractical, i.e., if **β** values are recommended equal values for the "uniform" and "non-uniform" site, than the *equivalent* (or at the respective) **FS** values for the "non-uniform" site would be substantially higher than the maximum **FS** usually applied in practice (which is about 4.00 to 5.00).

Table 5.5: The Recommended Functions For R_t, F_s and F_t for Q_p.

FACTOR	CRITERIA	EQUATION
R_t (11 pts)	[2"]	1.1775 * Exp(-0.0314*L_e/d)
	[D]	1.0748 * Exp(-0.0327*L_e/d)
	[C]	1.0720 * Exp(-0.0288*L_e/d)
	[DC]	1.1042 * Exp(-0.0294*L_e/d)
F_s (11 pts)	[2"]	3.5251 * Exp(-0.0017*L_e/d)
	[D]	4.1626 * Exp(-0.0095*L_e/d)
	[C]	9.6570 * Exp(-0.0264*L_e/d)
	[DC]	5.8616 * Exp(-0.0162*L_e/d)
F_t (11 pts)	[2"]	11.596 * Exp(-0.0493*L_e/d)
	[D]	6.4852 * Exp(-0.0393*L_e/d)
	[C]	15.824 * Exp(-0.0554*L_e/d)
	[DC]	11.525 * Exp(-0.0497*L_e/d)

Table 5.6: The Recommended Values of F_b and S for the Determination of Q_a.

FACTOR	CRITERIA			
	[2"]	[D]	[C]	[DC]
F_b (CRP Test)	1.233	1.117	1.011	1.082
F_b (CL Test)	0.461	0.312	0.361	0.363
F_b (Unknown Load Test)	1.136	0.978	0.863	0.952
S ("Uniform" Site)	0.12	0.12	0.11	0.10
S ("Non-uniform" Site)	0.25	0.30	0.27	0.27

Table 5.7: The Recommended **β** and **FS** for The Deterministic and Reliability-Based Approaches.

	CRITERIA							
	[2"]		[D]		[C]		[DC]	
	Unif. Site	Non-Unif. Site	Unif. Site	Non-Unif. Site	Unif. Site	Non-Unif. Site	Unif. Site	Non-Unif. Site
[1] Minimum Required								
FS	1.79	1.79	2.46	2.46	2.42	2.42	2.25	2.25
[2] Minimum Required								
β	1.43	0.46	2.93	0.88	2.48	0.75	2.85	0.79
[3] **Deterministic Approach (Recommended)**								
FS *(β)*	**2.00** *(1.82)*	**3.00** *(1.36)*	**2.50** *(2.99)*	**3.00** *(1.17)*	**2.50** *(2.61)*	**3.00** *(1.10)*	**2.00** *(2.34)*	**3.00** *(1.25)*
[4] **Reliability-Based Approach (Recommended)**								
β *(FS)*	**2.00** *(2.10)*	**1.50** *(3.25)*	**3.00** *(2.51)*	**1.25** *(3.17)*	**2.50** *(2.43)*	**1.25** *(3.30)*	**3.00** *(2.33)*	**1.25** *(3.00)*

[1] average of (Q_m / Q_p) from Case (iii); to ensure Q_a would not be more than Q_m
[2] from Eq. 5.32; and CFS is from Eq. 5.34
[3] Recommended FS from Deterministic Approach; its Equivalent β is shown in italics
[4] Recommended β from Reliability-Based Approach; its Equivalent FS is shown in italics

5.4.2.4 Design Charts

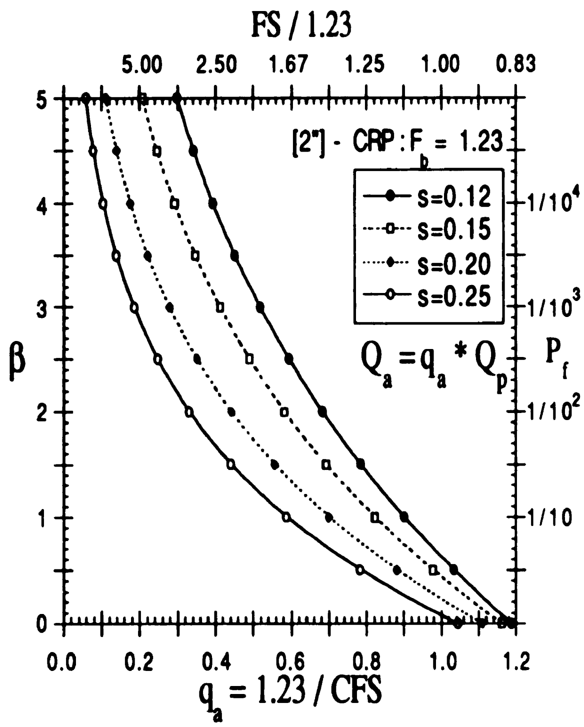
Figures 5.5, 5.6, and 5.7 presents plots of the related safety measures that could be easily used for rapid design checks. These figures use the F_b factors for constant rate of penetration test (CRP), the constant load type of loading test (CL), and the case for "unknown loading test" (NLT) respectively.

5.4.3 Allowable Capacity

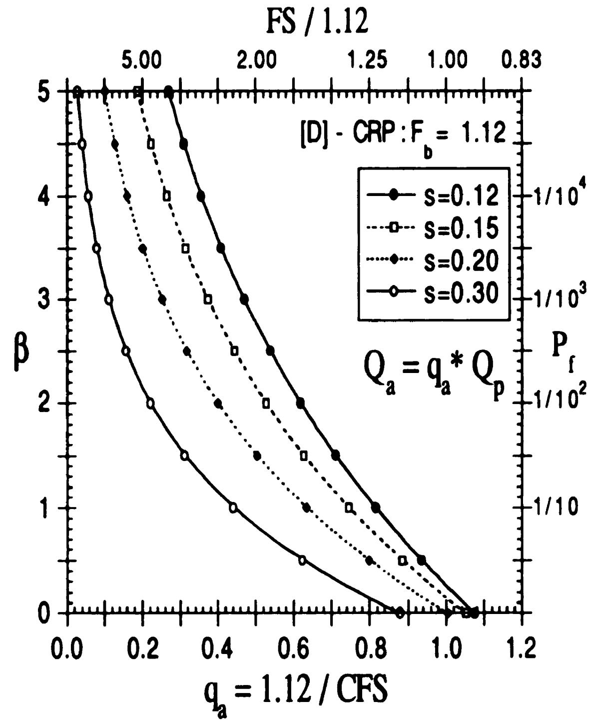
The determination of Q_p (and consequently the allowable capacity, Q_a) is computed by a depth integration process preferably set up on a spreadsheet for calculations for every foot of the embedded length of pile. The recommended procedures for the determination of Q_a are as presented in the following four steps:

STEP 1

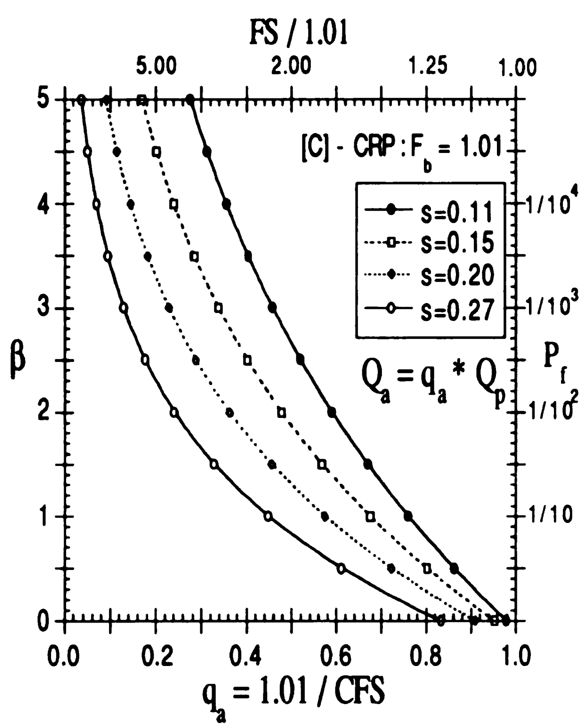
Select the recommended factor of safety, **FS**, or reliability index, β, from Table 5.7 for the respective method of determination (i.e., criteria [2"], [D], [C], and [DC]).



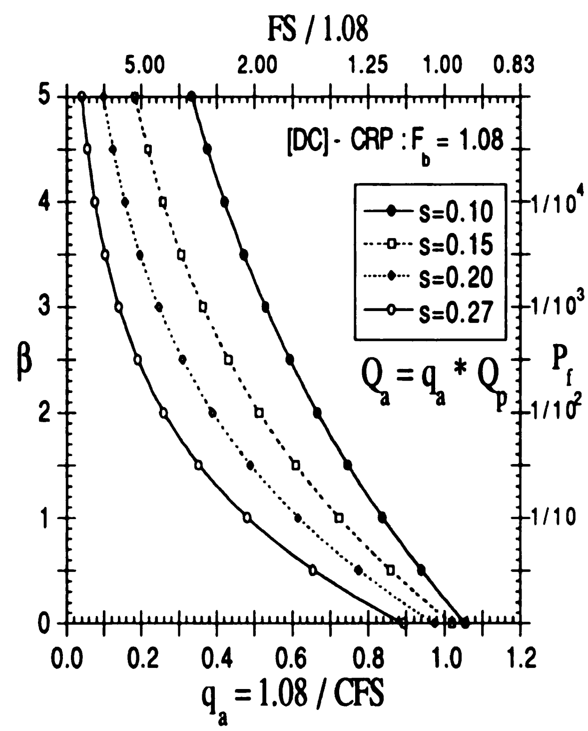
(a) 2" Movement



(b) Davisson's



(c) Chin's



(d) Ave Davisson's & Chin's

Fig. 5.5: Design Chart - For Constant Rate of Penetration Test Type (CRP).

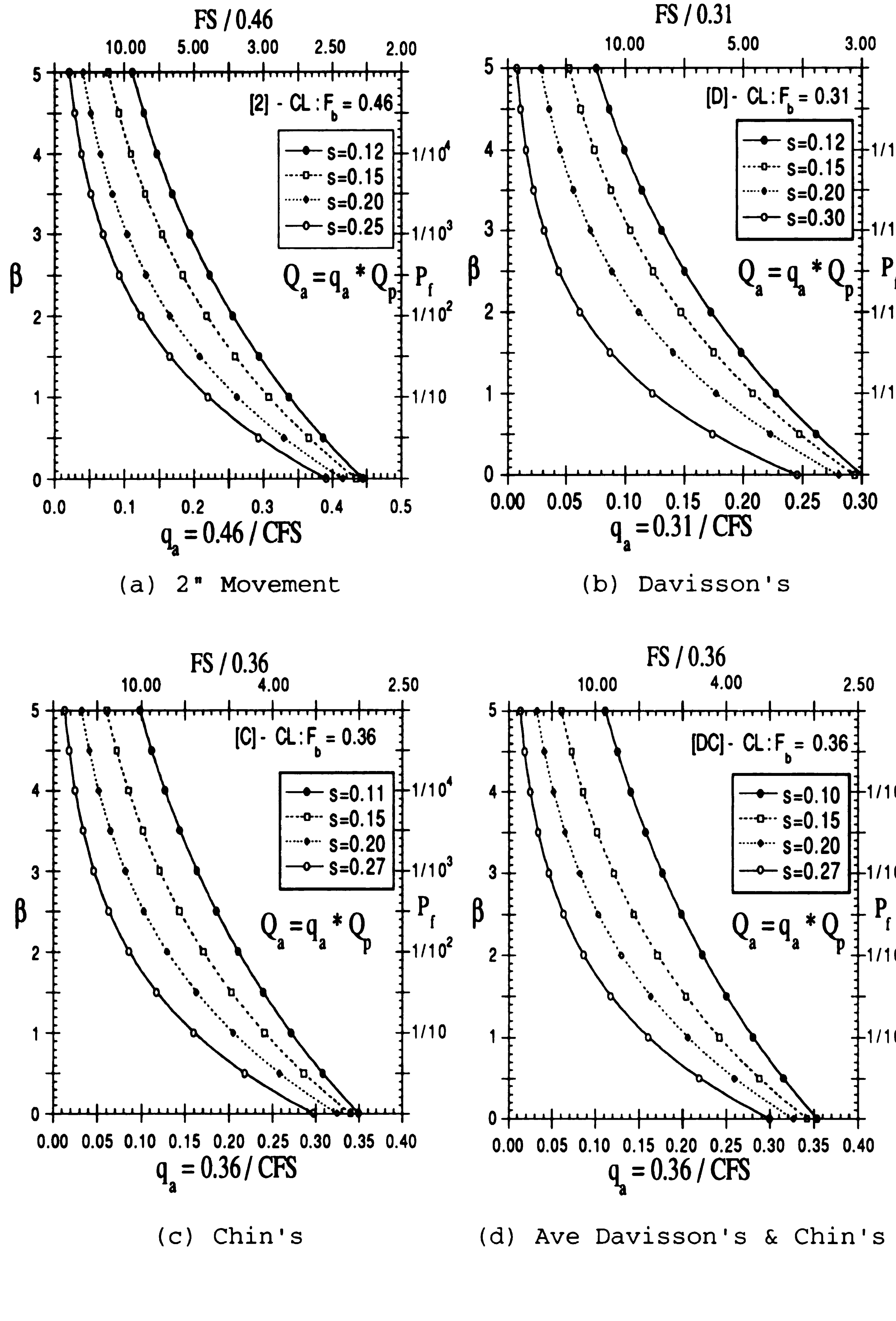
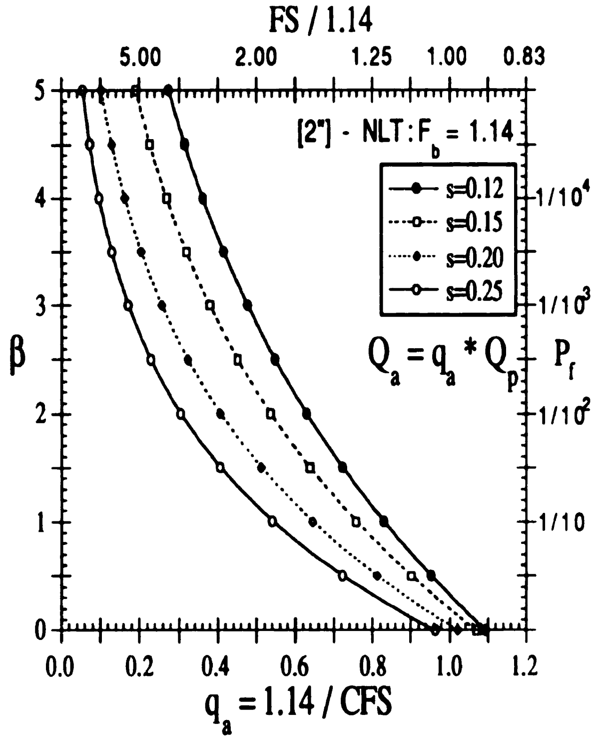
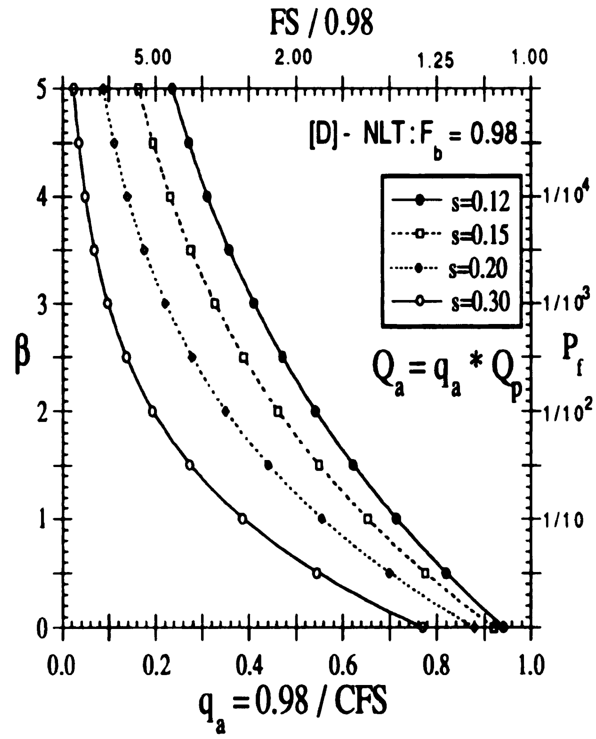


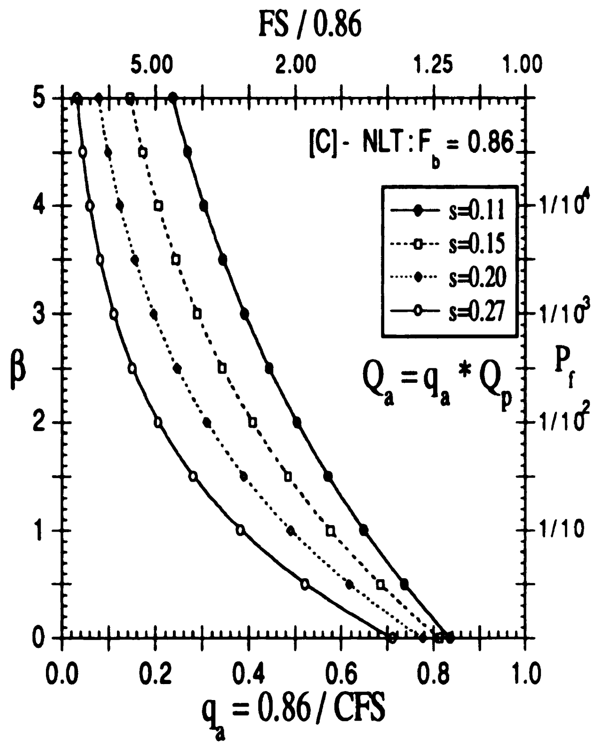
Fig. 5.6: Design Chart - For Constant Load Test Type (CL).



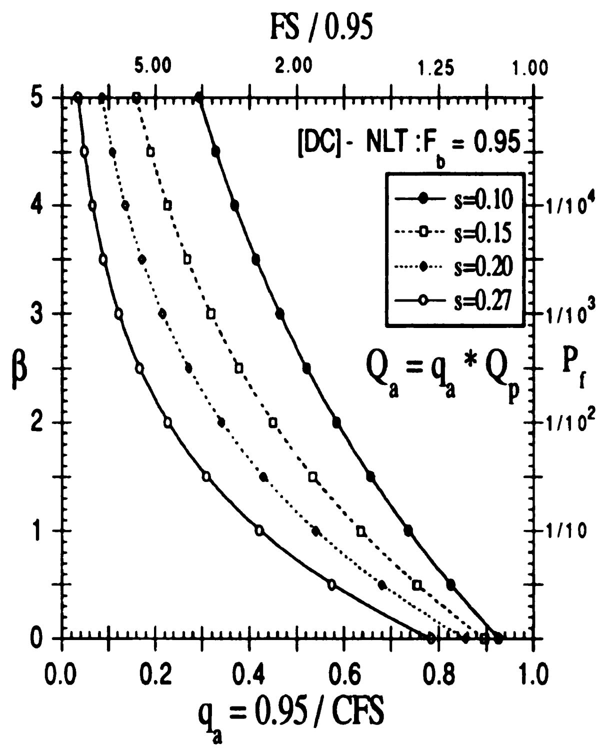
(a) 2" Movement



(b) Davisson's



(c) Chin's



(d) Ave Davisson's & Chin's

Fig. 5.7: Design Chart - For Unknown Loading Test (NLT).

STEP 2

Determine the perimeter surface area of pile shaft, A_s, and the toe bearing area, A_t, from pile properties. Following parameters are needed;

a. outside diameter, **d**

b. length of pile embedment, L_e.

STEP 3

Determine Q_p from Eq. 5.23, i.e.;

$$Q_p = \sum_{i=1}^{n} \left[\left(F_s\, p'\, K_s \tan\delta \right)_i (A_s)_i \right] + F_t \left(p_t'\, N_q^*\, A_t \right)$$

The following parameters are needed;

a. use correction factors F_s and F_t for shaft and toe respectively from the recommended equations in Table 5.5.

b. SPT N-values, interpolate at every foot of soil profile by the spreadsheet.

STEP 4

Determine Q_a from Eq. 5.33, i. e:

$$Q_a = \frac{F_b}{10^{\left(\beta_I s + \frac{\ln 10}{2} s^2\right)}} * Q_p$$

The recommended bias factor (F_b) and the variability of the predicted capacity of a particular site (S) can be found in Table 5.6. Alternatively, the charts in Figs. 5.5 through 5.7 can be used for the determination of Q_a.

5.5 Summary

This chapter has presented the development of an algorithm and calibration of the static pile formula for the determination of predicted axial pile capacity associated with four different interpretations of loading test. The algorithm uses SPT N-values from boring logs, and the N-values at every foot of the soil profile are interpolated. The predicted shaft and toe capacity are determined by adjusting the "modified" standard formula to bring them into conformance with the measured capacity from loading tests. The total capacity is again corrected by using a random variable bias factor reflecting the variability of the ratio of the measured to predicted capacity at the site. Consequently, with the knowledge of the type of loading test, the allowable capacity associated with the recommended safety measures (FS or β) can be determined. The algorithm in this study interrelates safety measures from the conventional deterministic design and reliability-based approach; thus at the recommended safety measures interpretation and design from either deterministic or reliability-based approaches is possible.

In short, with the combination of the concepts from first order uncertainty analysis and the deterministic approaches, the algorithm permits; (1) a systematic determination of the predicted and allowable pile capacity from SPT N-values, with much of the need for arbitrary personal judgment eliminated, and (2) a rational evaluation of factor of safety (**FS**) and reliability index (β) that could be used in design.

CHAPTER 6

TESTING OF THE ALGORITHM

6.1 Introduction

This chapter presents some illustrative examples as to how the developed algorithm can be used to predict axial pile capacity. Consequently, at the recommended safety measures, the allowable capacity can be determined. Data from 20 pile loading tests are used; 8 loading tests are available from 7 pipe piles, 9 loading tests are available from 8 H-piles, and 3 loading tests are from 3 concrete piles. These data have not been used to develop the appropriate parameters as described in Chapter 5. However, the criteria for choosing these piles are the same as the criteria used for the development of the algorithm in Chapter 5, i. e, (i) availability of the SPT N-values at the vicinity of the pile, (ii) piles are predominantly driven into cohesionless soil, (iii) piles are not end bearing (N-values of more than 100 at the toe, or piles with hard bearing layer near the toe are excluded).

6.2 Testing of the Algorithm

By using only 23 pile loading tests to calibrate the standard formula in Chapter 5, and with the limitations of data quality (e.g., unknown water table in Peck's collection which is used to derived the algorithm) and the methods of collection [recreation of the points for the load-movement curves from the selected references], it is perhaps inconclusive to claim that the method developed in this study is superior to other methods (e.g., Dennis and Olson, 1985; Coyle and Ungaro, 1991; Coyle and Ungaro, 1991; Ungaro, 1988 API, 1991). However, the predicted capacities determined from the algorithm are better than the unadjusted values from the "standard formula" as will be demonstrated in the subsequent sections. Twenty pile loading tests are felt enough to demonstrate the primary purpose of this study, which is to directly interrelate the safety measures from the deterministic and reliability design approaches in a consistent manner.

The algorithm is also extended for the determination of the allowable capacity for the H-piles. The equivalent circular pile diameter for all the H-pile is computed from the cross sectional area of the H-pile assuming the flanges are fully plugged. The equivalent pile diameter, $\mathbf{d_e}$, is therefore:

$$d_e = 2\sqrt{\frac{w\,h}{\pi}}$$

where; **w** = the width of the flange; and **h** = height of the web as given by the Tables of properties of steel sections. A similar computation was used for the square concrete piles; whereby **w** and **h** are the length of the sides.

The pile loading test data which are used to verify the developed algorithm are as presented in Table 6.1. The load-movement curves and the output from the algorithm for these 20 loading tests are presented in Appendix A and Appendix C respectively, except for Pile No.26 which is also presented in the next section as a typical example of the output. Other detail about Pile No.26 with regard to the site, the soil profile and the loading tests is presented in Section 6.2.3.

Table 6.1: Pile Loading Test Data - Illustrative Example.

PILE NO.	PILE	Embedded Length L_e (ft)	Diameter d (in)	Source
	Pipe Pile			
24	Northwestern (AT)	50	18.00	Finno (1989a)
25	Northwestern (SH)	50	18.00	Finno (1989a)
26	No.3, Kansas City	55	12.75	Williams (1960)
27	No.3A, Kansas City	55	14.00	Williams (1960)
28	No.7, Kansas City	55	14.00	Williams (1960)
29	No.7A, Kansas City	55	16.00	Williams (1960)
30	No.8, Aliquippa	88	12.75	AISI (1985)
31	No.28, Hamilton	95	12.75	AISI (1985)
	H - Pile			
32	Northwestern (AT)	50	H14x73	Finno (1989a)
33	Northwestern (SH)	50	H14x73	Finno (1989a)
34	No.7, Locks&Dam4	52	H14x73	Fruco & Associates (1964)
35	No.9, Locks&Dam4	54	H14x73	Fruco & Associates (1964)
36	No.4, Kansas City	55	12BP53	Williams (1960)
37	No.8, Kansas City	55	14BP73	Williams (1960)
38	No.2, Weirton	75	W14x102	AISI (1985)
39	No.17, S Lake City	59	H14x73	AISI (1985)
40	No.21, E. Chicago	102	H14x73	AISI (1985)
	Concrete Pile			
41	No.4, Locks&Dam4	40	16" Square	Fruco & Associates (1964)
42	No.5, Locks&Dam4	51	16" Square	Fruco & Associates (1964)
43	Locks&Dam25*	13	16" Octagonal	Conroy (1992)

6.2.1 Typical Example: Pile Test No.26

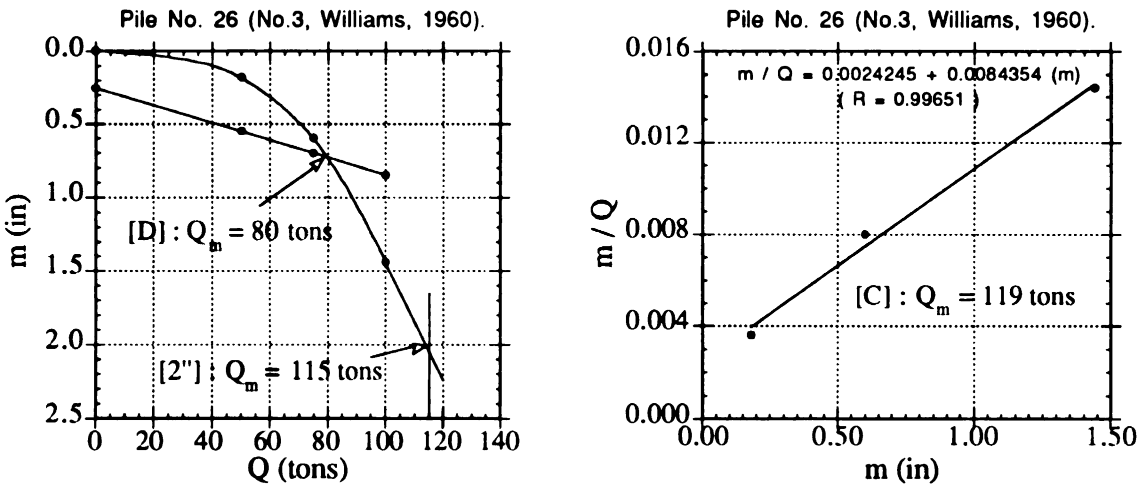
Pile No.26 is selected as a typical example of use of the developed algorithm. The determination of the measured capacities, Q_m, from the loading test as interpreted by the three test interpretation criteria (i.e., [2"], [D], and [C]) is as indicated in Fig. 6.1. A sample calculation of shaft and toe capacities as calculated from the "standard" formula, the calibrated shaft and toe capacities (for criteria [D] only) determined from the algorithm is as shown in Table 6.2. The detailed calculation (long output) for this pile can be found in Appendix C. The predicted capacities that are used for the determination of the allowable capacity in the design process are summarized in Table 6.3.

In Table 6.3, all calculations are set up on a spreadsheet program. The Table is the actual output from the spreadsheet. Since the focus is on the design and determining the allowable capacity, Q_a, the arrangement of numbers is not in a specific order.

The embedment length, L_e, of Pile No.26 is 55 ft. with an outside diameter (OD) of 12.75 in. The SPT N-values is available at several locations down the soil profile (bold print in column {2}, Table 6.2). Values are interpolated at every foot. Examples of other calculations are as listed in Table 6.2, and discussed below are the capacities in the design process as presented in Table 6.3.

6.2.1.1 Calculated Capacity: Standard Formula

In Table 6.3, for comparison with other capacities, the calculated shaft capacity, Q_{sc}, and calculated toe capacity, Q_{tc}, using Eqs. 5.2 and 5.3 are found to be 77 and 24 tons respectively. The sum of these two capacities is the total calculated capacity from the standard formula, Q_c, which is equal to 101 tons.



(a) Criteria [2"] & [D]   (b) Criteria [C]

Fig. 6.1: Measured Capacities - Steel Pipe, Pile No.26.

Table 6.2: Typical Spreadsheet Calculation, Pile No.26.

PIPE PILE		No.26	Assume:	
OD	*12.75*	(in)	γ_{sat}	γ_{wet}
t	*0.19*	(in)	(pcf)	(pcf)
$[A_s]_i$	3.34	(sq.ft)		
A_{steel}	7.42	(sq in)	130	120
A_{toe}	0.89	(sq.ft)		
L_e	55	(ft)		

{1} : Shown Depth of 20 ft only
{2} : Interpolated N-values;
in Bold is N-values fr Boring Log
{5} : 0.5*{18}
{6} : Eq. 5.4 (Liao et al, 1986)
{7} : Eq. 5.5 (Peck, 1974)
{8} : Table 2.4 (Potyondy,1961)
{9} : {7}*{8}
{10} : 1-Sin{9}
{11} : Table 5.3
{12} : {9}*{10}
{13} : {11}*{12}
{14} : {5}*{13}* $[A_s]_i$/2000
{15} : {5}*{12}* $[A_s]_i$/2000
{16} : Sum of {14}
{17} : Sum of {15}
{18} : {1}*{5}
{19} : Eq. 5.6 (Vesic, 1963)
{20} : {18}*{19}* A_{toe}/2000
{21} : Table 5.3
{22} : {20}*{21}

Depth (ft.)	SPT N No 26	γ_w	γ'	$[p'_{ov}]_i$ (psf)	N'	ϕ' (deg)	f_ϕ	δ (deg)	K_s	F_s [D]	β^* cal	β^* [D]	$[q_s]_i$ cal (Tons)	$[q_s]_i$ [D] (Ton)	Q_{sc} cal (Tons)	Q_{sp} [D]	p'_t toe (psf)	N_q^*	Q_{tc} cal (Tons)	F_t [D] (Ton)	Q_{tp} [D] (Tons)
{1}	{2}	{3}	{4}	{5}	{6}	{7}	{8}	{9}	{10}	{11}	{12}	{13}	{14}	{15}	{16}	{17}	{18}	{19}	{20}	{21}	{22}
0																					
1	**0**	0	120	60	0	27	0.76	20	.654	4.13	0.24	1.00	0.02	0.10	0.02	0.10	120	6	0.34	6.25	2.13
2	3	0	120	180	9	30	0.76	23	.617	4.09	0.26	1.05	0.08	0.31	0.10	0.41	240	9	0.96	6.02	5.80
3	7	0	120	300	16	32	0.76	24	.586	4.05	0.27	1.08	0.13	0.54	0.23	0.95	360	13	2.00	5.80	11.60
4	**10**	0	120	420	20	33	0.76	25	.572	4.02	0.27	1.09	0.19	0.76	0.42	1.72	480	15	3.14	5.59	17.56
5	11	0	120	540	20	33	0.76	25	.573	3.98	0.27	1.08	0.24	0.97	0.67	2.69	600	15	3.87	5.39	20.86
6	11	0	120	660	18	33	0.76	25	.579	3.94	0.27	1.06	0.30	1.17	0.96	3.85	720	14	4.32	5.19	22.42
7	12	0	120	780	19	33	0.76	25	.579	3.91	0.27	1.05	0.35	1.37	1.31	5.22	840	14	5.07	5.01	25.40
8	**12**	0	120	900	17	32	0.76	25	.583	3.87	0.27	1.04	0.40	1.56	1.72	6.78	960	13	5.51	4.82	26.61
9	10	0	120	1020	14	31	0.76	24	.597	3.84	0.26	1.01	0.45	1.72	2.16	8.50	1080	11	5.31	4.65	24.71
10	**8**	0	120	1140	10	30	0.76	23	.610	3.81	0.26	0.98	0.49	1.87	2.65	10.37	1200	10	5.16	4.48	23.11
11	8	0	120	1260	10	30	0.76	23	.612	3.77	0.26	0.97	0.54	2.04	3.20	12.41	1320	10	5.56	4.32	24.01
12	7	0	120	1380	8	30	0.76	22	.619	3.74	0.26	0.95	0.59	2.20	3.78	14.61	1440	9	5.68	4.16	23.65
13	7	0	120	1500	8	29	0.76	22	.620	3.70	0.25	0.94	0.64	2.36	4.42	16.97	1560	9	6.08	4.01	24.36
14	6	0	120	1620	7	29	0.76	22	.626	3.67	0.25	0.93	0.68	2.51	5.10	19.48	1680	8	6.19	3.86	23.91
15	6	0	120	1740	6	29	0.76	22	.626	3.64	0.25	0.92	0.73	2.67	5.84	22.15	1800	8	6.57	3.72	24.46
16	6	0	120	1860	6	29	0.76	22	.627	3.61	0.25	0.91	0.78	2.82	6.62	24.97	1920	8	6.95	3.59	24.94
17	5	0	120	1980	5	28	0.76	22	.632	3.57	0.25	0.89	0.83	2.95	7.45	27.92	2040	8	7.04	3.46	24.36
18	5	0	120	2100	5	28	0.76	22	.633	3.54	0.25	0.89	0.88	3.10	8.32	31.02	2160	8	7.41	3.33	24.71
19	4	0	120	2220	4	28	0.76	21	.637	3.51	0.25	0.87	0.92	3.23	9.24	34.25	2280	7	7.50	3.21	24.09
20	**4**	0	120	2340	4	28	0.76	21	.638	3.48	0.25	0.86	0.97	3.37	10.21	37.62	2400	7	7.86	3.10	24.34

Table 6.3: Typical Output of Allowable Capacity - Pile No.26.

Calculated (Formula)

Q_{sc}	Q_{tc}	Q_c	Q_{tc}/Q_c
77	24	101	0.24

Predicted (Algorithm) — RELIABILITY APPROACH, Uniform (Deterministic Equivalent)

Q_{sp}	Q_{tp}	Q_p		Site s	Provide β	Q_a	CFS	Equiv. FS
256	22	277	[2"]	0.12	2.00	**71**	1.81	**1.81**
231	20	251	[D]	0.12	3.00	**33**	2.38	**2.38**
311	22	333	[C]	0.11	2.50	**62**	1.95	**1.95**
261	21	282	[DC]	0.10	3.00	**50**	2.05	**2.05**

Measured (Loading Test) — Non-Uniform

Q_{sm}	Q_{tm}	(Q_t/Q)	CL F_b	Site s	Provide β	Q_a	CFS	Equiv. FS
88	27	0.23	0.461	0.25	1.50	**46**	2.80	**2.80**
62	18	0.22	0.312	0.30	1.25	**26**	3.01	**3.01**
90	29	0.24	0.361	0.27	1.25	**46**	2.64	**2.64**
76	24	0.24	0.363	0.27	1.25	**39**	2.64	**2.64**

DETERMINISTIC APPROACH, Uniform (Reliability-Based Equivalent)

	$Q_p^* F_b/Q_m$	Q_m	$Q_p^* F_b$	Site Provide FS	Q_a	Uniform Site s	CFS	Equiv. β
[2"]	1.11	*115*	**128**	2.00	**64**	0.12	2.00	**2.37**
[D]	0.98	*80*	**78**	2.50	**31**	0.12	2.50	**3.18**
[C]	1.01	*119*	**120**	2.50	**48**	0.11	2.50	**3.49**
[DC]	1.03	100	**102**	2.00	**51**	0.10	2.00	**2.90**

STEEL PIPE PILE

OD	*12.75*	(in)
t	*0.19*	(in)
[As]i	3.34	(sq.ft)
A steel	7.42	(sq in)
A toe	0.89	(sq.ft)
Le	55	(ft)

Non-Uniform

	Site Provide FS	Q_a	Non-Uniform Site s	CFS	Equiv. β
[2"]	3.00	**43**	0.25	3.00	**1.62**
[D]	3.00	**26**	0.30	3.00	**1.25**
[C]	3.00	**40**	0.27	3.00	**1.46**
[DC]	3.00	**34**	0.27	3.00	**1.46**

NOTE:
Input *Italics*
Output **Bold**

RELIABILITY APPROACH, Uniform (Deterministic Equivalent)

	NLT F_b	Site s	Provide β	Q_a	CFS	Equiv. FS
[2"]	1.136	0.12	2.00	**174**	1.81	**1.81**
[D]	0.978	0.12	3.00	**103**	2.38	**2.38**
[C]	0.863	0.11	2.50	**148**	1.95	**1.95**
[DC]	0.952	0.10	3.00	**131**	2.05	**2.05**

6.2.1.2 Measured Capacity: Loading Test

The measured pile capacities according to the [2"], [D], and [C] criteria are obtained from Fig. 6.1, which are equal to 115, 80, and 119 tons, respectively. The [DC] criterion is the average of the [D] and the [C] criteria, which is equal to 100 tons.

The proportion of the measured toe capacity, Q_{tm}, follows the equation of the toe capacity ratio, R_t, as found in Table 5.5. For example, for the [2"] criteria, $R_t=(Q_t/Q)=0.23$, which resulted in the measured toe (Q_{tm}) and the measured shaft capacities (Q_{sm}) of 27 and 88 tons respectively. A similar calculation procedure is done for the [D], [C] and [DC] criteria.

The Q_{sm} and Q_{tm} values are not used in the subsequent calculations (was used in Chapter 5 for the derivation of F_s and F_t). Therefore, Q_{sm} and Q_{tm} in Table 6.3 serve only as comparison with the respective values as determined from the standard formula and the algorithm.

6.2.1.3 Predicted Capacity: The Algorithm

The predicted capacity for the shaft (Q_{sp}) and toe (Q_{tp}) are determined from Eq. 5.23; and the total predicted capacity from the algorithm is found to be 277, 251, 333 and 282 for the [2"], [D], [C] and [DC] criteria respectively.

The factors for shaft (F_s) and toe (F_t) that are used with Eq. 5.23 are as found in Table 5.5.

Since the loading test for Pile No.26 is the constant load (CL) type, the capacities are corrected according to the recommended bias factor (F_b) of 0.461, 0.312, 0.361 and 0.363 for the [2"], [D], [C] and [DC] criteria respectively (see Table 5.6). Therefore, the corrected predicted capacities (i.e., Q_p*F_b) are equal to 128, 78, 120 and 102 for the [2"], [D], [C] and [DC] criteria respectively. These corrected predicted capacities are the ones used for the subsequent calculations for design (or the allowable capacity).

6.2.1.4 Design: Deterministic Approach

The capacity that is used for design is the allowable capacity, Q_a. For design according to the deterministic approach, the recommended factor of safety, **FS**, is taken from Table 5.7; which is equal to 2.00, 2.50, 2.50 and 2.00 for the [2"], [D], [C] and [DC] criteria respectively. By dividing the (Q_p*F_b) with **FS** (or Eq. 5.23), Q_a is found to be 64, 31, 48 and 51 tons for the [2"], [D], [C] and [DC] criteria respectively.

To determine the equivalent reliability index, β, the values of s from Table 5.6 are used. The recommended value of s for the "uniform" site is equal to 0.12, 0.12, 0.11 and 0.10 for the [2"], [D], [C] and [DC] criteria respectively. The respective central factor of safety (**CFS**) is determined by assuming that it is equal to the deterministic factor of safety (**FS**, see Eq. 5.32). Thus, the equivalent β is determined from Eq. 5.32; and is found to be 2.37, 3.18. 3.49 and 2.90 for the [2"], [D], [C] and [DC] criteria respectively.

Similarly, the procedure is repeated for the "non-uniform" site, but with a higher recommended **FS**.

6.2.1.5 Design: Reliability-Based Approach

The design from the reliability-based approach starts by selecting the recommended s values from Table 5.7; which are equal to 0.12, 0.12, 0.11 and 0.10 for the [2"], [D], [C] and [DC] criteria respectively. The provided reliability index (β) is taken from the recommended values as found in Table 5.7; which is equal to 2.00, 3.00, 2.50 and 3.00 for the [2"], [D], [C] and [DC] criteria respectively. The allowable capacity (Q_a) is than calculated from Eq. 5.33; which is equal to 71, 33, 62 and 50 tons for the [2"], [D], [C] and [DC] criteria respectively. Knowing the Q_a, the **CFS** and the equivalent **FS** is determined from Eq. 5.34; which is equal to 1.81, 2.38, 1.95 and 2.05 for the [2"], [D], [C] and [DC] criteria respectively.

Similarly, the procedure is repeated for the "non-uniform" site, but with a different set of s values (see also Table 5.6).

6.2.1.6 Allowable Capacity: Unknown Loading Test

Table 6.3 also presents the allowable capacity from the reliability-based approach by assuming that unknown type of loading test (NLT) will be done. That is, the allowable capacity is determined only from the SPT N-values, and no comparison with loading test would be made (see also Section 5.3).

The procedure to determine the allowable capacity for the NLT case is the same as in Section 6.2.1.5, except that the respective recommended F_b factors are used (see Table 5.6). At the recommended F_b (for NLT), for the "uniform" site, and at the recommended β, the allowable capacity is found to be 174, 103, 148 and 131 for the [2"], [D], [C] and [DC] criteria respectively.

Since the interpreted measured capacity from the loading test is itself dependent upon the type of loading test (i.e., CL or CRP type), the bias factor for the NLT condition should be used if no comparison is to be made with any loading test. If the loading test would be carried on the pile, the respective F_b should be used.

6.2.1.7 Design: Using The Chart

The allowable capacity as presented in Table 6.3 can be determined by using the Design Charts as presented in Section 5.3.4.4. Determination of the allowable capacity (Q_a) is as illustrated in Fig. 6.2 (for Pile No.26). The pile was tested according to the Constant Load (CL) type; and the curve in Fig. 6.2 is the Davisson's [D] criteria.

The predicted capacity (Q_p) as determined from the algorithm using the N-values on a spreadsheet program is found to be 251 tons (Table 6.3). The allowable capacity determined from the deterministic and reliability-based approaches are as described below.

(a) Reliability-Based Approach: For the [D] criteria, the recommended β from Table 5.7 is 3.00. At β=3.00, entering Fig. 6.2 from the scale on the left hand side, and deflecting downwards at s=0.12 (for the "uniform" site), the "unit allowable capacity" (q_a) is found to be equal to 0.13. The allowable capacity, ($Q_a=q_a*Q_p$), is then (0.13*251)=32.6 tons. Deflecting upwards from the s curve, or by taking the reciprocal of the q_a=0.13 (which is 7.69), the (**FS**/0.31) is equal to 7.69. Therefore, the equivalent **FS** is than equal to (7.69*0.31)=2.38. The approximate probability of failure (P_f) can be found on the scale on the right hand side of Fig. 6.2 (or simply $1/10^{-\beta}$).

(b) Deterministic Approach: Similar values as obtain from above can be obtained if the design is to be done from the deterministic approach. For the deterministic approach, the recommended **FS** in Table 5.6 is 2.50. Entering the scale at the top side of Fig. 6.2, i.e., at (2.50/0.31)=8.06 (or by taking the 8.06^{-1}, and enter the chart from the bottom scale), the q_a is found to be 0.124. The allowable capacity (Q_a) is then (0.124*251)=31.1 tons. Deflecting to the left at the "uniform" site (at s=0.12), the equivalent β=3.18. And deflecting to the right from the s curve, the P_f is found to be at about $2/10^{-3}$; or simply $1/10^{-\beta}$.

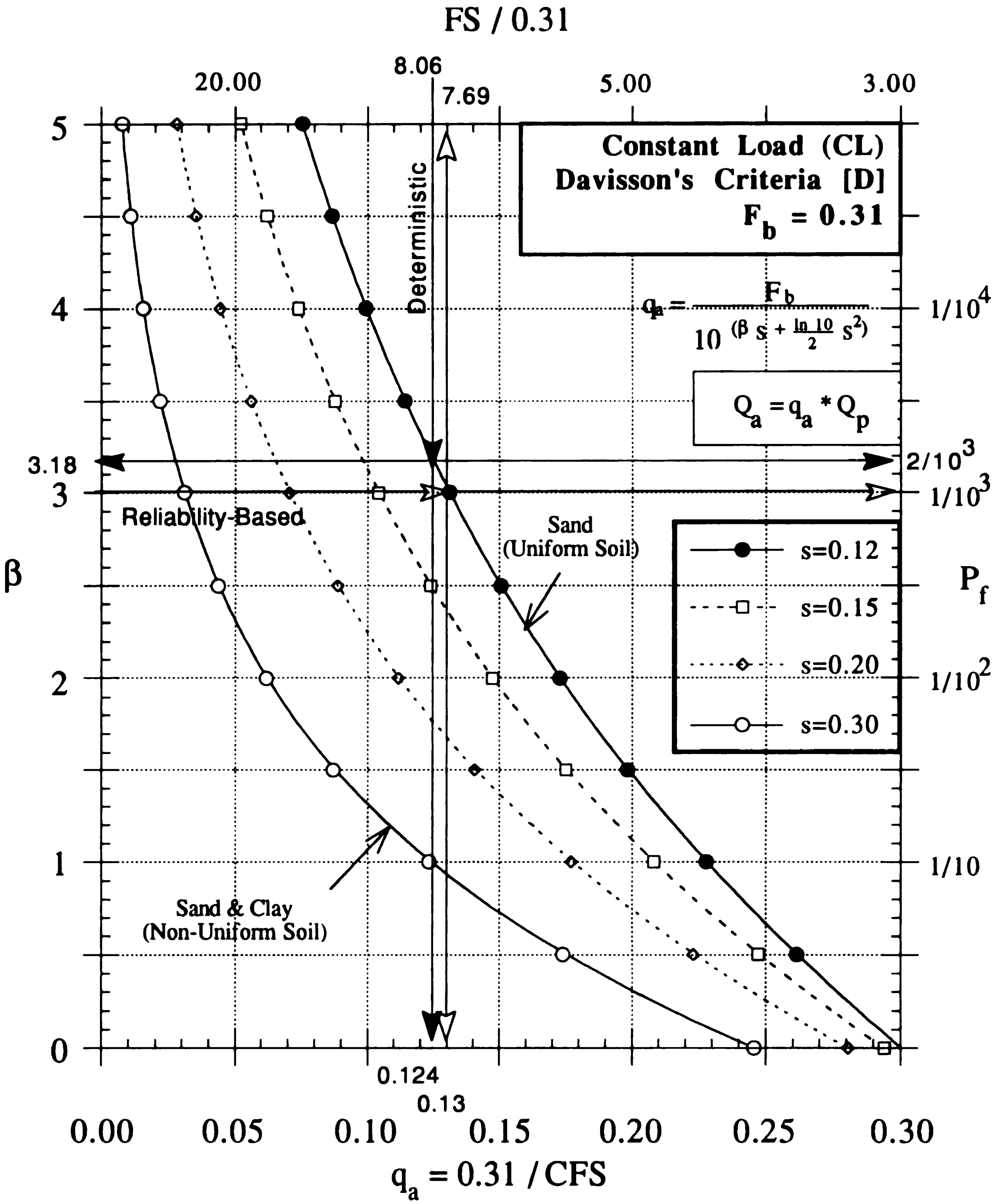


Fig. 6.2: Design By Chart - Davisson's Criteria [D]; Constant Load Test Type (CL).

6.2.2 Site at Northwestern University

Several axial loading tests were made in conjunction with the Foundation Engineering Congress at Northwestern University, Chicago, in June 1989 (Finno et al, 1989b). The purpose of that exercise was to evaluate the state of the profession's ability to predict pile response under axial load. Extensive laboratory and in-situ tests were made available to the 24 participants prior to the loading tests. The comparison of the predicted capacity as determined from the developed algorithm and the 24 participants will be presented in Section 6.3.

The stratigraphy at the test location is rather unusual; i.e., strong sand overlaying weak soft clay layer. The soil profile consists of 23 ft. of fine grained sand, then 45 ft. of soft to medium clay, then 12 ft of stiff clay and finally 10 ft. of hard silt. Beneath the silt, Niagaran dolomite bedrock is encountered. The water table is at about 15 ft. below the ground surface.

Four types of pile were tested (one HP14x73, one 18 in. diameter pipe pile, and two drilled piers); and each of the piles was tested three times at 2, 5, and 43 weeks after installation. Only data from the pipe pile and the H-pile are considered.

The pipe pile is 18 in outside diameter with the wall thickness of 0.375 in. The embedment length was 50 ft. The end of the pile was closed with a 19 in. outside diameter, 3/4 in thickness boot plate. The H-pile was also embedded at 50 ft.

All the piles were driven by Vulcan 06 hammer. To assist the driving, a 12 in. diameter **hole was preaugered** to a depth of 23 ft. at the location of the pipe pile (Finno et al, 1989a). The loading tests were performed in general conformance with ASTM standard D-1143-81. "Failure" (i.e., end to testing) was noted when the load could not be held constant (thus CL type of loading test) unless the hydraulic jack was continuously pumped.

For the Standard Penetration Test (SPT), there are two sets of N-values available. One set was obtained using an automatic trip hammer and the other set was obtained using the safety hammer. Therefore, two independent predictions of the capacities can be made for a single pile (listed in Table 6.1 as Pile No.24 and No.25 for the pipe pile, and Pile No.32 and No.33 for the H-pile).

The load-movement curve for the interpretation of ultimate capacities according to criteria [2"], [D], and [C] is presented in figures in Appendix A. The summary of capacities from the prediction and other related parameters are presented in Appendix C. The results will further be discussed in Section 6.3.

6.2.3 Site at Kansas City

Six piles were considered from Williams (1960), i.e., Pile No.26, No.27, No.28 and No.29 for pipe piles and Pile No.32 and No.33 for H-pile.

This project was a joint effort by the Kansas and Missouri Highway Departments for the bridge connecting Kansas City, Kansas and Kansas City, Missouri. The central portion of the intercity viaduct is located in the flood valleys of the Kansas River and the Missouri River not far from the junction of these two rivers.

Bedrock was approximately 85 to 65 ft. below ground level. Soil borings were made according to the proposed method for penetration tests and split spoon sampling of soils, ASTM Committee D-18 of 1958. The borings showed that the soil classification was fairly uniform for the entire structure. Fine sand is at the top 10 or 15 ft, the next 25 ft. consists of silty sand and sandy silt, and the balance consists of fine to coarse sand and gravel. The bedrock was shale and limestone. The ground water table was indicated at about 28 to 30 ft. below the ground surface. However, the distance of SPT borings and the actual locations of the respective piles were not available from Williams (1960).

The diameters of the pipe piles were 12, 14, 14, and 16 in. for Pile No.26, No.27, No.28 and No.29 respectively. The pipe thickness was 3/16 in; and were closed at the toe with flat metal plates. The H-pile, Pile No.32 and No.33 were of type 12BP53 and 14BP73 respectively. The embedment length of all of the piles was 55 ft. The piles were all driven with a modified Vulcan No. 1 Steam Hammer.

The loading tests were carried out not earlier than 48 hours after the driving of the piles. The load was maintained at all times during the test by a constant attention to load gage readings and jacking application (thus CL type of loading test). The first application of load was approximately 50 tons for Pile No.26, No.27, and 65 tons for Pile No.28 and No.29. The load increment after the first application is 25 tons per increment, applied not earlier than 1 hour after all measurable movement of the initial loading had ceased. The least movement considered measurable was 0.012 in. Failure (or termination of loading test) was defined when the rate of gross movement exceeded 0.03 in/ton for the last movement of load applied.

The respective measured capacities as well as the output from the spreadsheet according to interpretation criteria used in this study can be found in Appendix A and C. The results will further be discussed in Section 6.3.

6.2.4 Site at Locks & Dam No.4

Six pipe piles at the site of Locks and Dam No.4 (Fruco and Associates, 1964) have been used earlier in Chapter 5 to derive the appropriate factors in Section 5.3.2. The soil profile and the related pile testing procedures can be found in Section 5.2.2.

There are two H-piles and two concrete piles that conform to the selection criteria (Section 6.2). The two H-piles are of type 14BP73 which is represented by Pile No.34 and No.35. The two square 16 in. concrete piles is as represented by Pile No.41 and No.42.

The respective measured capacities as well as the output from the spreadsheet according to interpretation criteria used in this study can be found in Appendix A and C. The results will further be discussed in Section 6.3.

6.2.5 AISI Collection

Two steel pipe piles (Pile No.30 and No.31) that satisfy the selection criteria as presented in Section 6.2 are considered from the collection of AISI (1985). The H-piles are as represented by Pile No.38, No.39, and No.40.

Only the basic information about the piles and pile driving data were available; such as location, date of test, hammer type, pile type, embeddment length, and the load-movement curves. No detailed information about the soil profile is available; except for SPT N-values and the location of the water table.

Pile No.30 is from the site in Aliquippa, Pensylvania. The pile was a 12.75 in. outside diameter with 3/8 in. wall thickness; the toe closed with a 1-1/4 in. flat boot plate. The embedment length of the pile is 88 ft; and filled with 5,000 psi concrete. With increasing depth from the ground surface, the soil consists of 40 ft. of fill slag and alluvium, 10 ft. of soft organic silt, 40 ft. of dense gravel and beneath this the medium hard sandstone. The water table was about 45 ft. below the ground surface. The N-values are available to about 40 ft. below the ground surface. The loading test was performed in accordance to ASTM D1143-61T. The load was supplied using a hydraulic jack against a load platform, and Raymond hammer. Termination of each loading occurred when the settlement was approximately 0.001 in/hr.

Pile No.31 is from the site in Hamilton, Ontario. The pile was a 12.75 in. outside diameter with 3/8 in. wall thickness; toe closed with concrete core. The embedment length of the pile is 95 ft. With increasing depth from the ground surface, the soil consists of 30 ft. of fill sand and gravel, 30 ft. of fine to medium loose sand, 15 ft. of very stiff clayey silt, 25 ft. of very dense fine to medium gravel and beneath this is the weathered queenstone shale. There is no indication of water table. The loading test was performed in accordance to ASTM D1143-81. The load was supplied from two 400 ton hydraulic jacks against a dead load, and Demag D30-23 hammer. There is no indication of the type of loading test performed.

From the characteristics of the load-movement curve, it appears that both pipe piles were tested according to CRP type of loading test.

H-pile No.38, No.39, and No.40 did not quite satisfy the selection criteria (the N-values and the soil profile as mentioned in Section 6.2). Nevertheless, they were tried upon to observe any dispersion of the predicted capacity. From the characteristics of the load-movement curve as presented by AISI (1988), it appears that the loading test for these piles is of CL type.

Pile No. 38 is from the site in Weirton, West Virginia. The pile type is W14 X 102, A36 steel, with an embedment length of 75 ft. With increasing depth from the ground surface, the soil consists of about 40 ft. of medium dense fine sand with some silt, the next 40 ft. is medium dense fine sand with some gravel. At depth of 80 ft. and below, the *bearing layer* consists of hard silting shale and very hard sandy siltstone. The water table is about 40 ft. below the ground surface. The loading test was in accordance to ASTM D1143-61T. The loading is done by hydraulic jacking against load frame anchored to nearby reaction piles. The hammer type is Vulcan 80C weighing 8000 lbs. The test was terminated at 450 tons because of oil leakage from the hydraulic jack.

Pile No. 39 is the H-pile of type HP14 X 73, A36 steel driven at Salt Lake City, Utah. The toe of the pile is at 78 ft., but the head of the pile is embedment 19 ft. below the ground surface, resulting in the embedment length of the pile of 59 ft. The soil consists of silt and clay with traces of sand up to the depth of about 50 ft. below the ground surface; 30 ft. below that is sandy gravel with N-values of more than 100. Therefore, the pile is predominantly embedded *in clay* which also function as an *end bearing pile*. The loading device is similar to Data No.15, but the pile is driven by Delmag D-22 hammer.

Data No.17 is the H-pile of type H12x53, A36 steel. The embedment length of the pile is 102 ft. Top 50 ft. of the soil profile is fill slag, and the next 50 ft. is silty clay with sand (thus, predominantly in clay). The pile is driven with a Vulcan 140C hammer.

The respective measured capacities as well as the output from the spreadsheet according to interpretation criteria used in this study can be found in Appendix A and C. The results will further be discussed in Section 6.3.

6.2.6 Site at Locks and Dam No.25

One 16 in. octagonal concrete pile is used to test the algorithm (Conroy, 1992). The pile is supporting a bridge structure. The toe of the pile was embedded to a depth of 54 ft. Due to erosion and scouring of the soil at the site, the present ground level is at an elevation of 41 ft., leaving an embedment length of only 13 ft. **No load test** was done, but the SPT N-values is available from the nearby boring. The prediction of capacities from the algorithm using only the N-values can be found in Appendix C. The analysis of the capacities is presented in Section 6.3

6.3 Discussion of Results

Comparison of the measured capacities (Q_m) and the predicted capacities (Q_p*F_b) is shown in Appendix D. It is found that for all criteria there is no significant different between Q_m and Q_p*F_b. However, the student t-test was conducted for pipe pile only.

6.3.1 Predicted, Allowable & Measured Capacities

A summary of the outputs from the algorithm is presented in Tables 6.4 for the respective interpretation criteria respectively (Note: from Section 5.3.1, the [DC] criterion is **not the average** of the [D] and [C] outputs, but rather the parameters, i.e., R_t, F_S, and F_t are derived from the average of measured capacities (Q_m) as determined from the Davisson's and Chin's criteria). The predicted capacities (Q_p*F_b) as found in Table 6.4 are plotted against the measured capacities (Q_m) as shown in Fig. 6.3.

The mean (μ) of the predicted capacity from the algorithm divided by the measured capacity (Q_p*F_b/Q_m), is found to be 1.33, 0.98, 1.04 and 1.03 for the [2"], [D], [C] and [DC] criteria respectively for the pipe pile. The standard deviation (σ) is 0.21, 0.08, 0.21 and 0.13 for the [2"], [D], [C] and [DC] criteria respectively. From Table 6.4, the interpretation criteria of [2"] tends to overestimate the pile quite substantially (about 33%). This is probably due to an overestimate of the shaft capacity (especially for longer piles), since the factor F_S is not decreasing at an increasing depth but rather flat (see Fig. 5.4(c)). For the other criteria, the predicted capacities are found to be good, i.e., (Q_p*F_b/Q_m) of about 1.00.

Overall, the [D] criteria showed the best prediction of capacities when compared to measured capacities, μ = 0.98 and σ = 0.08; followed by [DC], [C] and [2"]. A larger bias (μ) and a larger scatter (σ) is obtained when all the 20 data points representing pipe pile, H-pile and concrete piles are used (see Table 6.4).

From the determined (Q_p*F_b), the allowable capacity (Q_a) can be determined by using Eqs. 5.3.3 and 5.3.4 for the β and **FS** respectively.

Table 6.4 also presents the Q_a assuming that the type of loading test is unknown (NLT). These Q_a (and other Q_a as determined from the algorithm) are the long term capacities (see assumptions in Section 5.3.1). The β and **s** for the "uniform" site are recommended if the pile is predominantly in sand (say, more than 50 % of embedded length), or else the parameters from the "non-uniform" site are recommended (see also the recommended values in Section 5.3.4).

The Q_a for the unknown loading test (NLT) condition is found to be at higher values since the F_b for the NLT condition is recommended higher (uncertainty whether the loading test would be done according to the CL or CRP type). The justification for higher recommended F_b factors has been discussed in Section 5.3.3 and 5.3.4. If however, the Q_a is to be compared with "future loading test," the F_b from the anticipated testing methods should be used (see also F_b factors for CL and CRP in Table 5.6).

The choice of F_b, loading test methods (CL or CRP) and the interpretation criteria ([2"], [D], [C], or [DC]) for the determination of (Q_p*F_b) and Q_m can be illustrated by examining Pile No.43 in Table C43 in Appendix C.

Table B20 presents the output from the spreadsheet program. Say for example the comparison of allowable capacity is to be made for CL type of loading test, and Q_m to be interpreted by the criteria [D], then (Q_p*F_b)=66 tons. If the deterministic approach is used (at **FS**=2.50) the allowable capacity (Q_a) is 26 tons. Similarly, if the reliability approach is used (β=3.00 and s=0.12) the allowable capacity (Q_a) is 28 tons. From the analysis done by Conroy (1992) for this pile, the equivalent (Q_p*F_b)=47 tons, with a large standard deviation of 24 tons. This indicates that the possible range of (Q_p*F_b) from Conroy's analysis could be in the wide range of (-25 to 119 tons) from negative 3 and positive 3 standard deviations from the expected value. Similarly, the equivalent Q_a is 31 tons with a standard deviation of 4.9 tons, which indicates that Q_a from Conroy's analysis could lie in between 16 to 46 tons. Therefore, the values from the algorithm from [D] criteria "are within" Conroy's values.

Looking at this data from the other perspective, it may suggest that the "real probable" Q_a could in fact be higher. Considering that the Davisson's interpretation of the loading test (Q_m) could be the lower limit and the Chin's interpretation could possibly be the upper limit, perhaps the values from the [DC] criteria should be used. At the recommended safety measures for the [DC] criteria, the Q_a is found to be 61 and 59 tons from the deterministic and reliability-based design approaches respectively (which is also within the Conroy's range).

6.3.2 Deterministic Vs. Reliability-Based Design

Using the recommended safety measures (**FS** or β, in Table 5.7) the allowable capacity as determined by the reliability-based approach could be slightly lower than the allowable capacity from the deterministic approach. This is because at the recommended **FS**, "the equivalent" β values are not recommended. The equivalent β values at the corresponding FS values are rounded up to one decimal place for convenience.

For example, at the recommended **FS** of 2.00, the "equivalent" or the **β** value that corresponds to **FS**=2.0 is actually 1.83 (the [2"] criteria for the "uniform" site in Table 5.7), but for convenience if the design is from the reliability approach "the recommended **β**" value is rounded up to 2.00. The reverse is true when the recommended **β** is actually lower than "the equivalent" **FS**, e.g., criteria [C] for the "unknown" site; whereby the recommended **β** is 2.50 instead of recommending "the equivalent" value of 2.61 at the respective recommended **FS** of 2.50. The small difference can be considered insignificant considering that both approaches are independent of each other.

Table 6.4: Summary of Q_a and Safety Measures From The Algorithm.

(a) Criteria [2"].

PILE NO.	PILE TYPE		[2"] CRITERIA			ALLOWABLE CAPACITY[3]		NLT[4] Uniform Site	NLT[4] Non-Unif. Site
	Pipe Pile	L_e (ft)	Q_m (tons)	$Q_p^* F_b$ (tons)	$Q_p^* F_b$ / Q_m	[1]Q_a (tons)	[2]Q_a (tons)	Q_a (tons)	Q_a (tons)
24	Northwestern (AT)	50	100	122	1.22	61	68	-	108
25	Northwestern (SH)	50	100	124	1.24	62	69	-	109
26	No.3, Kansas City	55	115	127	1.11	64	71	174	-
27	No.3A, Kansas City	55	112	144	1.29	72	80	197	-
28	No.7, Kansas City	55	130	148	1.14	74	82	203	-
29	No.7A, Kansas City	55	140	181	1.29	91	100	247	-
30	No.8, Aliquippa	88	480	760	1.58	380	421	388	-
31	No.28, Hamilton	95	580	923	1.59	461	511	471	-
	H - Pile								
32	Northwestern (AT)	50	103	102	0.99	51	56	-	90
33	Northwestern (SH)	50	103	104	1.01	52	57	-	91
34	No.7, Locks&Dam[4]	52	250	117	0.47	58	65	-	103
35	No.9, Locks&Dam[4]	54	260	128	0.49	64	71	-	113
36	No.4, Kansas City	55	124	136	1.10	68	75	-	120
37	No.8, Kansas City	55	122	179	1.47	90	99	-	158
38	No.2, Weirton	75	550	317	0.58	159	176	-	208
39	No.17, S Lake City	59	375	791	2.11	396	438	-	697
40	No.21, E. Chicago	102	280	421	1.50	211	233	-	371
	Concrete Pile								
41	No.4, Locks&Dam[4]	40	225	110	0.49	55	61	-	96
42	No.5, Locks&Dam[4]	51	295	148	0.50	74	82	-	131
43	Locks&Dam25	13	NA	153	NA	77	85	-	135

		Pipe Pile[5]	H-Pile Only[6]	All Data
($Q_p^* F_b$/ Q_m)	μ	1.33	0.92	1.19
	σ	0.21	0.38	0.45

[1]Q_a from DETERMINISTIC approach for the "Uniform" Site at FS=2.00; equivalent β=2.37
[2]Q_a from RELIABILITY-BASED approach for the "Uniform" Site at β=2.00; equivalent FS=1.81
[3]Allowable capacity (Q_a) as compared to Measured Capacity (Q_m)
[4]Allowable Capacity from SPT only (unknown loading test, NLT)
[5]Excluding Pile No.24 & No.25 - hole preaugered before pile is driven
[6]Excluding Pile No.38, No. 39, and No.40 (Pile No.38 & Pile No.39 - Toe bearing; Pile No.39 & Pile No.40 - pile is predominantly embedded in clay)

Table 6.4: Continued

(b) Criteria [D].

PILE NO.	PILE TYPE		[D] CRITERIA			ALLOWABLE CAPACITY[3]		NLT[4] Uniform Site	NLT[4] Non-Unif. Site
	Pipe Pile	L_e (ft)	Q_m (tons)	$Q_p^* F_b$ (tons)	$Q_p^* F_b$ / Q_m	^1Q_a (tons)	^2Q_a (tons)	Q_a (tons)	Q_a (tons)
24	Northwestern (AT)	50	60	79	1.32	32	33	-	83
25	Northwestern (SH)	50	60	81	1.35	32	34	-	84
26	No.3, Kansas City	55	80	78	0.98	31	33	103	-
27	No.3A, Kansas City	55	95	90	0.95	36	38	119	-
28	No.7, Kansas City	55	110	93	0.85	37	39	122	-
29	No.7A, Kansas City	55	115	115	1.00	46	48	151	-
30	No.8, Aliquippa	88	500	551	1.10	221	232	203	-
31	No.28, Hamilton	95	640	638	1.00	255	268	235	-
	H - Pile								
32	Northwestern (AT)	50	90	66	0.73	26	28	-	69
33	Northwestern (SH)	50	90	67	0.74	27	28	-	70
34	No.7, Locks&Dam[4]	52	220	73	0.33	29	31	-	76
35	No.9, Locks&Dam[4]	54	240	80	0.33	32	34	-	83
36	No.4, Kansas City	55	90	85	0.94	34	35	-	80
37	No.8, Kansas City	55	102	113	1.11	45	48	-	118
38	No.2, Weirton	75	440	194	0.44	78	82	-	203
39	No.17, S Lake City	59	340	510	1.50	204	214	-	531
40	No.21, E. Chicago	102	260	217	0.83	87	91	-	226
	Concrete Pile								
41	No.4, Locks&Dam[4]	40	190	66	0.35	26	27	-	68
42	No.5, Locks&Dam[4]	51	245	91	0.37	37	38	-	95
43	Locks&Dam25	13	NA	66	NA	26	28	-	69

		Pipe Pile[5]	H-Pile Only[6]	All Data
($Q_p^* F_b / Q_m$)	μ	0.98	0.70	0.91
	σ	0.08	0.32	0.31

[1]Q_a from DETERMINISTIC approach for the "Uniform" Site at FS=2.50; equivalent β=3.18

[2]Q_a from RELIABILITY-BASED approach for the "Uniform" Site at β=3.00; equivalent FS=2.38

[3]Allowable capacity (Q_a) as compared to Measured Capacity (Q_m)

[4]Allowable Capacity from SPT only (unknown loading test, NLT)

[5]Excluding Pile No.24 & No.25 - hole preaugered before pile is driven

[6]Excluding Pile No.38, No. 39, and No.40 (Pile No.38 & Pile No.39 - Toe bearing; Pile No.39 & Pile No.40 - pile is predominantly embedded in clay)

Table 6.4: Continued

(c) Criteria [C].

PILE NO.	PILE TYPE		[C] CRITERIA			ALLOWABLE CAPACITY[3]		NLT[4] Uniform Site	NLT[4] Non-Unif. Site
	Pipe Pile	L_e (ft)	Q_m (tons)	$Q_p^* F_b$ (tons)	$Q_p^* F_b / Q_m$	^1Q_a (tons)	^2Q_a (tons)	Q_a (tons)	Q_a (tons)
24	Northwestern (AT)	50	119	149	1.25	60	76	-	135
25	Northwestern (SH)	50	119	152	1.28	61	78	-	138
26	No.3, Kansas City	55	119	120	1.01	48	62	148	-
27	No.3A, Kansas City	55	121	144	1.19	58	74	177	-
28	No.7, Kansas City	55	137	148	1.08	59	76	182	-
29	No.7A, Kansas City	55	150	193	1.29	77	99	238	-
30	No.8, Aliquippa	88	495	508	1.03	203	261	223	-
31	No.28, Hamilton	95	827	550	0.67	220	283	241	-
	H - Pile								
32	Northwestern (AT)	50	108	119	1.10	47	61	-	109
33	Northwestern (SH)	50	108	121	1.12	49	62	-	110
34	No.7, Locks&Dam[4]	52	297	124	0.42	49	64	-	112
35	No.9, Locks&Dam[4]	54	316	133	0.42	53	68	-	120
36	No.4, Kansas City	55	143	133	0.93	53	68	-	120
37	No.8, Kansas City	55	132	191	1.45	76	98	-	173
38	No.2, Weirton	75	986	285	0.29	114	147	-	258
39	No.17, S Lake City	59	482	623	1.29	249	320	-	564
40	No.21, E. Chicago	102	339	243	0.72	97	125	-	220
	Concrete Pile								
41	No.4, Locks&Dam[4]	40	228	127	0.56	51	65	-	115
42	No.5, Locks&Dam[4]	51	321	163	0.51	65	84	-	148
43	Locks&Dam25	13	NA	159	NA	64	82	-	144
			Pipe Pile[5]		H-Pile Only[6]		All Data		
	$(Q_p^* F_b / Q_m)$	μ	1.04		0.91		0.97		
		σ	0.21		0.41		0.35		

[1] Q_a from DETERMINISTIC approach for the "Uniform" Site at FS=2.50; equivalent β=3.49

[2] Q_a from RELIABILITY-BASED approach for the "Uniform" Site at β=2.50; equivalent FS=1.95

[3] Allowable capacity (Q_a) as compared to Measured Capacity (Q_m)

[4] Allowable Capacity from SPT only (unknown loading test, NLT)

[5] Excluding Pile No.24 & No.25 - hole preaugered before pile is driven

[6] Excluding Pile No.38, No. 39, and No.40 (Pile No.38 & Pile No.39 - Toe bearing; Pile No.39 & Pile No.40 - pile is predominantly embedded in clay)

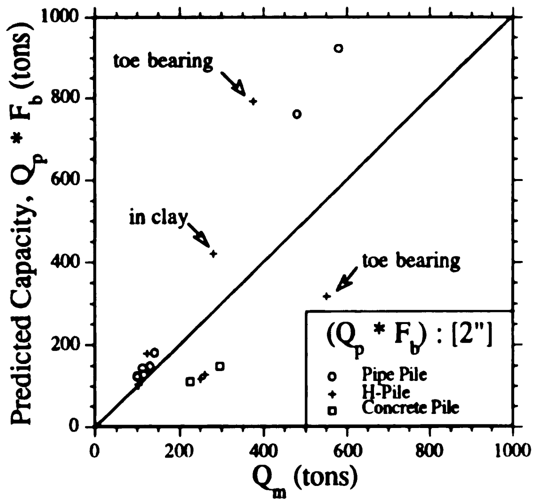
Table 6.4: Continued

(d) Criteria [DC].

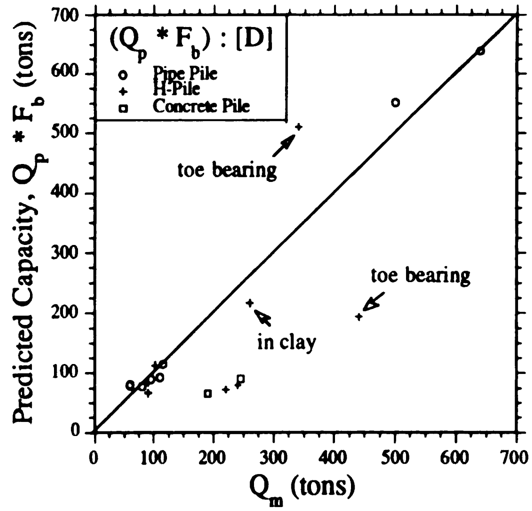
PILE NO.	PILE TYPE		[DC] CRITERIA			ALLOWABLE CAPACITY[3]		NLT[4] Uniform Site	NLT[4] Non-Unif. Site
	Pipe Pile	L_e (ft)	Q_m (tons)	$Q_p^* F_b$ (tons)	$Q_p^* F_b / Q_m$	[1]Q_a (tons)	[2]Q_a (tons)	Q_a (tons)	Q_a (tons)
24	Northwestern (AT)	50	90	114	1.27	57	56	-	113
25	Northwestern (SH)	50	90	116	1.29	58	57	-	115
26	No.3, Kansas City	55	100	102	1.02	51	50	131	-
27	No.3A, Kansas City	55	108	120	1.11	60	58	153	-
28	No.7, Kansas City	55	124	123	0.99	62	60	158	-
29	No.7A, Kansas City	55	133	157	1.18	78	76	201	-
30	No.8, Aliquippa	88	498	531	1.07	266	259	228	-
31	No.28, Hamilton	95	734	598	0.81	299	292	257	-
	H - Pile								
32	Northwestern (AT)	50	99	93	0.94	46	45	-	92
33	Northwestern (SH)	50	99	95	0.96	47	46	-	94
34	No.7, Locks&Dam[4]	52	259	101	0.39	50	49	-	100
35	No.9, Locks&Dam[4]	54	278	109	0.39	55	53	-	109
36	No.4, Kansas City	55	117	112	0.96	56	54	-	111
37	No.8, Kansas City	55	117	155	1.32	77	76	-	154
38	No.2, Weirton	75	713	249	0.35	124	121	-	247
39	No.17, S Lake City	59	411	599	1.46	300	292	-	595
40	No.21, E. Chicago	102	300	246	0.82	123	120	-	244
	Concrete Pile								
41	No.4, Locks&Dam[4]	40	209	98	0.47	49	48	-	98
42	No.5, Locks&Dam[4]	51	283	130	0.46	65	64	-	130
43	Locks&Dam25	13	NA	122	NA	61	59	-	121

		Pipe Pile[5]	H-Pile Only[6]	All Data
$(Q_p^* F_b / Q_m)$	μ	1.03	0.83	0.96
	σ	0.13	0.37	0.33

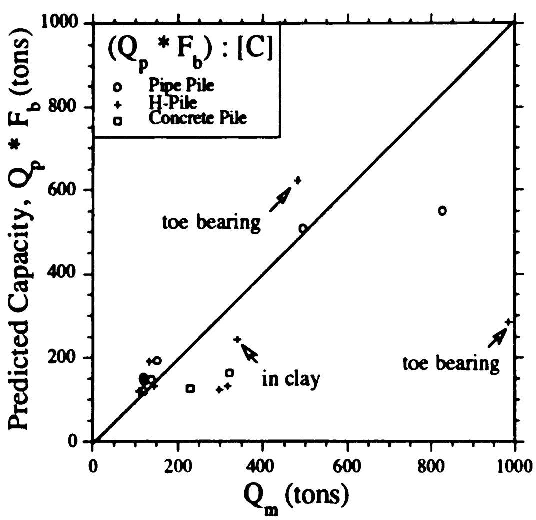
[1]Q_a from DETERMINISTIC approach for the "Uniform" Site at FS=2.00; equivalent β=2.90
[2]Q_a from RELIABILITY-BASED approach for the "Uniform" Site at β=3.00; equivalent FS=2.05
[3]Allowable capacity (Q_a) as compared to Measured Capacity (Q_m)
[4]Allowable Capacity from SPT only (unknown loading test, NLT)
[5]Excluding Pile No.24 & No.25 - hole preaugered before pile is driven
[6]Excluding Pile No.38, No. 39, and No.40 (Pile No.38 & Pile No.39 - Toe bearing; Pile No.39 & Pile No.40 - pile is predominantly embedded in clay)



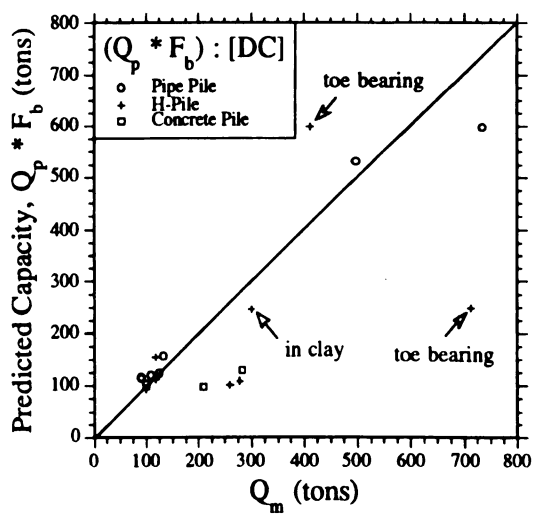
(a) 2" Movement



(b) Davisson's



(c) Chin's



(d) Ave Davisson's & Chin's

Fig. 6.3: Predicted Capacity - The Algorithm Vs. Measured Capacity.

6.3.3 The Algorithm Vs. Previous Studies

6.3.3.1 Site at Northwestern University

One specific comparison of the predicted capacity from the developed algorithm and previous studies can be made by examining the pipe and H-pile tested for the pile capacity prediction event at Northwestern University (Finno et al, 1989b). Since there are two sets of SPT N-values available, the pipe pile in that particular study is represented by Pile No.24 and No.25 for Automatic Trip (AT) Hammer and Safety Hammer (SH) respectively. Similarly, the H-pile is represented by Pile No.32 and No.33 for the AT and SH respectively.

As indicated by the tables of output in Appendix C, the predicted capacities (Q_p*F_b) do not vary much between the two types of SPT hammers (only in the order of 2 to 3 tons).

From the load-movement curves (Fig. A24), the interpreted measured capacity, Q_m, according to [2"], [D], [C], and [DC] criteria are found to be 100, 60, 119, and 90 tons respectively for the pipe pile. Similarly for the H-pile (Fig. A32), Q_m is interpreted at 103, 90, 108 & 99 tons using criteria [2"], [D], [C], and [DC] respectively.

Comparing the Q_m with the predicted capacities from the algorithm, say Pile No.24, the predicted capacities from the algorithm somewhat "overpredicts" the capacity. This is indicated by the ratio of (Q_p*F_b/Q_m) from the algorithm (Table

C24) which are found to be 1.22, 1.32, 1.25, & 1.27 for [2"], [D], [C], and [DC] criteria respectively. For the H-pile, the ratio of (Q_p*F_b/Q_m) is found to be at 0.99, 0.73, 1.10, & 0.94 for the [2"], [D], [C], and [DC] criteria respectively; indicating no trend of "overprediction."

The "overprediction" of the pipe pile No.24 could be due to **preaugered hole** of 12 in. diameter for the 18 in. pipe pile prior to installation (see Section 6.2.1). The other reason is that because the developed algorithm was designed to determined the long term capacity (see Section 5.3.1). From the load-movement curve (in Appendix A), clearly if the measured capacity is determined from the loading test at 43 weeks, the algorithm **would not** "overpredict" the capacity (i.e., using *data points at 43 weeks instead of data points at 2 weeks* to determine the measured capacities). However, the data points of the loading test at 43 weeks were not considered in this example since for all the other piles, the loading tests were done at relatively shorter time, data points at 2 weeks were considered in this case to maintain the consistency of interpreting Q_m at "shortly" after pile is driven. Thus, for this particular pile, it is not entirely true to say that the algorithm "overpredict" the capacity, but rather the Q_m might be "**underestimated**."

Therefore, for this particular site, it is not quite valid to compare the (Q_p*F_b) from the algorithm with the Q_m from the loading test. The predicted capacities from the algorithm can however be fairly compared to the other 24 participants who had made prediction at the foundation congress. The "long term" (after one year) capacity as reported by Finno et al (1989b) by the 24 participants for the pipe pile ranges from 63 tons (about 125 kips) to 185 tons (370 kips) with the mean of 108 tons (215 kips). For the pipe pile as indicated in Fig. 6.4(a), the predicted capacities from the algorithm in general were found to be in agreement with the range of predicted capacities submitted by other participant (Predictor No.25 is the result from the developed algorithm). The predicted capacities from the algorithm were found to be 122, 79, 144, and 114 tons using criteria [2"], [D], [C], and [DC] respectively. The [DC] criterion (at 114 tons) gives *the best* result and matches well with the *mean* of the 24 predictors (i.e., at 108 tons). The [DC] criteria would be the choice if a single value is to be chosen for actual design ([D] is the lower limit and [C] is the upper limit, see also the assumptions in Section 5.3.1).

As indicated in Fig. 6.4(a), the predicted capacity from the [D] criterion lies within the lower range, together with other predictors. Conversely, the predicted capacity from [C] criterion lies within the upper range. In fact, this is the general trend for the developed algorithm.

For the H-pile in Fig. 6.4(b), the algorithm somewhat predicted lower capacities as compared to the *mean* of the 24 predictors for criteria [D] and [DC]. In fact, this general trend is also shown by the mean, μ, for the H-piles in Table

6.4; which is less than unity. However, good agreement is obtained for the interpretation by criteria [2"] and [C]. This gives the notion that for H-pile, "maximum mobilization" of the shaft resistance may have occurred (which is one of the main assumptions of these 2 criteria, see also Sections 3.3.1 and 3.3.3), thus resulting in a better prediction (as compared to [D] and [DC] criteria). Nevertheless (for H-Pile, No.32), good agreement is obtained when the predicted capacity from the algorithm is compared to the measured capacity as indicated by the ratios of the predicted to the measured capacities of 0.99, 0.73, 1.10 and 0.94 by using criteria [2"], [D], [C] and [DC] respectively in Table B9 (Appendix B).

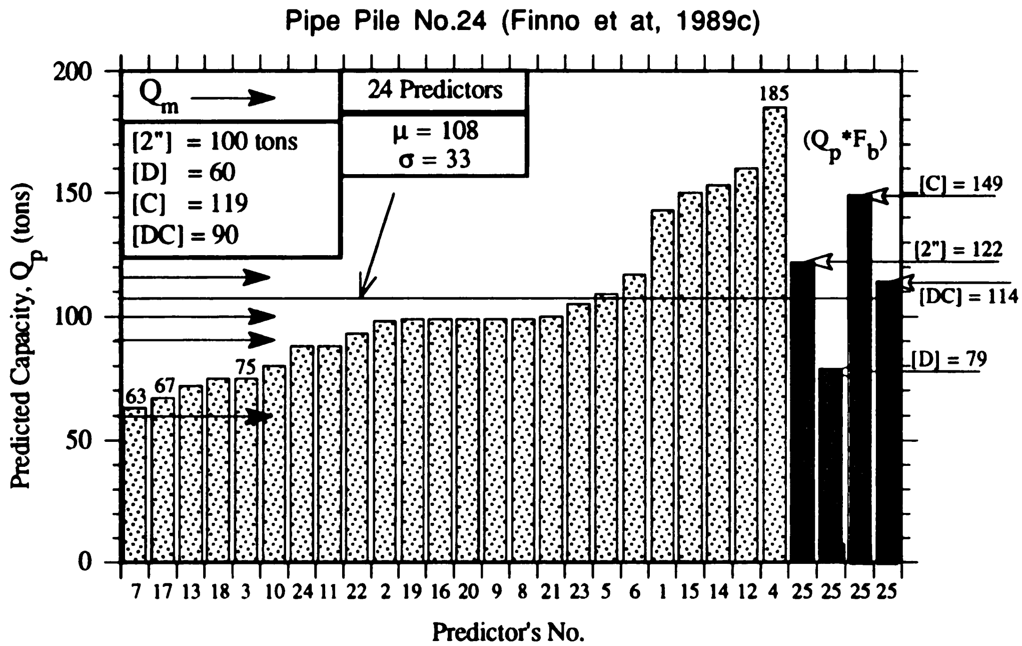
In Fig. 6.4, predictors No.3 and No. 17 represent the capacities as predicted by Wolff (1989) and Coyle and Tucker (1989) respectively.

For the pipe pile in Fig. 6.4(a), both predictors predicted somewhat lower values [at 75 tons (150 kips) and 67 tons (134 kips) by Wolff and Coyle and Tucker respectively], as compared to the mean of the 24 predictors (at 108 tons). However, like many other predictors, the predicted capacity of 75 tons by Wolff is among "the comfortable" single value for design since it is within the "lower limit" of criteria [D] and "upper limit" of criteria [C].

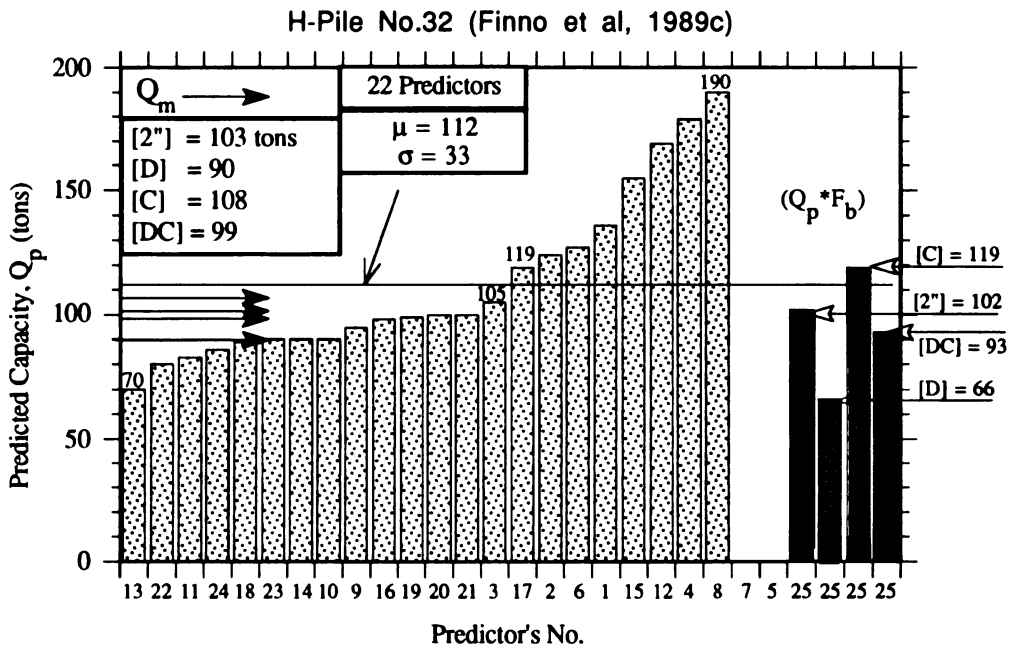
For the H-pile, both Wolff and Coyle and Tucker predicted the closest value as compared to the mean of the 22 predictors (at 112 tons). Wolff and Coyle and Tucker predicted the capacity at 105 tons (210 kips) and 119 tons (238 kips) respectively; which is also 7 tons below and above the mean of the 22 predictors respectively. However, for practical purpose in actual design, and following the general recommendation in this study (i.e., interpret loading test from the average of criteria [D] and [C]), then among the 22 predictors, Wolff's prediction of 105 tons is found to be the nearest to [DC] criteria (at 99 tons).

Using the developed algorithm in this study, the *predicted* capacity of the H-pile as interpreted by [2"] criterion (at 102 tons) is the closest to the *measured* [DC] criteria (99 tons); and the *predicted* capacity using the [C] criterion (at 119 tons) is the closest to the *mean of the 22 predictors* (at 112 tons). This gives a notion that the algorithm should be limited to predict capacity only by criteria [2"] or [C] for H-pile to obtain "good results."

The algorithm developed in this study is for the most part an improvement of the method by Wolff (1989), i.e., the characterization of the relevant parameters in the static equation by integrating the shaft resistance over the corresponding surface area of the pile. However, the algorithm uses a similar static equation as presented by Coyle and Castello (1981) for the shaft resistance (see also Table 5.2); and modified by the inclusion of various factors as presented in Chapter 5.



(a) 18" Pipe Pile, No.24.



(b) H14x73 Pile, No.32.

Fig. 6.4: Predicted Capacity - The Algorithm Vs. Other Predictors.

6.3.3.2 Site at Kansas City

The other set of data that could be compared to the previous study is the data taken from the site in Kansas City (Williams, 1960). Columns {3} and {4} in Table 6.5 are the measured and allowable capacities respectively as reported by Fuller (1960). Columns {5} through {8} are the allowable capacities from the algorithm as calculated from the deterministic approach at the recommended factor of safety (see also the Recommended **FS** in Section 5.3.4). In columns {5} through {9}, the equivalent reliability index is also presented.

At the recommended **FS**, the general trend as before for the allowable capacities is also observed (i.e., [2"] give the highest Q_a, followed by [C], and [D] give the lowest). The allowable capacity for the [DC] criterion is somewhat higher than both the [D] and the [C] criteria mainly because the **FS** is recommended at 2.00. If the **FS**=2.50 and criteria [DC] is used (column {9}), the allowable capacities are found to be very close to the values as reported by Fuller, especially for the pipe piles. For the [DC] criteria, using the **FS**=2.5 is perhaps on the conservative side since the equivalent reliability index is 3.86 (high). Therefore, for the [DC] criteria, the recommended **FS**=2.00 (at an equivalent β=2.90) is felt reasonable. Since the allowable capacities determined by the algorithm is very close to Fuller's values (at **FS**=2.5), lower safety measures (i.e., minimum of **FS**=2.00

widely used in practice) can in fact be considered with confidence.

Table 6.5: Allowable Capacity - The Algorithm Vs. Previous Study.

PILE NO.	PILE TYPE	AS REPORTED BY FULLER (1960)		OUTPUT FROM THE ALGORITHM				
				At the Rec'md FS (Table 5.7)				
				[2"]	[D]	[C]	[DC]	[DC]
			FS=2.0	FS=2.0 β 2.37	FS=2.5 β 3.18	FS=2.5 β 3.49	FS=2.0 β 2.90	FS=2.5 β 3.86
		Q_m (tons)	Q_a (tons)	Q_a (tons)	Q_a (tons)	Q_a (tons)	Q_a (tons)	Q_a
{1}	{2}	{3}	{4}	{5}	{6}	{7}	{8}	{9}
	Pipe Pile							
26	No.3, Kansas City	85	**42.5**	64	31	48	51	**41**
27	No.3A, Kansas City	105	**52.5**	72	36	58	60	**48**
28	No.7, Kansas City	110	**55**	74	37	59	62	**49**
29	No.7A, Kansas City	123	**61.5**	91	46	77	78	**63**
	H - Pile							
36	No.4, Kansas City	110	**55**	68	68	53	56	**45**
37	No.8, Kansas City	116	**58**	90	90	76	77	**62**

6.4 Summary

This chapter has illustrated how the developed algorithm could be used to determine the allowable capacity either from deterministic or reliability-based approaches. Using the recommended safety measures, the appropriate allowable capacity can be found from the predicted capacity. The predicted capacity as determined from the algorithm compares very well with other studies. In design process, the charts can be used for rapid determination of allowable capacities.

The predicted capacities (i.e., Q_p*F_b) for pipe piles compare very well with the measured capacity from loading test values; the (Q_p*F_b) normalized by Q_m is found to be very close to unity for all the criteria, except the [2"] criteria which is slightly overestimated.

For the H-piles, the predicted capacity (i.e., Q_p*F_b) is slightly underestimated, but the [2"] and [C] criteria can be used.

The developed algorithm perhaps works best for pipe piles. This could be due to the parameters in the algorithm, which was derived using only data from pipe piles.

CHAPTER 7

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

7.1 Summary

The purpose of this research was to develop a direct relation of safety measures as determined from the conventional deterministic approach and the reliability-based approach. It was felt that a direct interrelation between the two approaches could help the interpretation of safety measures (factor of safety, **FS**, and reliability index, β), and the design process itself (i.e., the determination of the allowable capacity for pile foundation).

At present, the determination of the allowable capacity from Standard Penetration Test N-values is primarily done either by deterministic or reliability-based approach. In current design methods, the safety measures as found by the two approaches, in general, have not been correlated; i.e., no direct interrelation between the deterministic (**FS**) and reliability index (β).

In the deterministic design process, the procedure with respect to assigning the factor of safety is a "straight forward" procedure from static formula, thus routinely used by practitioners. On the other hand, design of pile foundation by reliability methods, besides the need to "select" the critical parameter values in the static formula (as in the case of deterministic approach), demands an additional statistical procedure. Therefore, sometimes design by reliability approach gives an impression that the process is a "statistical procedure" rather than a "geotechnical problem". Due to the unfamiliar theories and an added effort (to geotechnical engineers), the reliability-based methods have lagged behind in application and have not been applied on routine basis. Nevertheless, with recent improvement in reliability theories, and with added improvement in computing capabilities, reliability methods could be the trend of the future. For example, tedious calculations once not possible to do manually are now possible on micro computers.

Thus, this study was concentrated on developing an interrelation algorithm of safety measures which are applicable by both design approaches; in the form of equations and design charts. The developed charts could possibly help designers in understanding the safety measures involved; and serve as a transition tool towards reliability-based design of the future.

Due to the complexity of problems associated with pile foundations, one major issue was immediately encountered when an attempt was made to develop the interrelation, i.e, the determination of the predicted capacity, Q_p. Therefore, the secondary work in this research was to define a consistent and systematic procedure to determine Q_p. This was achieved by, (i) identifying a specific analytical formula to calculate Q_p, and (ii) calibrating the calculated Q_p with the measured capacity (Q_m) from the loading test. In the calibration process of the predicted capacity (Q_p) with the "true" measured capacity (Q_m), the ratio of (Q_p/Q_m) was taken as a random variable and defined as the bias factor (F_b).

To interrelate the deterministic factor of safety (FS) and the equivalent safety measure from the reliability-based (β), first-order uncertainty analysis was then applied. The usage of the developed model was then illustrated by examples.

7.2 Conclusions

The algorithm developed in this research is not intended to replace proven and well established methods for the determination of allowable capacity of a pile foundation. Nevertheless, it is felt that the recommended equations, design charts and the recommended safety measures could be a valuable tool in the design process of a pile foundation. The direct interrelation of FS and β could be an alternative method of understanding the reliability of pile foundation design. Thus, the algorithm developed in this research could be a supplementary tool for designers, and specification and code writers.

The major conclusions which resulted from this study can be stated as follows.

7.2.1 Predicted Capacity

With respect to the predicted capacity (Q_p), following conclusions can be made:

1. In using data from Standard Penetration Test (SPT), the "high degree of engineering judgement" required to select a realistic N-value for design in routine design practice is **virtually eliminated** in the developed algorithm. This is also true about the complex issue related to the "selection" of the site lateral earth pressure (K_s) and other parameters used (see implied assumptions in Section 5.4).
2. The ratio of the toe capacity to the total capacity (R_t), the shaft correction factor (F_s), and the toe correction factor (F_t) can be expressed as functions of the embedment length of the pile (L_e) and pile diameter (d). The inclusion of correction factors (F_s and F_t) is in essence what is better called as the "**modified β^*-Method**".
3. Different bias factors (F_b) should be used for the determination of predicted capacity (Q_p), depending on what loading test criteria the comparison is to be made (i.e., according to the 2" Movement [2"], Davisson's [D], Chins' [C] or the average of Davissons' and Chins' [DC]). Also, the bias factor chosen is dependent upon the type of loading test

(i.e., Constant Rate of Penetration, CRP, or Constant Load, CL). If the type of loading test is unknown, the F_b as recommended for NLT should be used.

4. The predicted capacity is sensitive to the **underlying** variability at the **site**, i.e., the standard deviation of the distribution of measured capacity over predicted capacity (s values); small changes in s could affect the value of allowable capacity, Q_a, quite measurably. Therefore, it is appropriate that the Q_a determined for "non-uniform" site soil properties (i.e., sites with sand and clay) based on a relatively larger s values is conservative. For "uniform" site soil properties (i.e., site which is predominantly in sand) lower s values should be applied.
5. As indicated by the value of μ (i.e., Q_p*F_b/Q_m), the predicted capacity for the pipe piles are found to have shown better results as compared to the H-piles or the concrete piles.

7.2.2 Safety Measures

With respect to the safety measures (deterministic factor of safety, **FS**, or reliability index, β), the following conclusions can be made:

1. For the determination of allowable capacity, the factor of safety (**FS**) as used for design in deterministic approach **can be interrelated** to the reliability index (β) as found in reliability-based approach.

2. The recommended safety measures are dependent upon the value of the statistical bias factor of the prediction (F_b) and the variability of the predicted capacity at the site (s).

3. The **reliability-based approach provides more** information about scatter and dispersion associated with uncertainties of soil properties at the site. For example, if designing by the deterministic approach and without any knowledge about the value of s (which was actually derived from the reliability approach), the same value of **FS** might be assigned for a "uniform" and "non-uniform" sites. However, if designing by the reliability approach, this study indicated that different value of safety measure should be used (i.e, at the respective value of s). And rational selection of safety measures can be made.

7.3 Recommendations

The algorithm was developed from past experience of several previous studies with proven acceptable results. The recommended parameters (R_t, F_s, F_t, F_b and s) are derived in a systematic and rational manner. A similar procedure can be followed for the derivation of new sets of parameters (i.e., R_t, F_s, F_t, F_b and s) for other set of situation (e.g. other in-situ soil tests and/or for cohesive soils).

Following are recommendations with respect to the application of the developed algorithm and further research.

7.3.1 Application of the Algorithm

This study was limited to using soil exploration data from the Standard Penetration Test (SPT), and the loading test data was limited to certain selection criteria as described in Section 5.2. With respect to application of the algorithm, the following suggestions and limitations should be realized:

1. Application of the developed algorithm is intended and most appropriately used only for cohesionless soil.
2. In the derivation of the recommended parameters (i.e., R_t, F_s, F_t, F_b and s);

(i) certain important issues related to pile foundations which may directly affect the predicted capacity are not specifically addressed (e.g., residual stresses in piles after driving, and stress history of soil),

(ii) the parameters are derived from limited SPT data base and limited load-movement selection (see Section 5.2).

7.3.2 Further Research

Potential refinement to the recommended parameters, and consequently the algorithm, may be explored by;

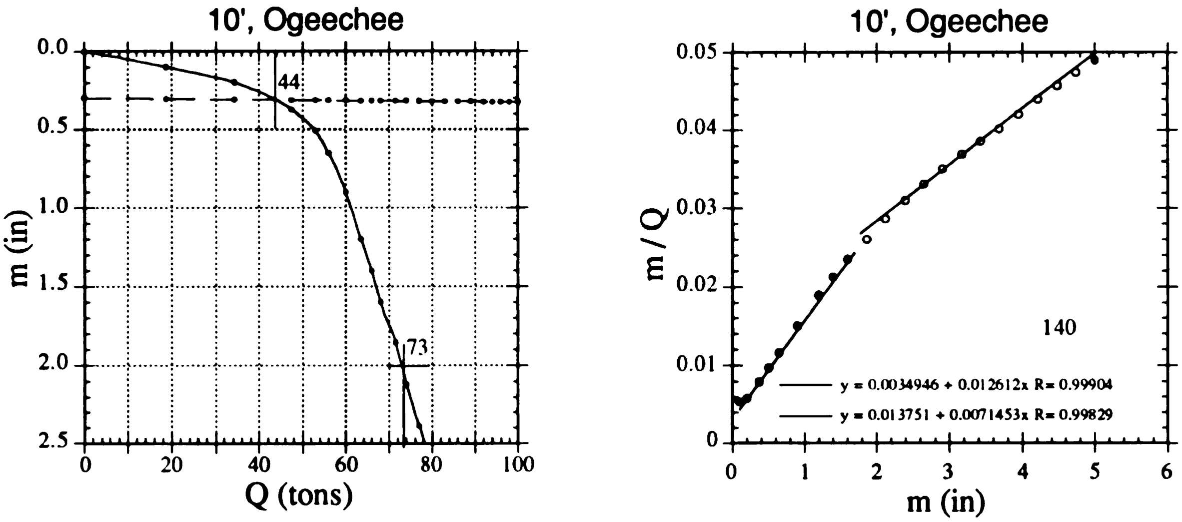
1. Use more data and better "data quality" for the derivation.
2. Employ more rigorous curve fitting procedures (such as step-wise regression analysis for better curve fitting and determine which pile or soil parameter affect the desired factors the most).
3. In this study, the determination of the bias factor, F_b, was lumped together for the shaft and toe to determine the predicted capacity, Q_p. Perhaps better prediction of Q_p could be achieved if separate bias factors were found for the shaft and toe.

4. Perhaps the most important consideration for the overall prediction of pile capacity is to obtain an entirely new set of R_t, F_s, F_t, F_b and s. This can be done by using data from other rigorous and more versatile soil exploration methods (such as Cone Penetration Test or Dilatometer test) or correlation from laboratory analysis. A similar derivation as used in this research can be employed, which in addition could also include the above three recommendations.

APPENDICES

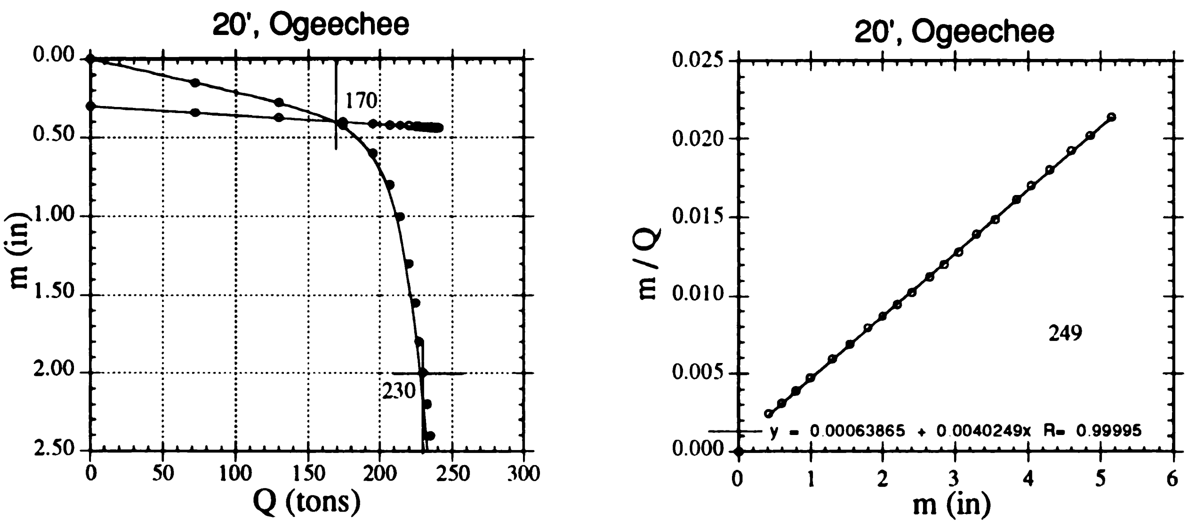
APPENDIX A

Fig. A: Measured Capacities From Load-Movement Curves



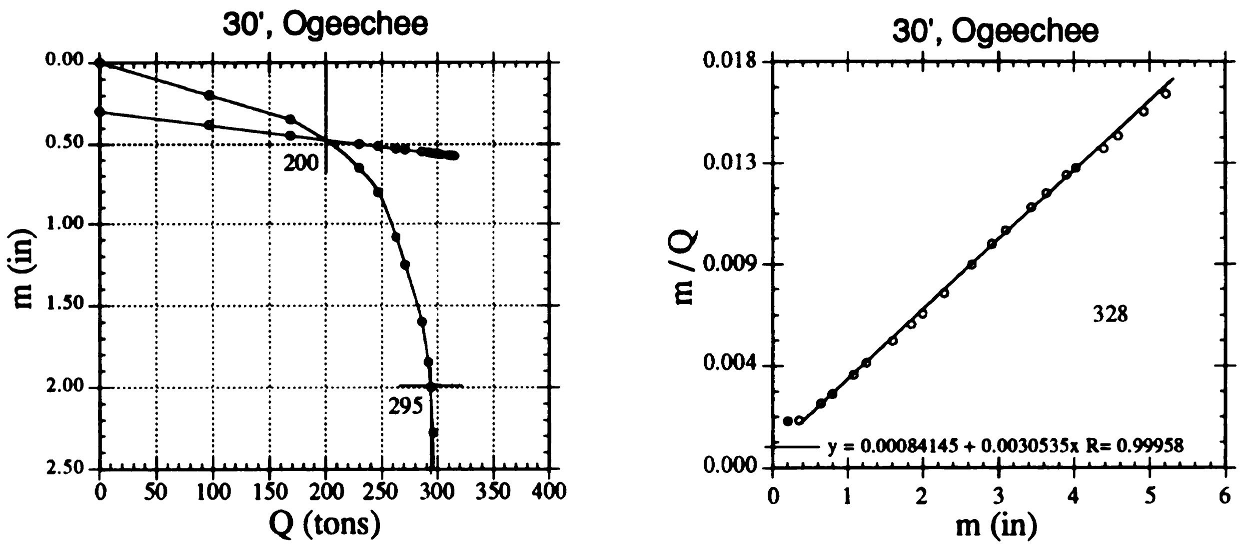
(a) Criteria [2"] & [D] (b) Criteria [C]

Fig. A1 Measured Capacities - Pipe Pile No.1.



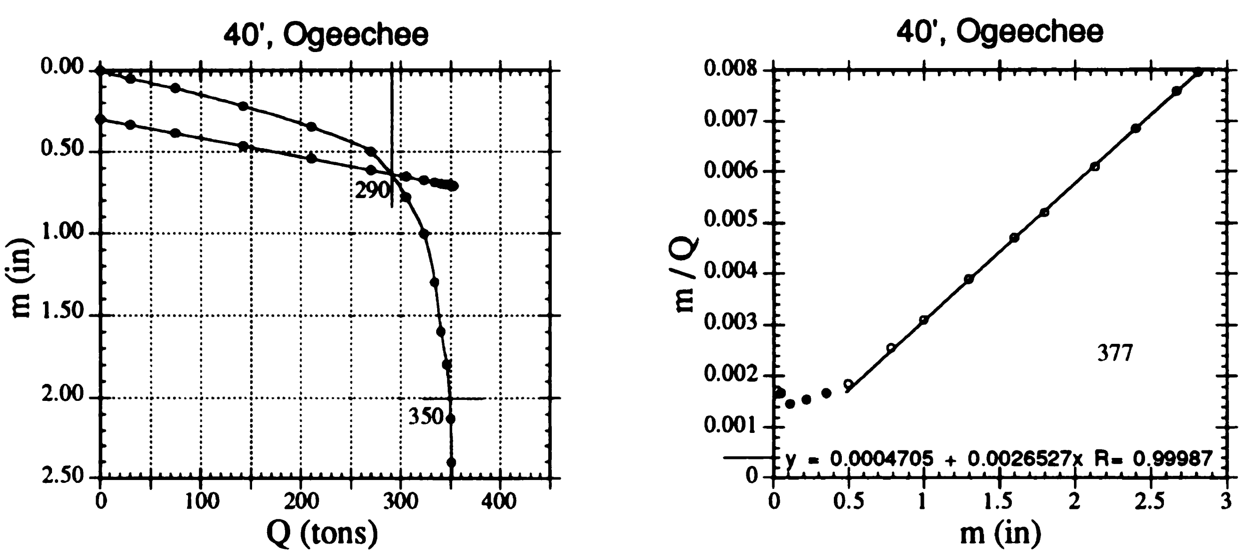
(a) Criteria [2"] & [D] (b) Criteria [C]

Fig. A2 Measured Capacities - Pipe Pile No.2.



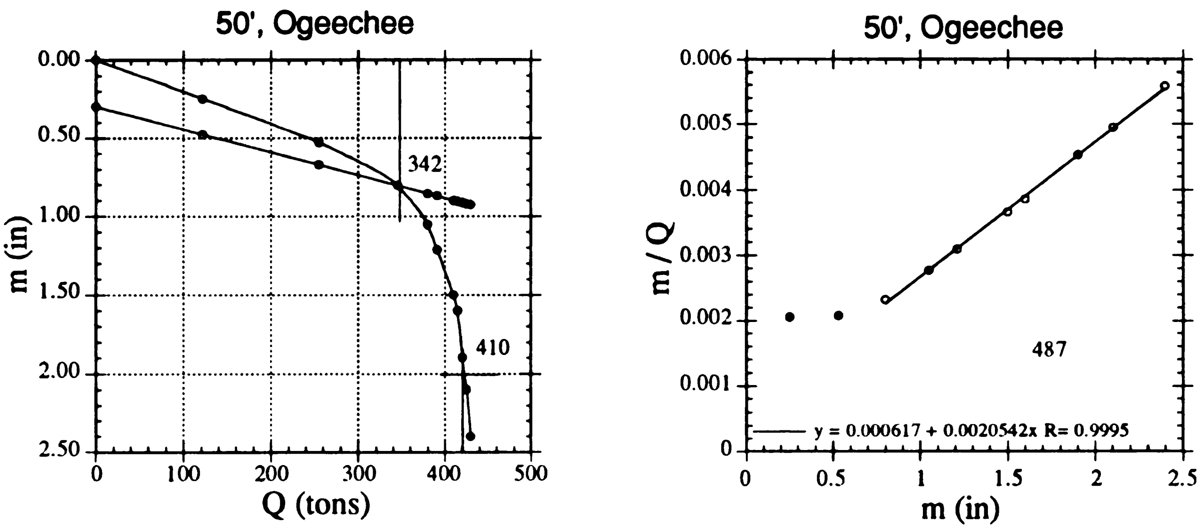
(a) Criteria [2"] & [D]   (b) Criteria [C]

Fig. A3 Measured Capacities - Pipe Pile No.3.



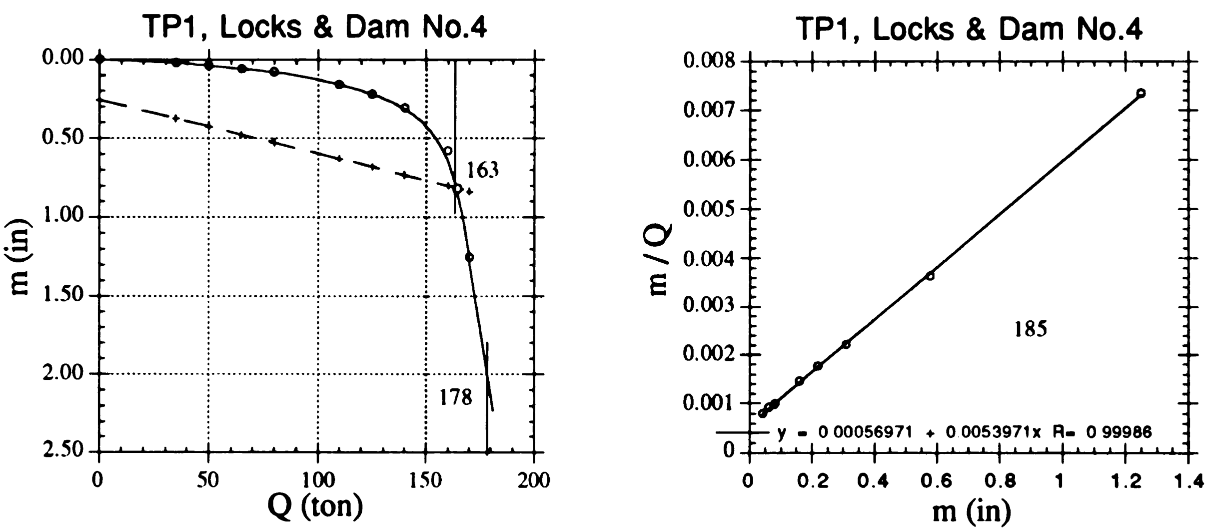
(a) Criteria [2"] & [D]   (b) Criteria [C]

Fig. A4 Measured Capacities - Pipe Pile No.4.



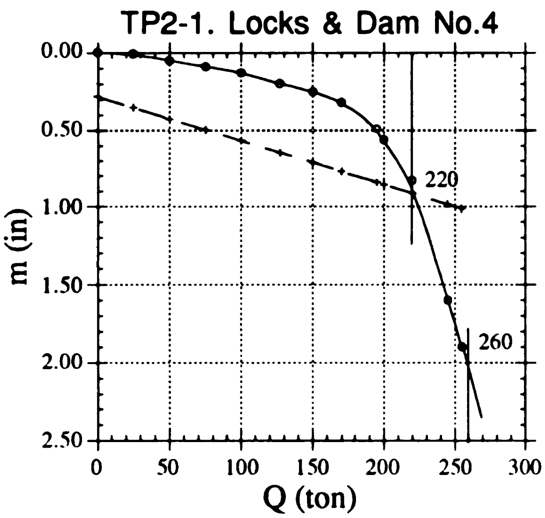
(a) Criteria [2"] & [D] (b) Criteria [C]

Fig. A5 Measured Capacities - Pipe Pile No.5.

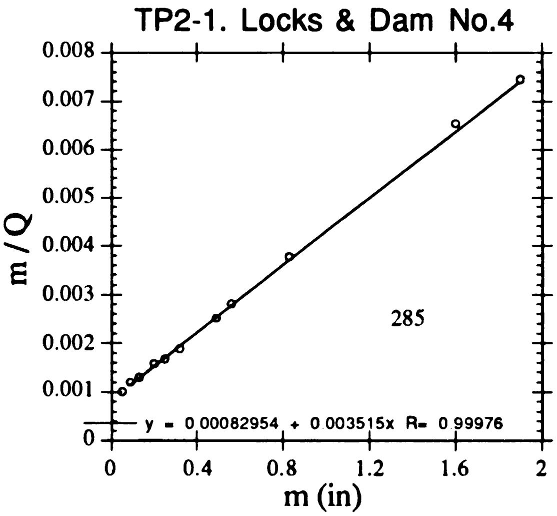


(a) Criteria [2"] & [D] (b) Criteria [C]

Fig. A6 Measured Capacities - Pipe Pile No.6.

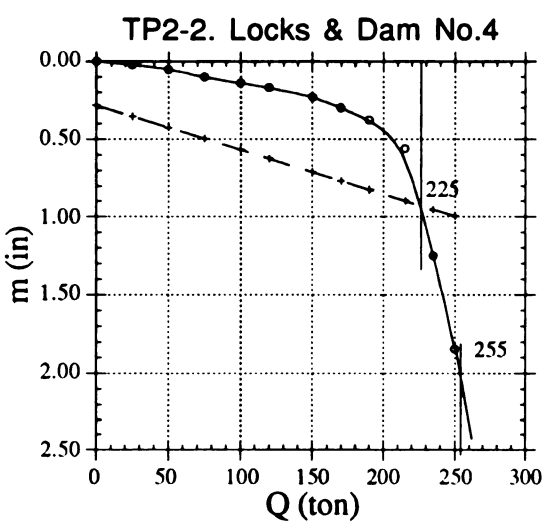


(a) Criteria [2"] & [D]

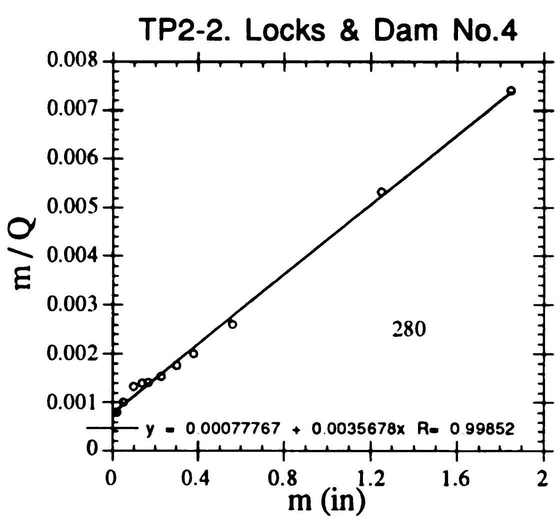


(b) Criteria [C]

Fig. A7 Measured Capacities - Pipe Pile No.7.

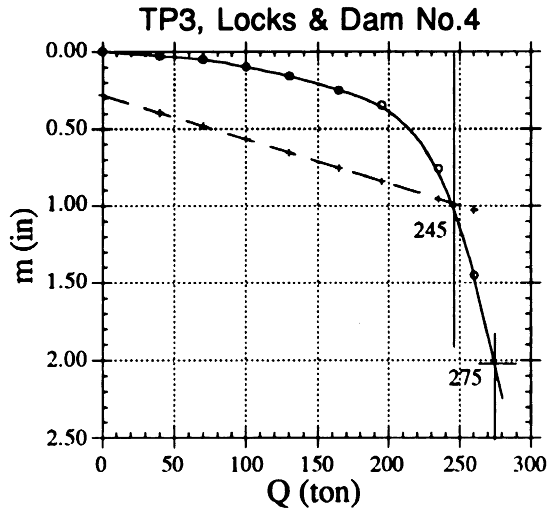


(a) Criteria [2"] & [D]

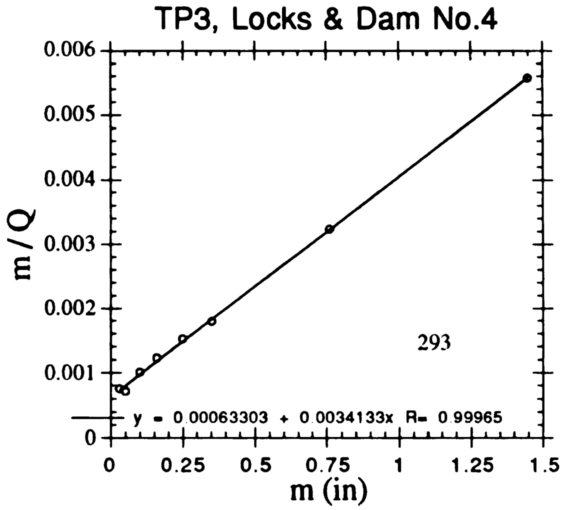


(b) Criteria [C]

Fig. A8 Measured Capacities - Pipe Pile No.8.

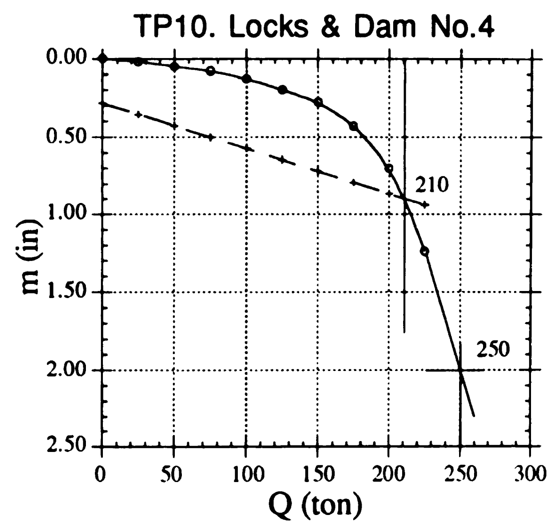


(a) Criteria [2"] & [D]

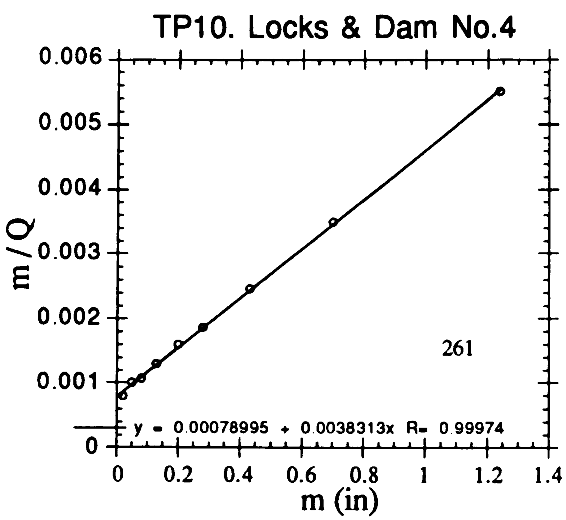


(b) Criteria [C]

Fig. A9 Measured Capacities - Pipe Pile No.9.

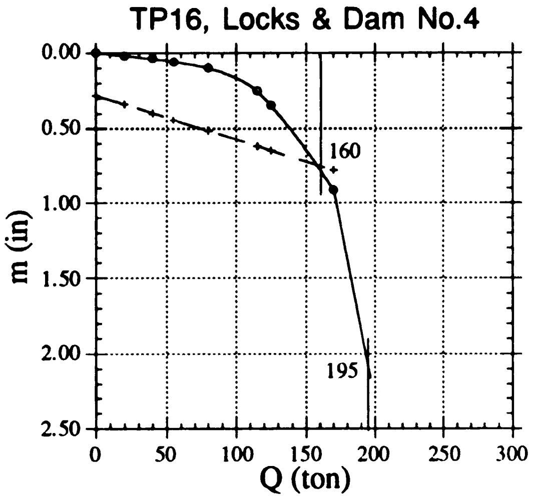


(a) Criteria [2"] & [D]

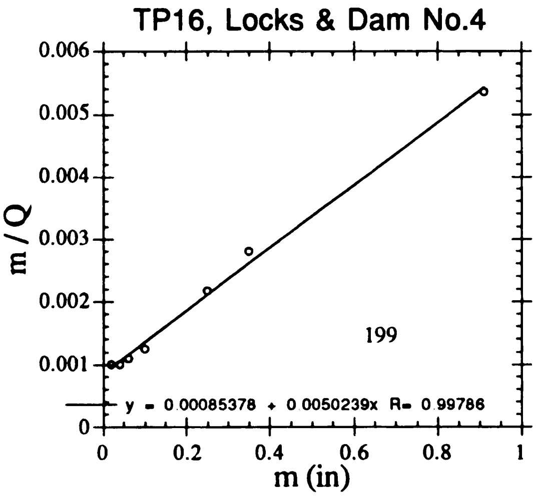


(b) Criteria [C]

Fig. A10 Measured Capacities - Pipe Pile No.10.

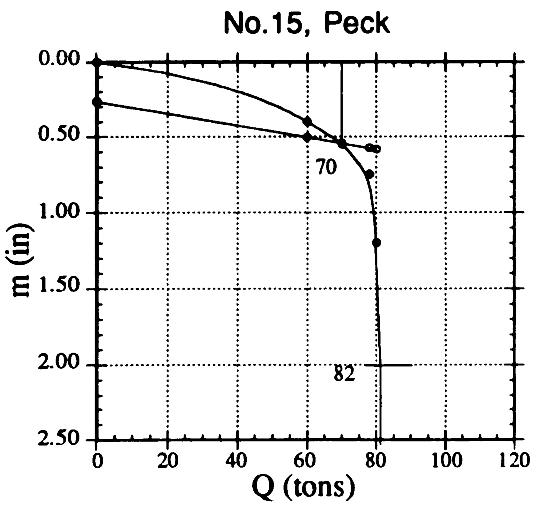


(a) Criteria [2"] & [D]

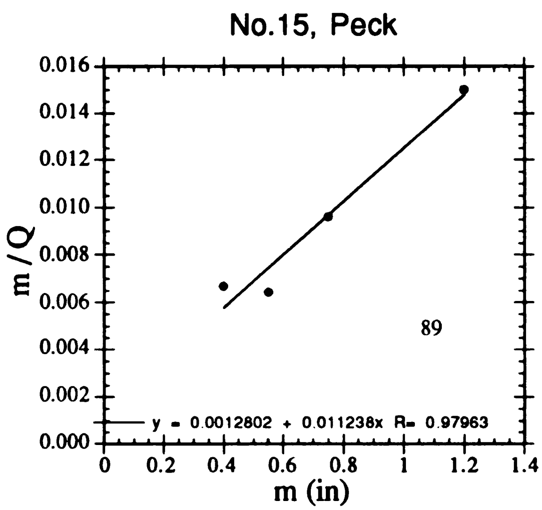


(b) Criteria [C]

Fig. A11 Measured Capacities - Pipe Pile No.11.

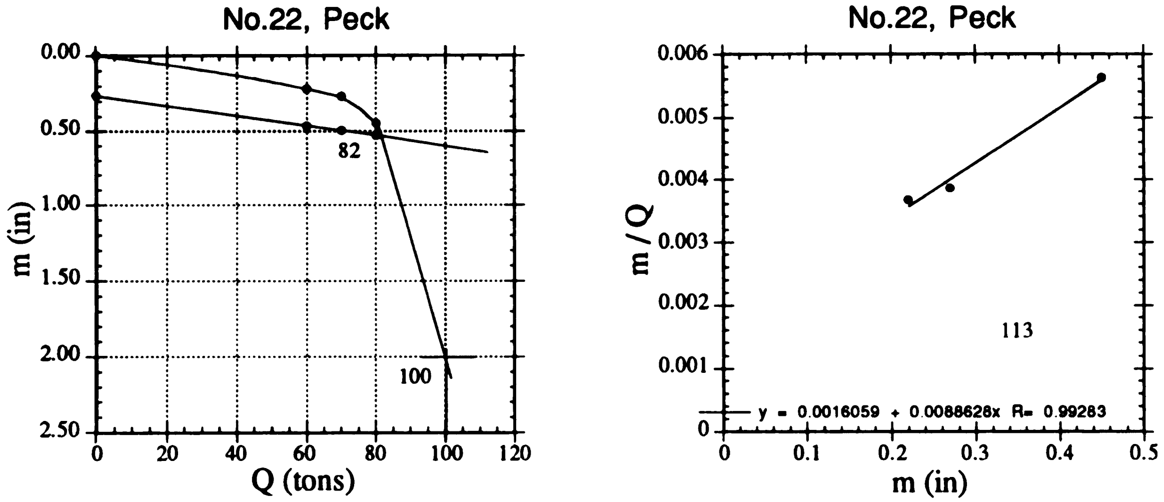


(a) Criteria [2"] & [D]



(b) Criteria [C]

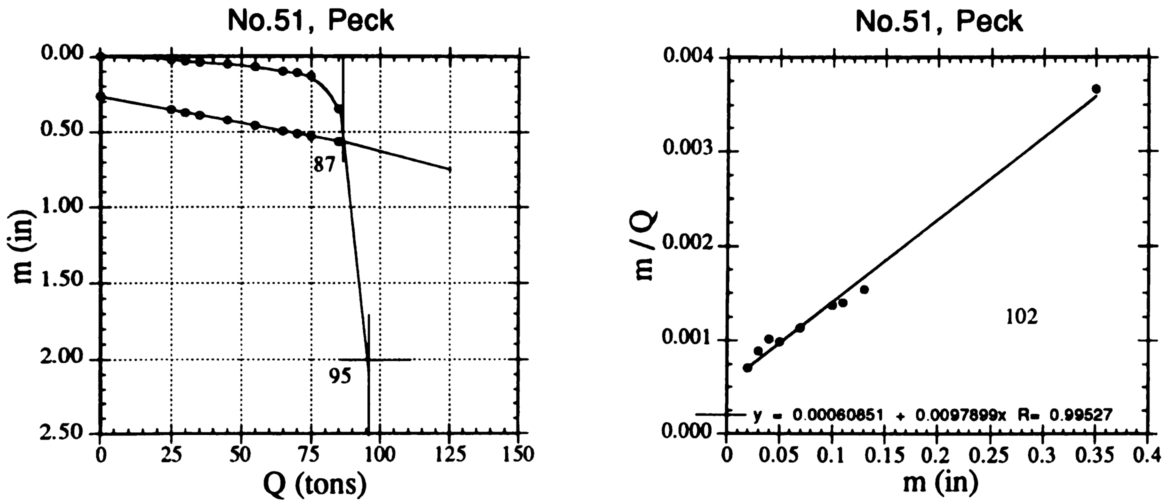
Fig. A12 Measured Capacities - Pipe Pile No.12.



(a) Criteria [2"] & [D]

(b) Criteria [C]

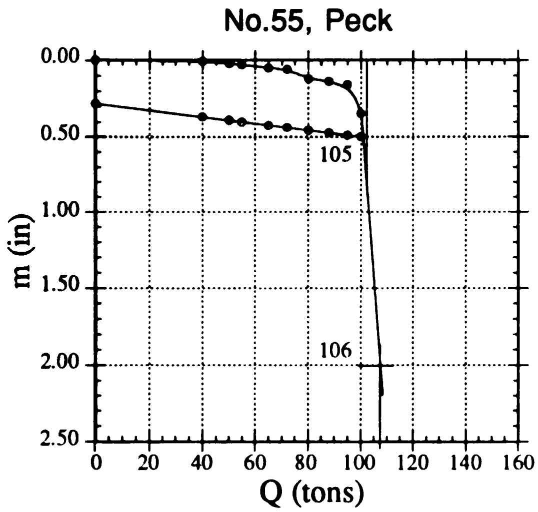
Fig. A13 Measured Capacities - Pipe Pile No.13.



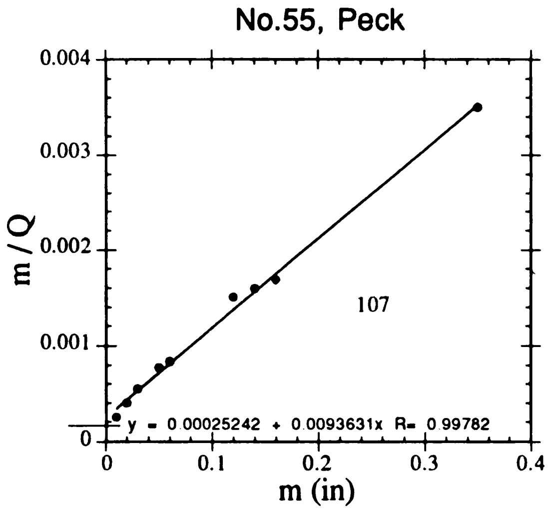
(a) Criteria [2"] & [D]

(b) Criteria [C]

Fig. A14 Measured Capacities - Pipe Pile No.14.

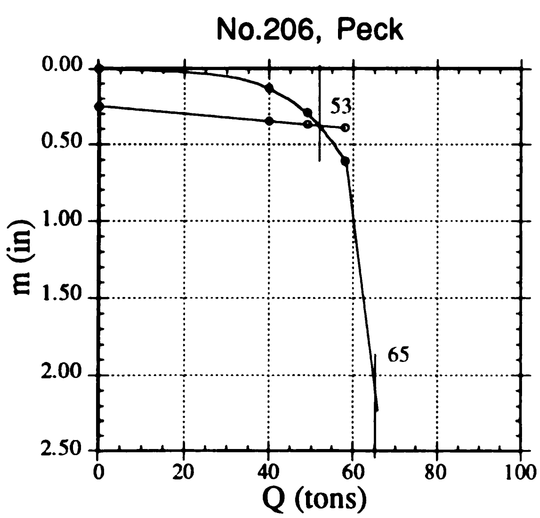


(a) Criteria [2"] & [D]

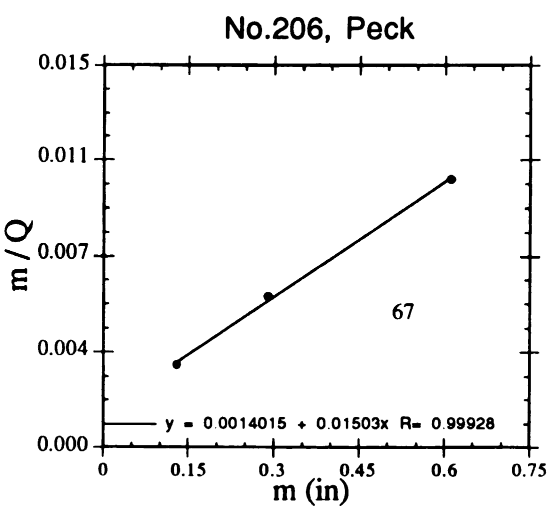


(b) Criteria [C]

Fig. A15 Measured Capacities - Pipe Pile No.15.

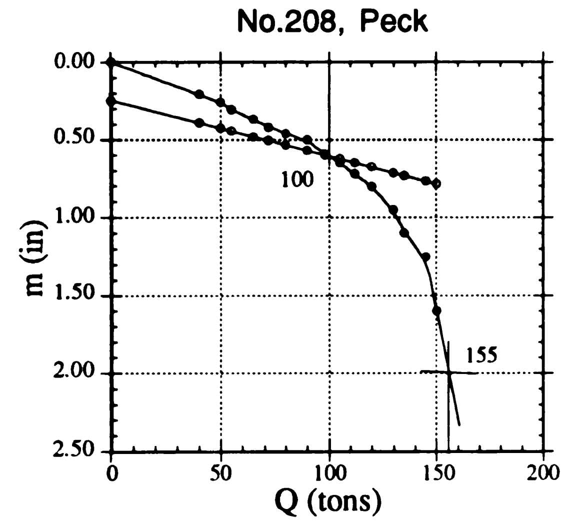


(a) Criteria [2"] & [D]

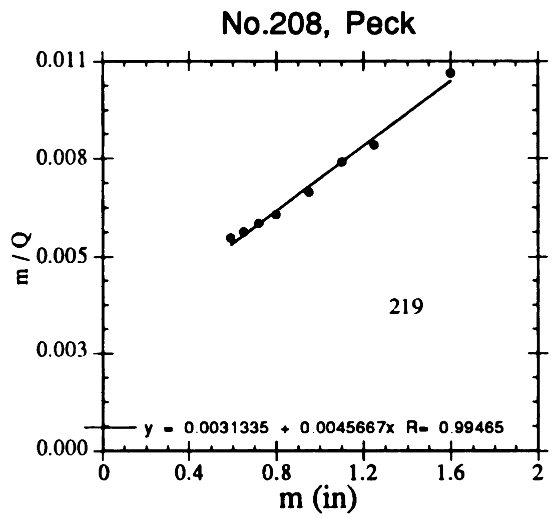


(b) Criteria [C]

Fig. A16 Measured Capacities - Pipe Pile No.16.

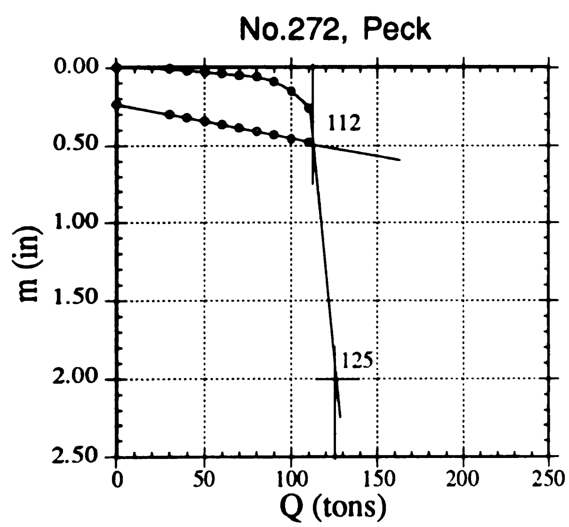


(a) Criteria [2"] & [D]

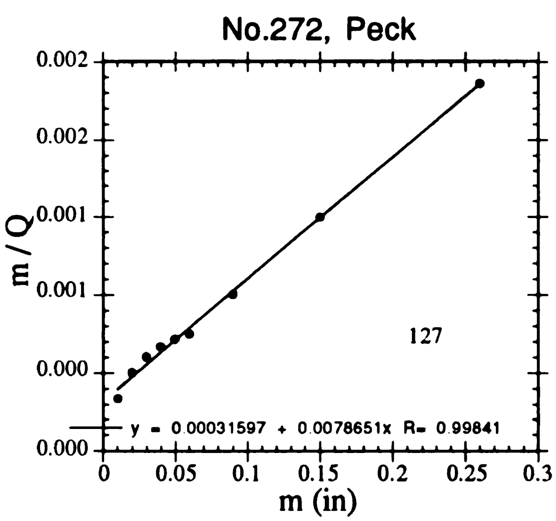


(b) Criteria [C]

Fig. A17 Measured Capacities - Pipe Pile No.17.

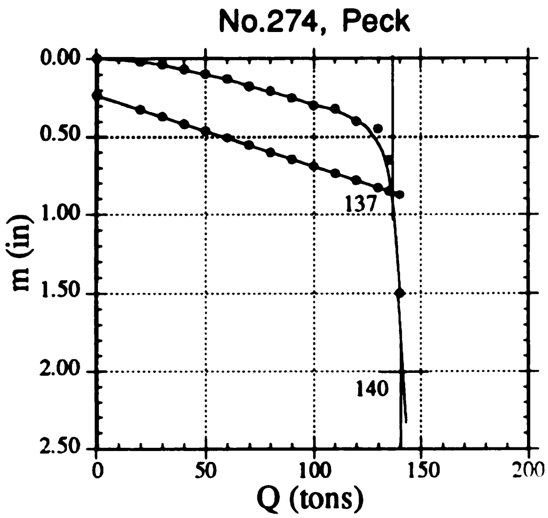


(a) Criteria [2"] & [D]

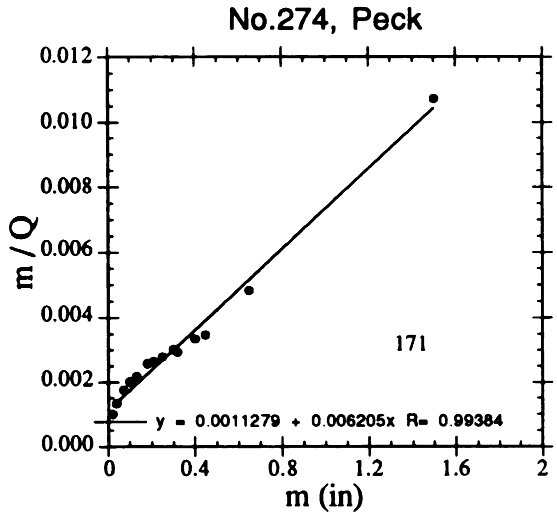


(b) Criteria [C]

Fig. A18 Measured Capacities - Pipe Pile No.18.

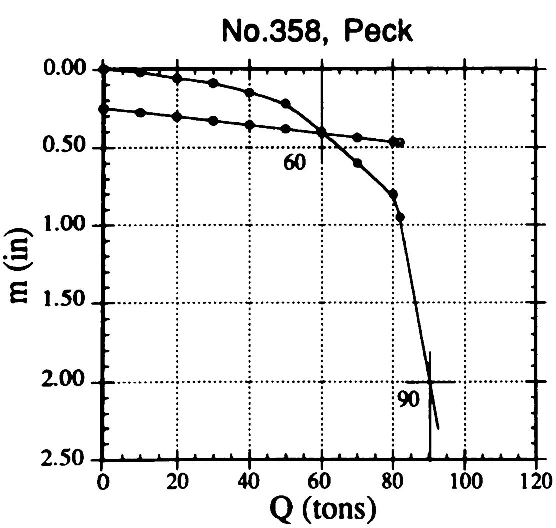


(a) Criteria [2"] & [D]

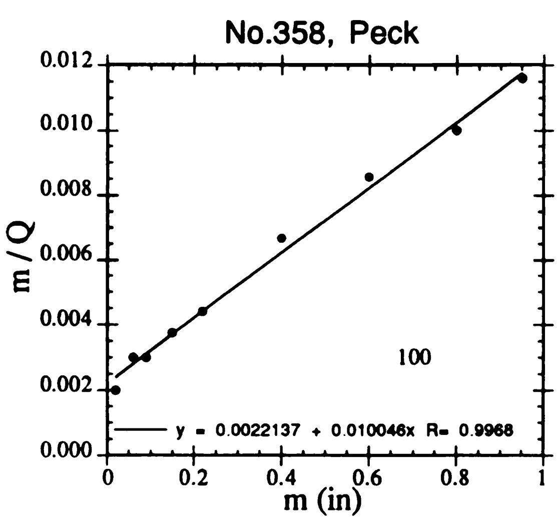


(b) Criteria [C]

Fig. A19 Measured Capacities - Pipe Pile No.19.

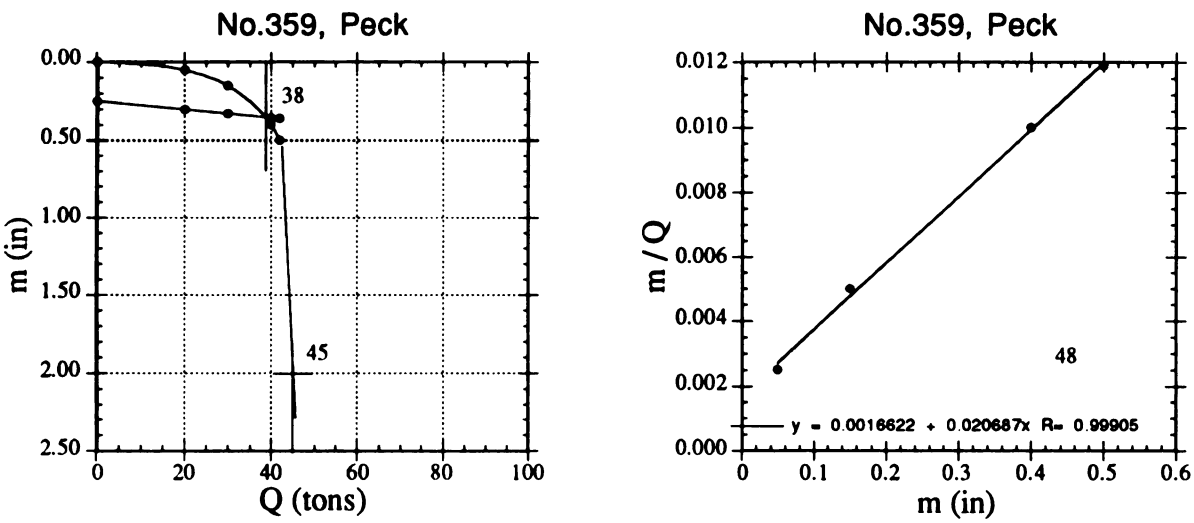


(a) Criteria [2"] & [D]



(b) Criteria [C]

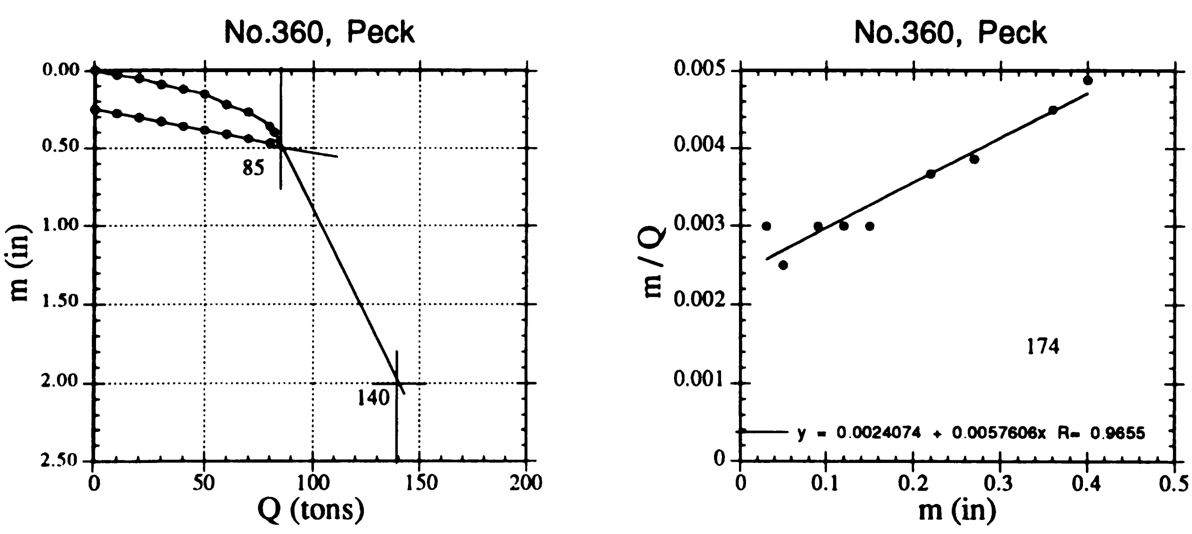
Fig. A20 Measured Capacities - Pipe Pile No.20.



(a) Criteria [2"] & [D]

(b) Criteria [C]

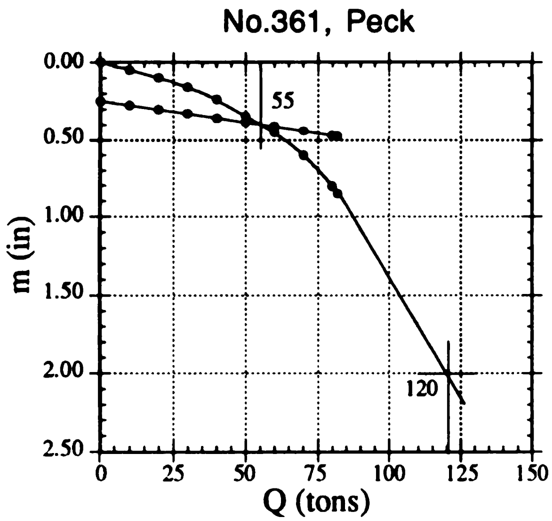
Fig. A21 Measured Capacities - Pipe Pile No.21.



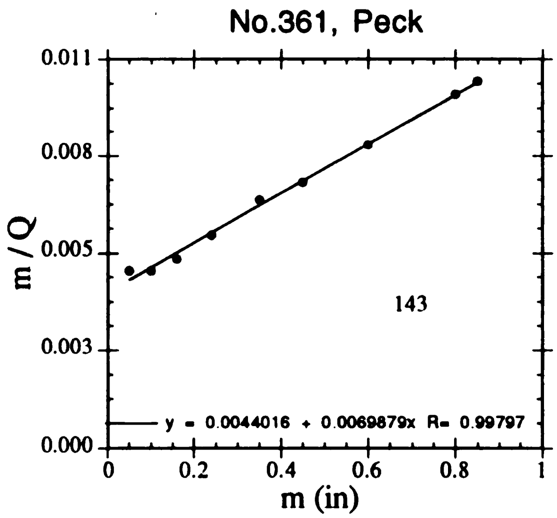
(a) Criteria [2"] & [D]

(b) Criteria [C]

Fig. A22 Measured Capacities - Pipe Pile No.22.

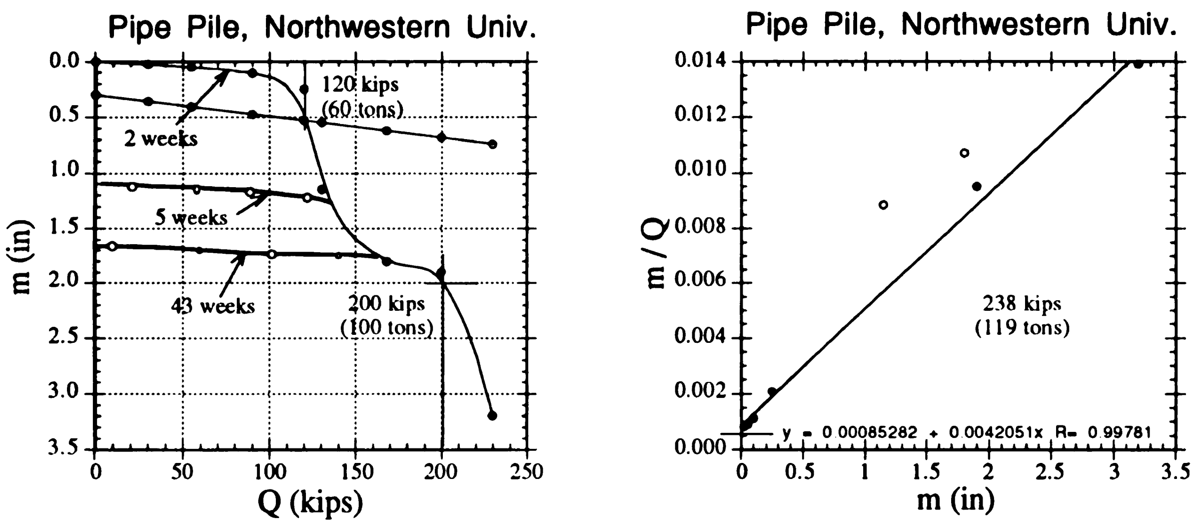


(a) Criteria [2"] & [D]



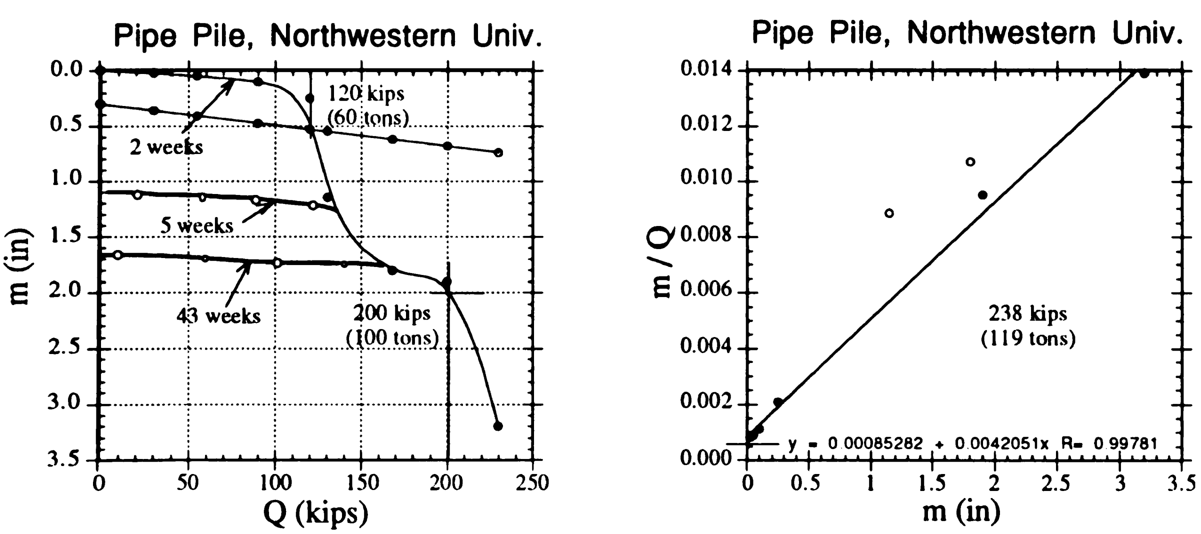
(b) Criteria [C]

Fig. A23 Measured Capacities - Pipe Pile No.23.



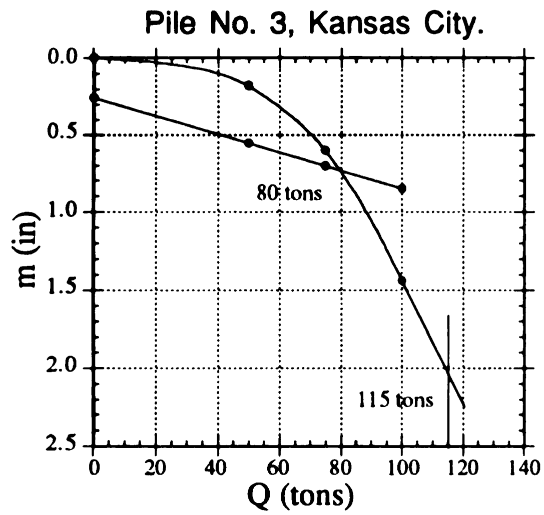
(a) Criteria [2"] & [D] (b) Criteria [C]

Fig. A24 Measured Capacities - Pipe Pile No.24.

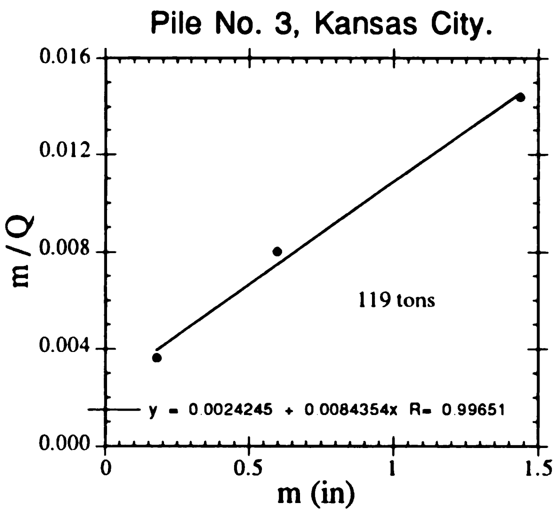


(a) Criteria [2"] & [D] (b) Criteria [C]

Fig. A25 Measured Capacities - Pipe Pile No.25.

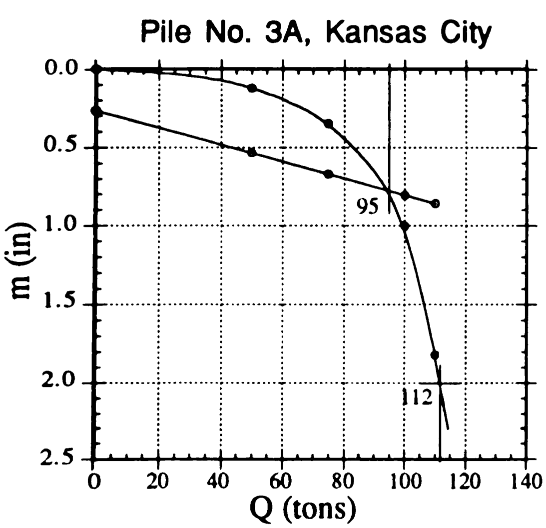


(a) Criteria [2"] & [D]

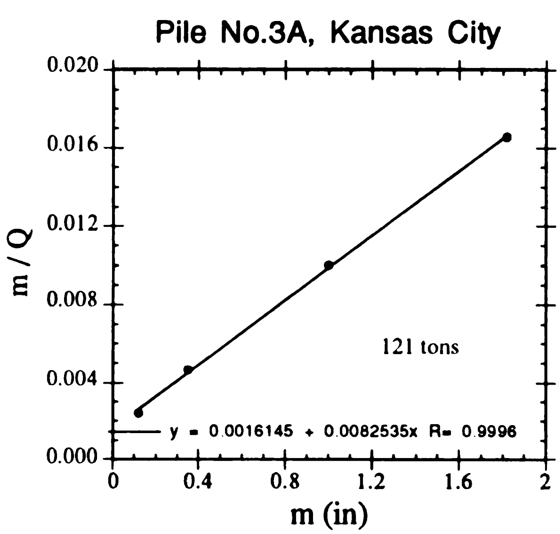


(b) Criteria [C]

Fig. A26 Measured Capacities - Pipe Pile, No.26.

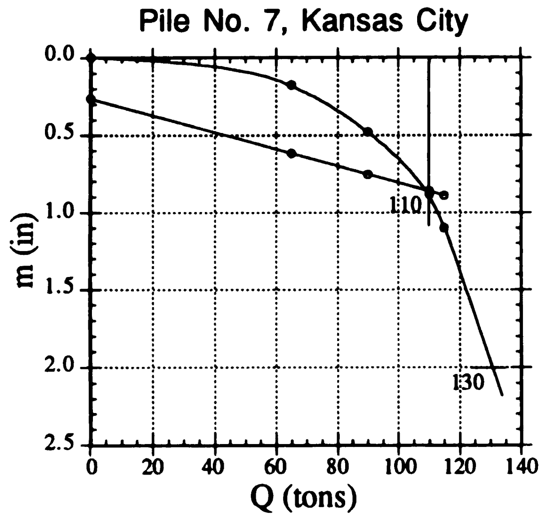


(a) Criteria [2"] & [D]

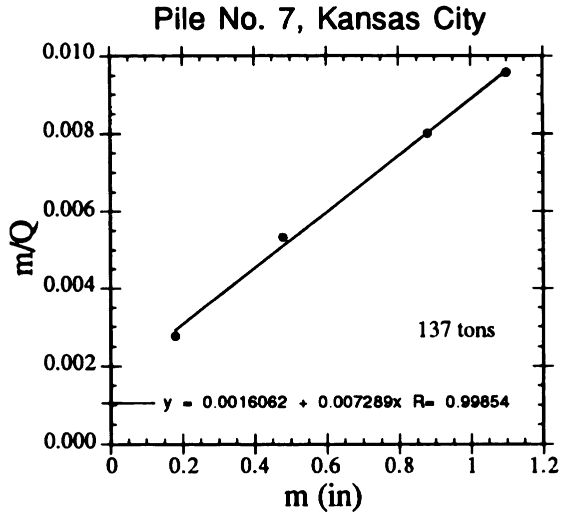


(b) Criteria [C]

Fig. A27 Measured Capacities - Pipe Pile, No.27.

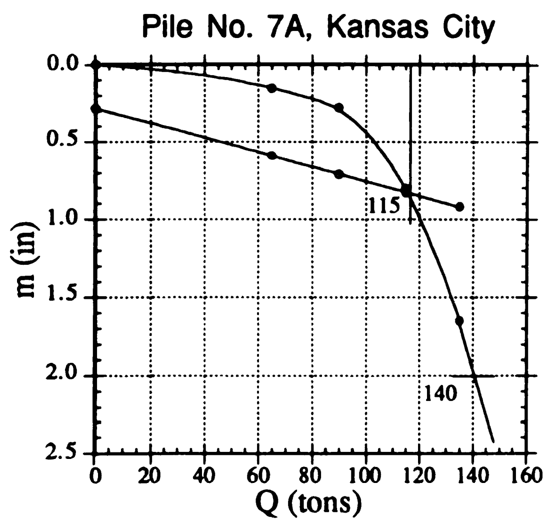


(a) Criteria [2"] & [D]

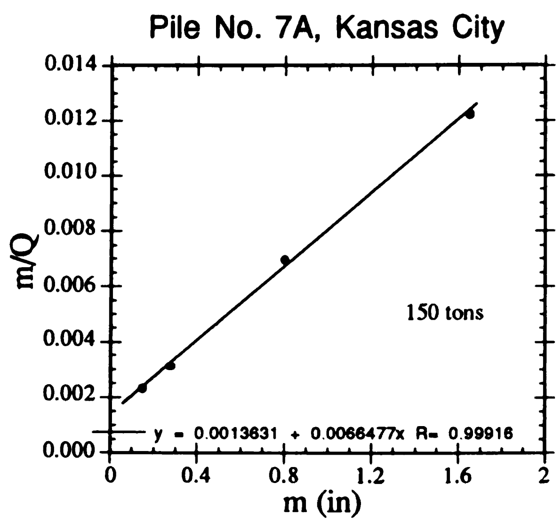


(b) Criteria [C]

Fig. A28 Measured Capacities - Pipe Pile, No.28.

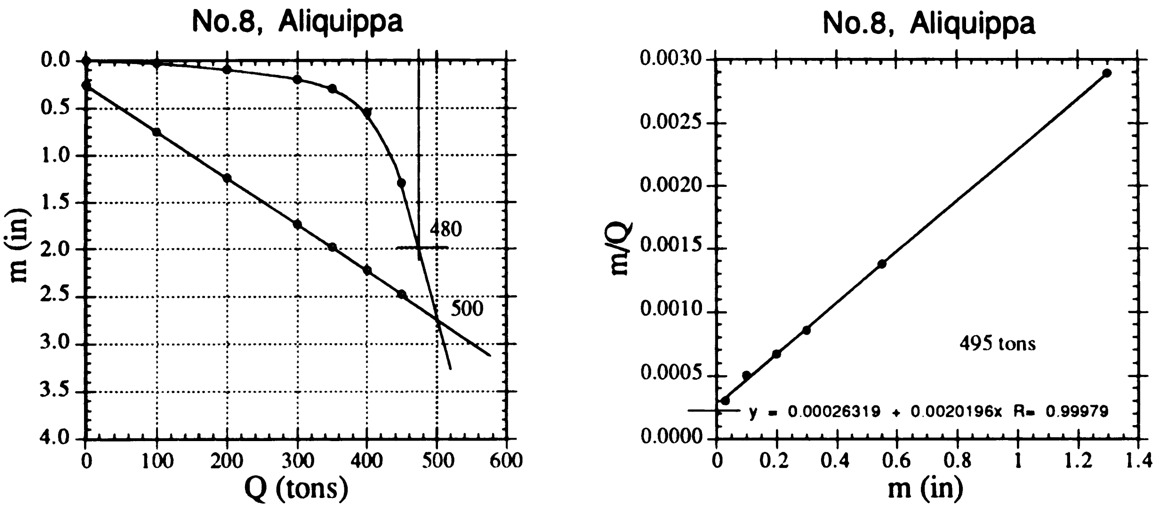


(a) Criteria [2"] & [D]



(b) Criteria [C]

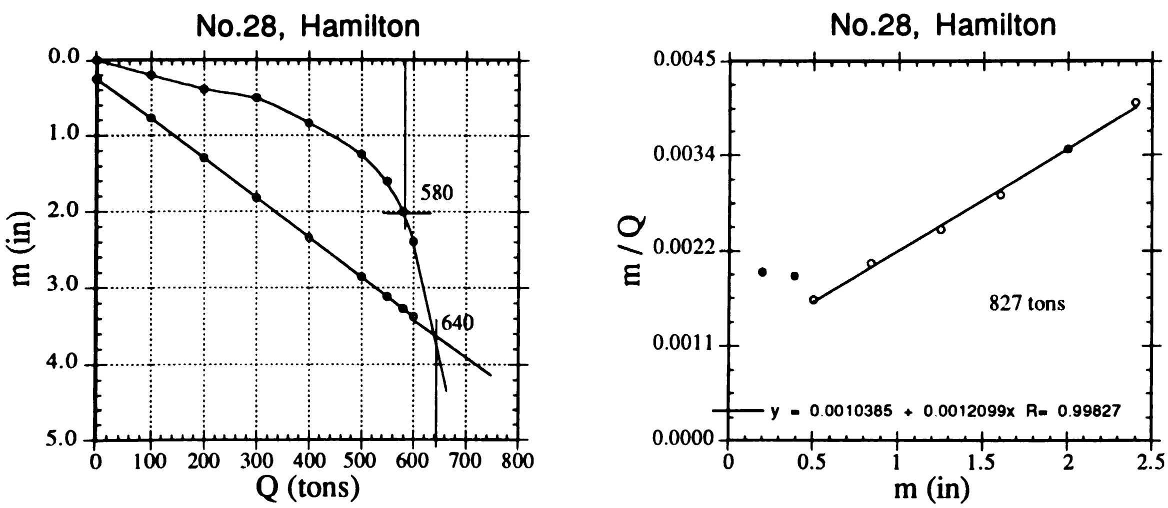
Fig. A29 Measured Capacities - Pipe Pile, No.29.



(a) Criteria [2"] & [D]

(b) Criteria [C]

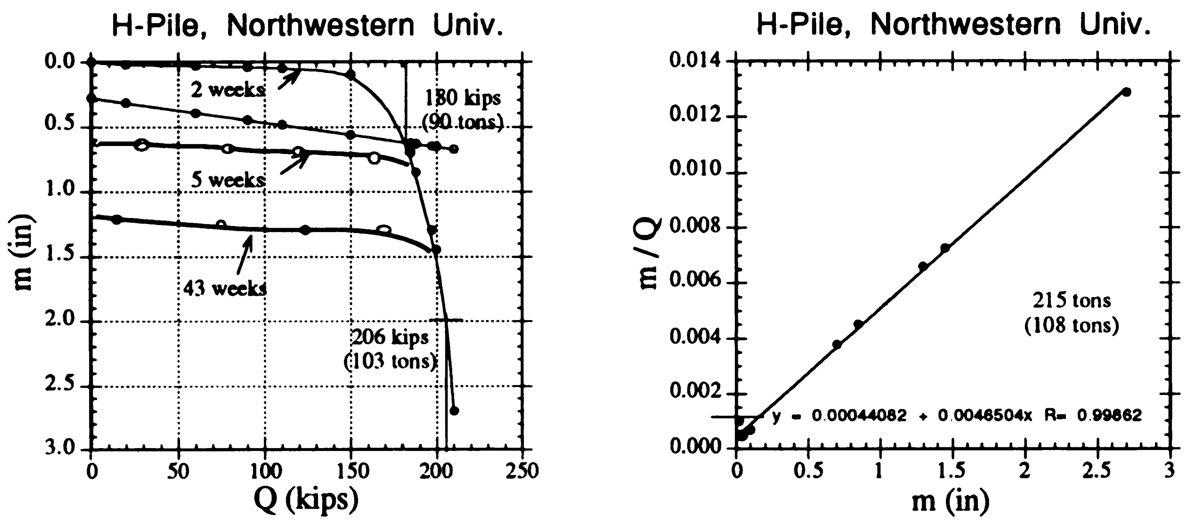
Fig. A30 Measured Capacities - Pipe Pile, No.30.



(a) Criteria [2"] & [D]

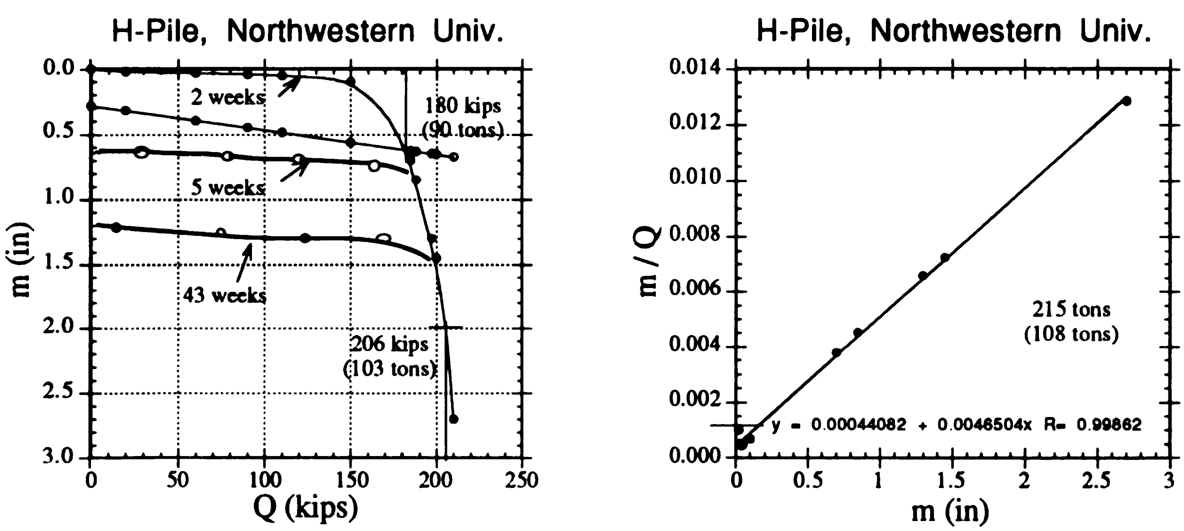
(b) Criteria [C]

Fig. A31 Measured Capacities - Pipe Pile, No.31.



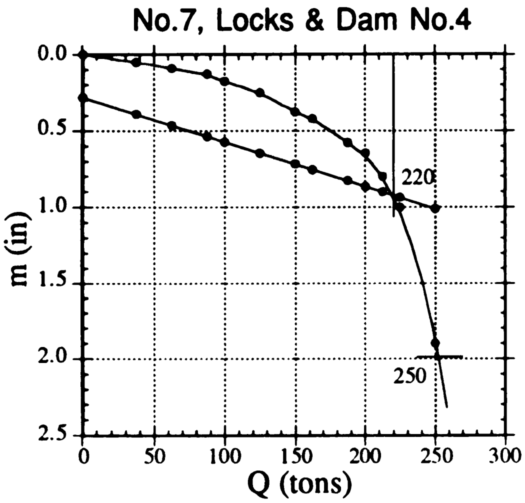
(a) Criteria [2"] & [D]　　　(b) Criteria [C]

Fig. A32 Measured Capacities - H-Pile, No.32.

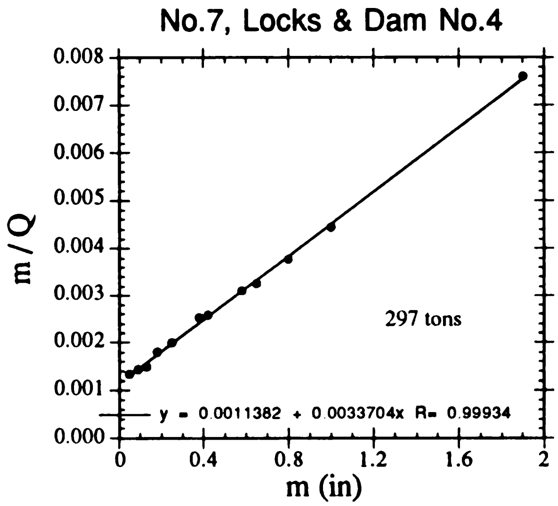


(a) Criteria [2"] & [D]　　　(b) Criteria [C]

Fig. A33 Measured Capacities - H-Pile, No.33.

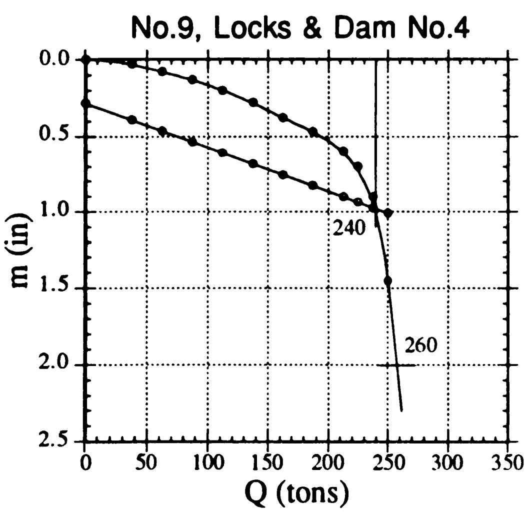


(a) Criteria [2"] & [D]

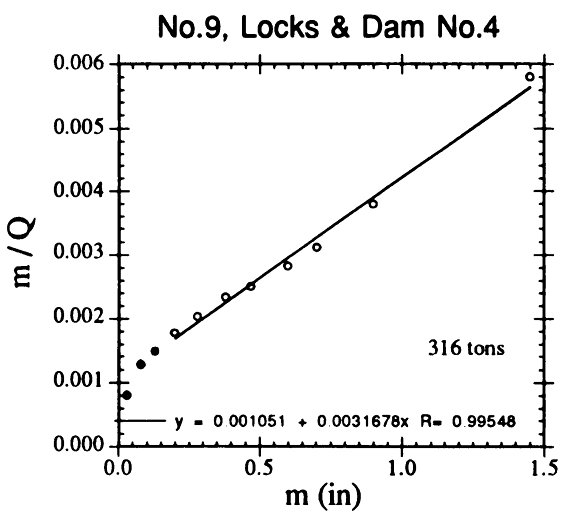


(b) Criteria [C]

Fig. A34 Measured Capacities - H Pile, No.34.

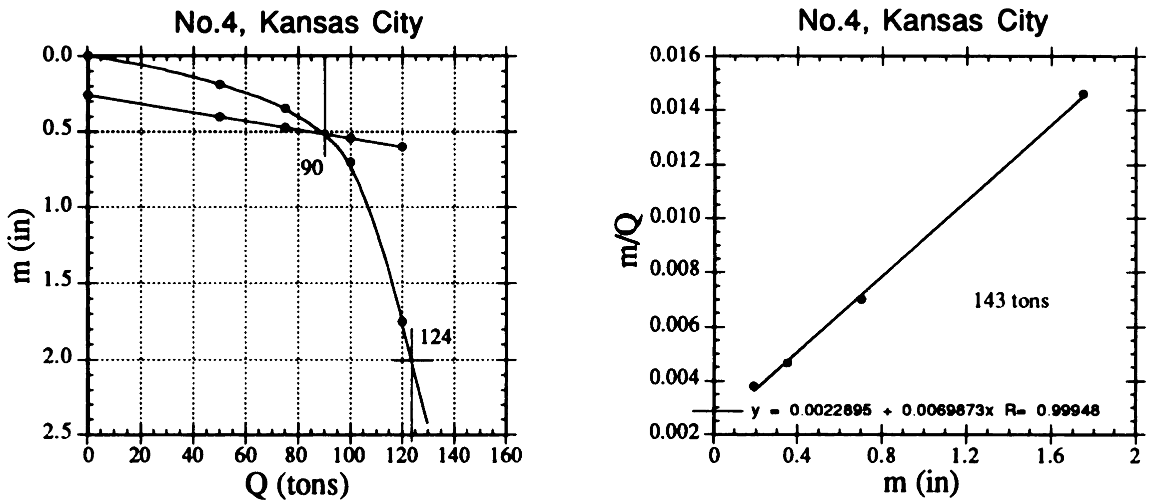


(a) Criteria [2"] & [D]



(b) Criteria [C]

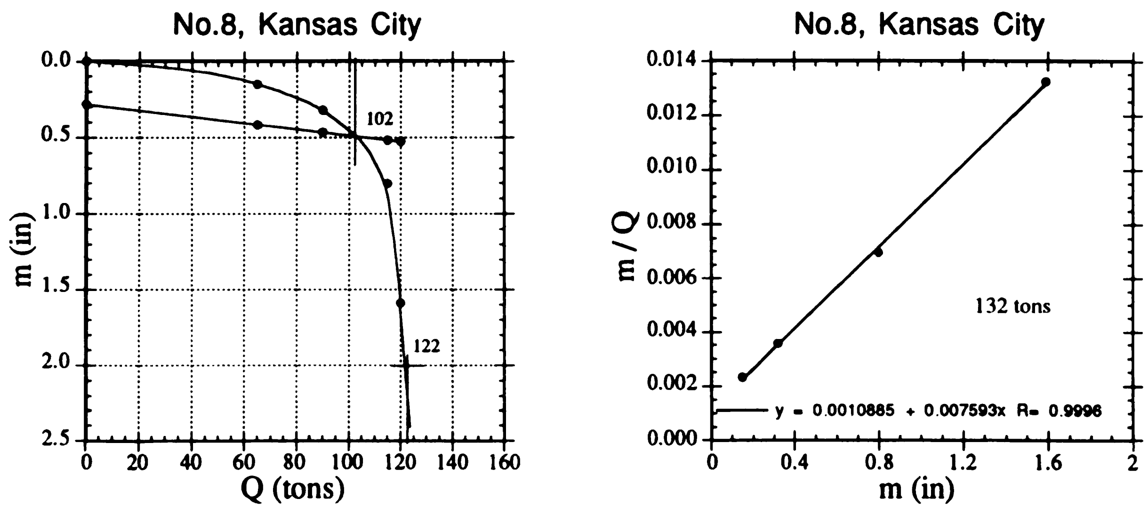
Fig. A35 Measured Capacities - H Pile, No.35.



(a) Criteria [2"] & [D]

(b) Criteria [C]

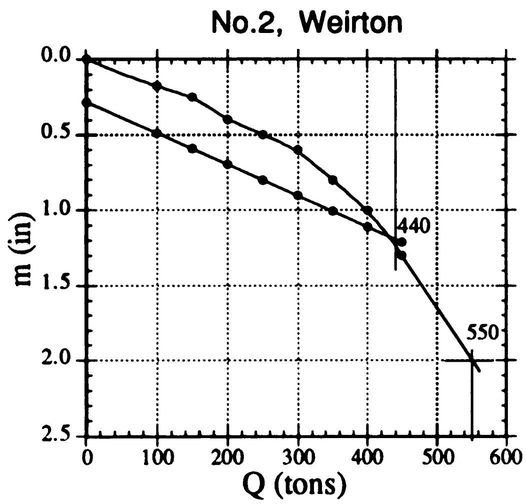
Fig. A36 Measured Capacities - H Pile, No.36.



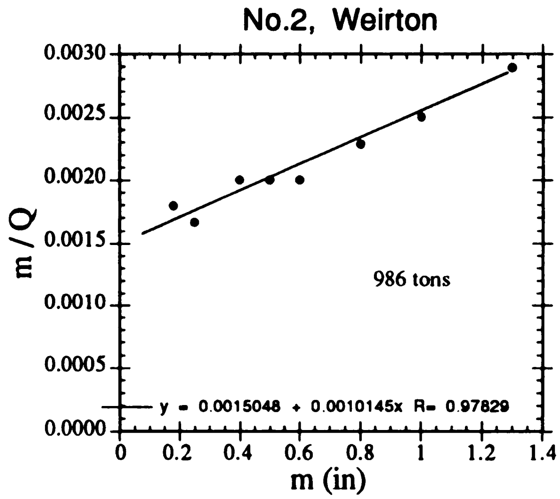
(a) Criteria [2"] & [D]

(b) Criteria [C]

Fig. A37 Measured Capacities - H Pile, No.37.

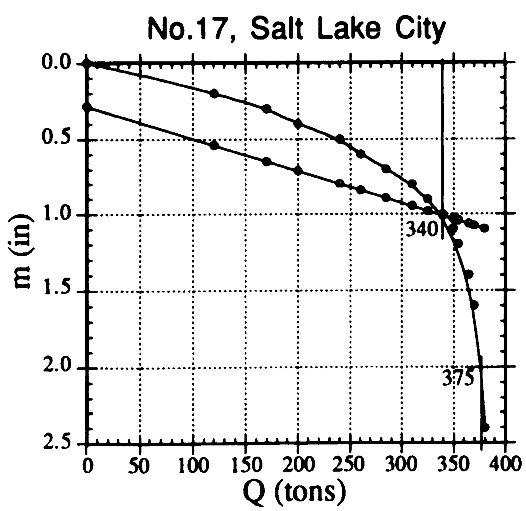


(a) Criteria [2"] & [D]

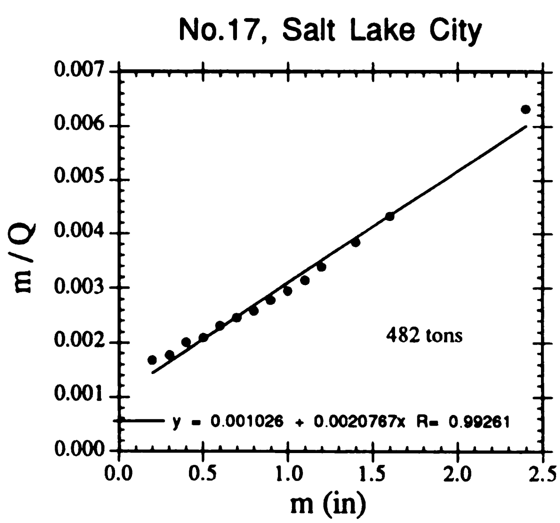


(b) Criteria [C]

Fig. A38 Measured Capacities - H Pile, No.38.

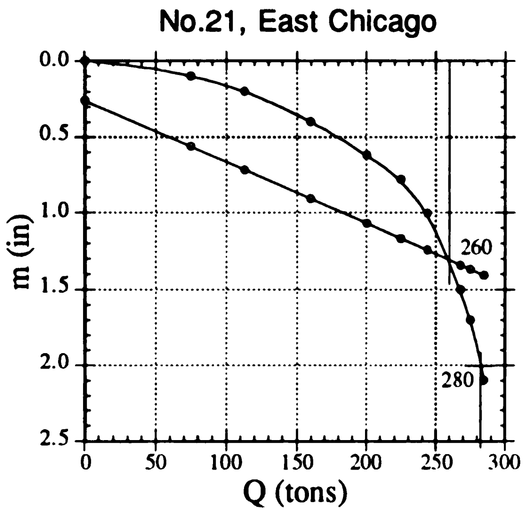


(a) Criteria [2"] & [D]

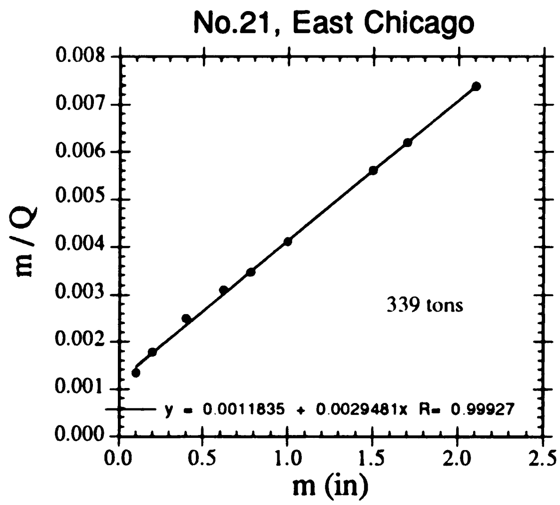


(b) Criteria [C]

Fig. A39 Measured Capacities - H Pile, No.39.

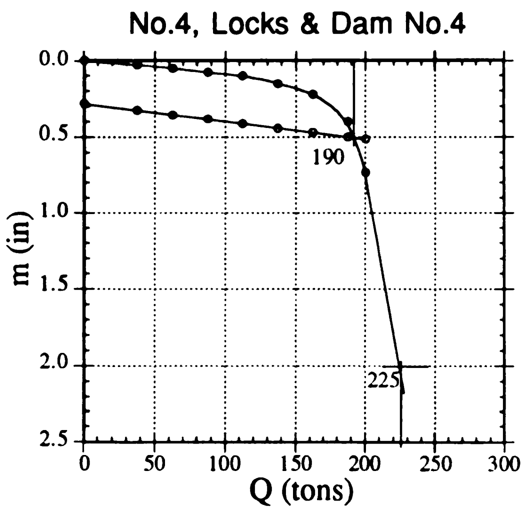


(a) Criteria [2"] & [D]

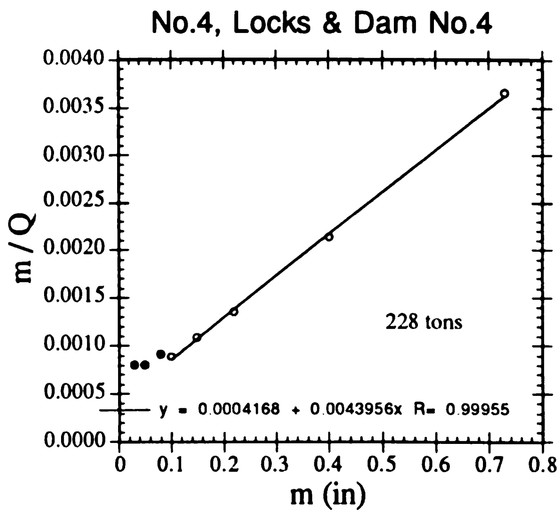


(b) Criteria [C]

Fig. A40 Measured Capacities - H Pile, No.40.

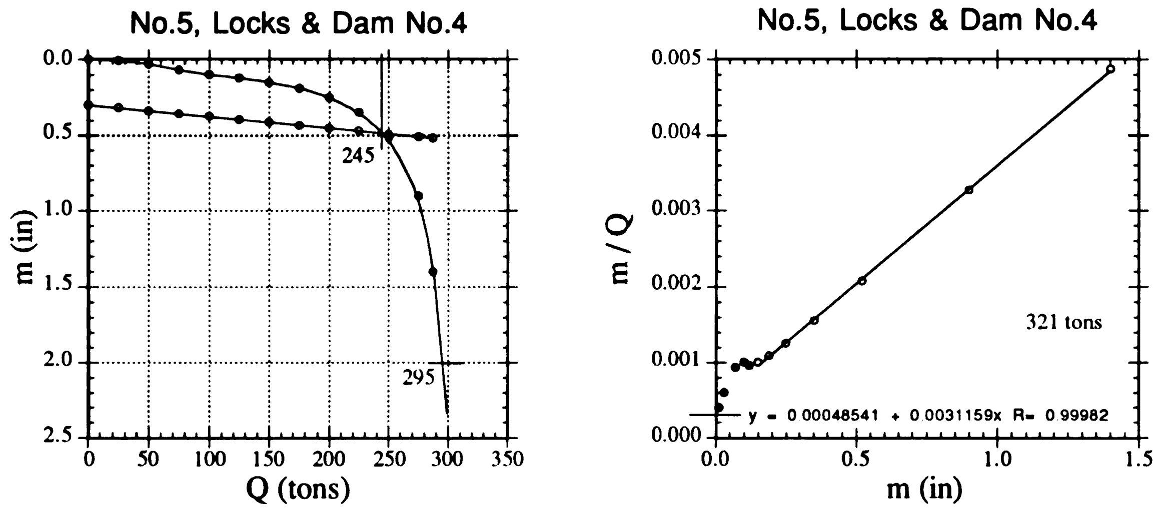


(a) Criteria [2"] & [D]



(b) Criteria [C]

Fig. A41 Measured Capacities - Concrete Pile, No.41.



(a) Criteria [2"] & [D]

(b) Criteria [C]

Fig. A42 Measured Capacities - Concrete Pile, No.42.

APPENDIX B

Table B: A Typical Spreadsheet Calculation, Pipe Pile No.26

Ta

Dep

(ft

46
47
48
49
50
51
52
53
54
55

Table B1: Spreadsheet Calculation, Pipe Pile No.26

Depth (ft.)	SPT N No 26	γ_w	γ'	$[p'_{ov}]_i$ (psf)	N'	ϕ' (deg)	f_ϕ	δ (deg)	K_s	F_s [2"]	F_s [D]	F_s [C]	F_s [DC]	β^* cal
0														
1	*0*	0.00	120.00	60	0	27	0.76	20	.654	3.52	4.13	9.42	5.77	0.24
2	3	0.00	120.00	180	9	30	0.76	23	.617	3.51	4.09	9.19	5.69	0.26
3	7	0.00	120.00	300	16	32	0.76	24	.586	3.51	4.05	8.96	5.60	0.27
4	*10*	0.00	120.00	420	20	33	0.76	25	.572	3.50	4.02	8.74	5.51	0.27
5	11	0.00	120.00	540	20	33	0.76	25	.573	3.50	3.98	8.53	5.43	0.27
6	11	0.00	120.00	660	18	33	0.76	25	.579	3.49	3.94	8.32	5.35	0.27
7	12	0.00	120.00	780	19	33	0.76	25	.579	3.49	3.91	8.11	5.27	0.27
8	*12*	0.00	120.00	900	17	32	0.76	25	.583	3.48	3.87	7.91	5.19	0.27
9	10	0.00	120.00	1020	14	31	0.76	24	.597	3.48	3.84	7.72	5.11	0.26
10	*8*	0.00	120.00	1140	10	30	0.76	23	.610	3.47	3.81	7.53	5.03	0.26
11	8	0.00	120.00	1260	10	30	0.76	23	.612	3.46	3.77	7.35	4.96	0.26
12	7	0.00	120.00	1380	8	30	0.76	22	.619	3.46	3.74	7.17	4.88	0.26
13	7	0.00	120.00	1500	8	29	0.76	22	.620	3.45	3.70	6.99	4.81	0.25
14	6	0.00	120.00	1620	7	29	0.76	22	.626	3.45	3.67	6.82	4.73	0.25
15	6	0.00	120.00	1740	6	29	0.76	22	.626	3.44	3.64	6.65	4.66	0.25
16	6	0.00	120.00	1860	6	29	0.76	22	.627	3.44	3.61	6.49	4.59	0.25
17	5	0.00	120.00	1980	5	28	0.76	22	.632	3.43	3.57	6.33	4.52	0.25
18	5	0.00	120.00	2100	5	28	0.76	22	.633	3.43	3.54	6.17	4.45	0.25
19	4	0.00	120.00	2220	4	28	0.76	21	.637	3.42	3.51	6.02	4.39	0.25
20	*4*	0.00	120.00	2340	4	28	0.76	21	.638	3.42	3.48	5.87	4.32	0.25
21	4	0.00	120.00	2460	4	28	0.76	21	.638	3.41	3.45	5.73	4.25	0.25
22	5	0.00	120.00	2580	4	28	0.76	21	.635	3.40	3.42	5.59	4.19	0.25
23	5	0.00	120.00	2700	4	28	0.76	21	.635	3.40	3.39	5.45	4.13	0.25
24	6	0.00	120.00	2820	5	28	0.76	22	.632	3.39	3.36	5.32	4.06	0.25
25	*6*	0.00	120.00	2940	5	28	0.76	22	.632	3.39	3.33	5.19	4.00	0.25
26	6	0.00	120.00	3060	5	28	0.76	22	.633	3.38	3.30	5.06	3.94	0.25
27	5	0.00	120.00	3180	4	28	0.76	21	.637	3.38	3.27	4.93	3.88	0.25
28	5	0.00	120.00	3300	4	28	0.76	21	.637	3.37	3.24	4.81	3.82	0.25
29	4	0.00	120.00	3420	3	28	0.76	21	.640	3.37	3.21	4.69	3.77	0.25
30	*4*	0.00	120.00	3540	3	28	0.76	21	.641	3.36	3.18	4.58	3.71	0.25
31	6	0.00	120.00	3660	4	28	0.76	21	.635	3.36	3.15	4.47	3.65	0.25
32	8	0.00	120.00	3780	6	29	0.76	22	.629	3.35	3.12	4.36	3.60	0.25
33	10	0.00	120.00	3900	7	29	0.76	22	.623	3.35	3.10	4.25	3.54	0.25
34	12	0.00	120.00	4020	8	30	0.76	22	.618	3.34	3.07	4.15	3.49	0.26
35	14	0.00	120.00	4140	10	30	0.76	23	.613	3.33	3.04	4.04	3.44	0.26
36	16	0.00	120.00	4260	11	30	0.76	23	.608	3.33	3.01	3.94	3.38	0.26
37	18	0.00	120.00	4380	12	31	0.76	23	.603	3.32	2.99	3.85	3.33	0.26
38	20	0.00	120.00	4500	13	31	0.76	24	.599	3.32	2.96	3.75	3.28	0.26
39	22	0.00	120.00	4620	14	31	0.76	24	.594	3.31	2.93	3.66	3.23	0.26
40	*24*	0.00	120.00	4740	15	32	0.76	24	.590	3.31	2.91	3.57	3.18	0.27
41	22	0.00	120.00	4860	14	31	0.76	24	.596	3.30	2.88	3.48	3.14	0.26
42	19	0.00	120.00	4980	12	31	0.76	23	.604	3.30	2.86	3.40	3.09	0.26
43	17	0.00	120.00	5100	11	30	0.76	23	.609	3.29	2.83	3.31	3.04	0.26
44	14	0.00	120.00	5220	9	30	0.76	23	.617	3.29	2.81	3.23	2.99	0.26
45	*12*	0.00	120.00	5340	7	29	0.76	22	.622	3.28	2.78	3.15	2.95	0.25
46	12	0.00	120.00	5460	7	29	0.76	22	.623	3.28	2.76	3.08	2.90	0.25
47	11	0.00	120.00	5580	7	29	0.76	22	.626	3.27	2.73	3.00	2.86	0.25
48	11	0.00	120.00	5700	6	29	0.76	22	.626	3.27	2.71	2.93	2.82	0.25
49	10	0.00	120.00	5820	6	29	0.76	22	.628	3.26	2.68	2.85	2.77	0.25
50	10	0.00	120.00	5940	6	29	0.76	22	.629	3.26	2.66	2.78	2.73	0.25
51	10	0.00	120.00	6060	6	29	0.76	22	.629	3.25	2.64	2.72	2.69	0.25
52	9	0.00	120.00	6180	5	28	0.76	22	.632	3.25	2.61	2.65	2.65	0.25
53	9	0.00	120.00	6300	5	28	0.76	22	.632	3.24	2.59	2.58	2.61	0.25
54	8	0.00	120.00	6420	4	28	0.76	21	.634	3.24	2.57	2.52	2.57	0.25
55	*8*	0.00	120.00	6540	4	28	0.76	21	.635	3.23	2.54	2.46	2.53	0.25

Table B1: Continued.

β^* [2"]	β^* [D]	β^* [C]	β^* [DC]	$[q_s]_i$ cal (Tons)	$[q_s]_i$ [2"]	$[q_s]_i$ [D]	$[q_s]_i$ [C]	$[q_s]_i$ [DC]	Q_{sc} cal (Tons)	Q_{sp} [2"]	Q_{sp} [D]
0.85	1.00	2.27	1.39	0.02	0.09	0.10	0.23	0.14	0.02	0.09	0.10
0.90	1.05	2.35	1.45	0.08	0.27	0.31	0.71	0.44	0.10	0.36	0.41
0.93	1.08	2.39	1.49	0.13	0.47	0.54	1.20	0.75	0.23	0.82	0.95
0.95	1.09	2.37	1.49	0.19	0.67	0.76	1.66	1.05	0.42	1.49	1.72
0.95	1.08	2.31	1.47	0.24	0.85	0.97	2.08	1.32	0.67	2.34	2.69
0.94	1.06	2.23	1.44	0.30	1.03	1.17	2.46	1.58	0.96	3.37	3.85
0.94	1.05	2.18	1.42	0.35	1.22	1.37	2.84	1.84	1.31	4.59	5.22
0.93	1.04	2.12	1.39	0.40	1.40	1.56	3.18	2.08	1.72	5.99	6.78
0.91	1.01	2.03	1.34	0.45	1.55	1.72	3.45	2.29	2.16	7.55	8.50
0.90	0.98	1.95	1.30	0.49	1.71	1.87	3.70	2.47	2.65	9.25	10.37
0.89	0.97	1.89	1.28	0.54	1.88	2.04	3.98	2.68	3.20	11.13	12.41
0.88	0.95	1.83	1.25	0.59	2.03	2.20	4.21	2.87	3.78	13.16	14.61
0.88	0.94	1.78	1.22	0.64	2.20	2.36	4.46	3.07	4.42	15.36	16.97
0.87	0.93	1.72	1.20	0.68	2.35	2.51	4.66	3.23	5.10	17.72	19.48
0.87	0.92	1.68	1.18	0.73	2.52	2.67	4.87	3.42	5.84	20.24	22.15
0.87	0.91	1.63	1.16	0.78	2.69	2.82	5.07	3.59	6.62	22.93	24.97
0.86	0.89	1.58	1.13	0.83	2.84	2.95	5.23	3.74	7.45	25.76	27.92
0.86	0.89	1.54	1.11	0.88	3.00	3.10	5.40	3.90	8.32	28.76	31.02
0.85	0.87	1.49	1.09	0.92	3.14	3.23	5.53	4.03	9.24	31.91	34.25
0.85	0.86	1.46	1.07	0.97	3.31	3.37	5.68	4.18	10.21	35.21	37.62
0.84	0.85	1.42	1.05	1.02	3.47	3.51	5.83	4.33	11.23	38.68	41.12
0.85	0.85	1.39	1.04	1.07	3.65	3.67	5.99	4.49	12.30	42.33	44.79
0.85	0.84	1.36	1.03	1.12	3.81	3.80	6.11	4.63	13.42	46.15	48.59
0.85	0.84	1.33	1.02	1.18	4.00	3.95	6.26	4.78	14.60	50.14	52.54
0.85	0.83	1.30	1.00	1.23	4.16	4.08	6.36	4.91	15.82	54.30	56.62
0.85	0.82	1.26	0.98	1.28	4.32	4.21	6.45	5.03	17.10	58.61	60.83
0.84	0.81	1.23	0.96	1.32	4.45	4.31	6.50	5.12	18.42	63.06	65.14
0.84	0.80	1.19	0.95	1.37	4.61	4.43	6.58	5.23	19.78	67.67	69.57
0.83	0.79	1.16	0.93	1.41	4.74	4.52	6.61	5.30	21.19	72.42	74.09
0.83	0.78	1.13	0.91	1.46	4.90	4.64	6.68	5.41	22.65	77.32	78.73
0.84	0.79	1.11	0.91	1.52	5.11	4.80	6.80	5.56	24.17	82.42	83.52
0.84	0.79	1.10	0.90	1.59	5.31	4.96	6.91	5.70	25.76	87.74	88.48
0.85	0.79	1.08	0.90	1.65	5.52	5.11	7.01	5.84	27.41	93.26	93.59
0.85	0.78	1.06	0.89	1.71	5.73	5.26	7.11	5.98	29.12	98.98	98.85
0.86	0.78	1.04	0.88	1.78	5.93	5.41	7.19	6.11	30.90	104.92	104.26
0.86	0.78	1.02	0.88	1.84	6.13	5.55	7.27	6.23	32.74	111.05	109.81
0.87	0.78	1.00	0.87	1.91	6.34	5.70	7.33	6.35	34.65	117.39	115.51
0.87	0.78	0.98	0.86	1.97	6.54	5.83	7.39	6.47	36.62	123.92	121.34
0.87	0.77	0.97	0.85	2.03	6.74	5.97	7.45	6.57	38.65	130.67	127.31
0.88	0.77	0.95	0.84	2.10	6.94	6.10	7.49	6.68	40.75	137.61	133.42
0.87	0.76	0.92	0.83	2.14	7.06	6.16	7.44	6.70	42.89	144.66	139.57
0.86	0.74	0.89	0.80	2.17	7.14	6.19	7.36	6.69	45.05	151.81	145.76
0.85	0.73	0.86	0.79	2.20	7.25	6.23	7.30	6.70	47.26	159.06	152.00
0.84	0.72	0.83	0.77	2.23	7.33	6.25	7.20	6.67	49.48	166.38	158.25
0.83	0.71	0.80	0.75	2.26	7.43	6.29	7.13	6.67	51.75	173.81	164.54
0.83	0.70	0.78	0.74	2.31	7.58	6.37	7.11	6.71	54.06	181.39	170.92
0.83	0.69	0.76	0.72	2.35	7.70	6.43	7.06	6.73	56.41	189.08	177.34
0.82	0.68	0.74	0.71	2.40	7.85	6.50	7.03	6.77	58.81	196.93	183.85
0.82	0.67	0.72	0.70	2.44	7.97	6.56	6.97	6.78	61.26	204.90	190.40
0.82	0.67	0.70	0.69	2.49	8.12	6.63	6.94	6.81	63.75	213.02	197.03
0.82	0.66	0.68	0.68	2.54	8.26	6.70	6.90	6.84	66.29	221.28	203.73
0.81	0.65	0.66	0.66	2.58	8.38	6.74	6.84	6.84	68.87	229.66	210.47
0.81	0.65	0.65	0.65	2.63	8.53	6.81	6.80	6.87	71.50	238.19	217.29
0.81	0.64	0.63	0.64	2.67	8.64	6.85	6.73	6.86	74.17	246.83	224.14
0.80	0.63	0.61	0.63	2.72	8.79	6.92	6.69	6.89	76.89	255.62	231.05

Table B1: Continued.

Q_{sp} [C]	Q_{sp} [DC]	p_t' toe (psf)	N_q^*	Q_{tc} cal (Tons)	F_t [2"]	F_t [D]	F_t [C]	F_t [DC]	Q_{tp} [2"]	Q_{tp} [D]	Q_{tp} [C]	Q_{tp} [DC]
0.23	0.14	120	6	0.37	11.07	6.25	15.02	11.00	4.04	2.28	5.48	4.02
0.93	0.58	240	9	1.03	10.57	6.02	14.26	10.50	10.93	6.23	14.74	10.85
2.13	1.32	360	13	2.15	10.09	5.80	13.53	10.02	21.65	12.45	29.03	21.49
3.79	2.37	480	15	3.37	9.63	5.59	12.85	9.56	32.47	18.86	43.30	32.22
5.87	3.69	600	15	4.16	9.20	5.39	12.19	9.12	38.22	22.40	50.67	37.91
8.33	5.28	720	14	4.63	8.78	5.19	11.57	8.70	40.68	24.07	53.63	40.33
11.17	7.12	840	14	5.45	8.38	5.01	10.99	8.31	45.66	27.27	59.84	45.25
14.35	9.20	960	13	5.92	8.00	4.82	10.43	7.93	47.38	28.57	61.74	46.94
17.80	11.49	1080	11	5.71	7.64	4.65	9.90	7.56	43.59	26.53	56.48	43.17
21.50	13.96	1200	10	5.54	7.29	4.48	9.39	7.22	40.38	24.81	52.02	39.97
25.48	16.65	1320	10	5.97	6.96	4.32	8.92	6.89	41.58	25.78	53.25	41.14
29.70	19.52	1440	9	6.10	6.65	4.16	8.46	6.57	40.56	25.39	51.65	40.11
34.15	22.58	1560	9	6.52	6.35	4.01	8.03	6.27	41.40	26.16	52.42	40.93
38.81	25.82	1680	8	6.64	6.06	3.86	7.63	5.99	40.24	25.67	50.66	39.77
43.68	29.23	1800	8	7.05	5.78	3.72	7.24	5.71	40.80	26.27	51.06	40.30
48.75	32.82	1920	8	7.46	5.52	3.59	6.87	5.45	41.21	26.78	51.27	40.68
53.98	36.56	2040	8	7.56	5.27	3.46	6.52	5.20	39.86	26.15	49.32	39.34
59.39	40.46	2160	8	7.96	5.03	3.33	6.19	4.97	40.07	26.54	49.29	39.53
64.92	44.49	2280	7	8.05	4.80	3.21	5.88	4.74	38.69	25.86	47.31	38.15
70.60	48.67	2400	7	8.44	4.59	3.10	5.58	4.52	38.73	26.14	47.09	38.18
76.43	53.00	2520	7	8.83	4.38	2.98	5.29	4.31	38.69	26.35	46.77	38.12
82.42	57.49	2640	8	9.55	4.18	2.87	5.03	4.12	39.94	27.46	48.00	39.33
88.54	62.12	2760	8	9.95	3.99	2.77	4.77	3.93	39.71	27.56	47.45	39.09
94.79	66.90	2880	8	10.70	3.81	2.67	4.53	3.75	40.76	28.56	48.43	40.11
101.16	71.81	3000	8	11.10	3.64	2.57	4.30	3.58	40.37	28.55	47.69	39.71
107.61	76.84	3120	8	11.50	3.47	2.48	4.08	3.41	39.93	28.51	46.90	39.26
114.11	81.96	3240	7	11.53	3.32	2.39	3.87	3.26	38.22	27.54	44.63	37.56
120.69	87.18	3360	7	11.92	3.17	2.30	3.68	3.11	37.73	27.45	43.81	37.07
127.31	92.49	3480	7	11.94	3.02	2.22	3.49	2.97	36.09	26.50	41.67	35.44
133.98	97.89	3600	7	12.33	2.89	2.14	3.31	2.83	35.57	26.37	40.83	34.92
140.78	103.45	3720	8	13.49	2.75	2.06	3.14	2.70	37.15	27.79	42.39	36.45
147.69	109.16	3840	8	14.71	2.63	1.99	2.98	2.58	38.69	29.23	43.90	37.95
154.70	115.00	3960	8	16.02	2.51	1.91	2.83	2.46	40.21	30.66	45.36	39.42
161.81	120.98	4080	9	17.40	2.40	1.84	2.69	2.35	41.70	32.09	46.77	40.86
169.00	127.09	4200	9	18.86	2.29	1.78	2.55	2.24	43.14	33.52	48.11	42.26
176.27	133.32	4320	10	20.40	2.18	1.71	2.42	2.14	44.56	34.94	49.40	43.63
183.60	139.67	4440	10	22.02	2.09	1.65	2.30	2.04	45.93	36.35	50.63	44.95
190.99	146.14	4560	11	23.73	1.99	1.59	2.18	1.95	47.25	37.76	51.79	46.23
198.44	152.71	4680	11	25.53	1.90	1.53	2.07	1.86	48.54	39.15	52.89	47.47
205.93	159.39	4800	12	27.43	1.81	1.48	1.97	1.77	49.77	40.53	53.93	48.66
213.37	166.09	4920	11	26.45	1.73	1.42	1.87	1.69	45.83	37.66	49.36	44.78
220.73	172.78	5040	10	24.88	1.65	1.37	1.77	1.62	41.15	34.14	44.08	40.20
228.03	179.47	5160	10	24.06	1.58	1.32	1.68	1.54	37.99	31.82	40.46	37.09
235.23	186.15	5280	9	22.71	1.51	1.27	1.60	1.47	34.24	28.94	36.25	33.41
242.37	192.82	5400	9	22.02	1.44	1.23	1.51	1.40	31.69	27.04	33.35	30.91
249.48	199.53	5520	9	22.43	1.37	1.18	1.44	1.34	30.82	26.55	32.26	30.06
256.54	206.26	5640	8	22.30	1.31	1.14	1.36	1.28	29.25	25.44	30.44	28.51
263.57	213.03	5760	8	22.71	1.25	1.10	1.30	1.22	28.44	24.97	29.43	27.71
270.54	219.81	5880	8	22.58	1.20	1.06	1.23	1.16	27.00	23.92	27.77	26.30
277.48	226.62	6000	8	22.99	1.14	1.02	1.17	1.11	26.24	23.47	26.84	25.55
284.39	233.46	6120	8	23.40	1.09	0.98	1.11	1.06	25.49	23.02	25.92	24.81
291.23	240.30	6240	8	23.26	1.04	0.95	1.05	1.01	24.20	22.06	24.47	23.54
298.03	247.17	6360	8	23.67	0.99	0.91	1.00	0.97	23.50	21.62	23.62	22.85
304.76	254.03	6480	8	23.53	0.95	0.88	0.95	0.92	22.31	20.72	22.30	21.69
311.45	260.92	6600	8	23.93	0.91	0.85	0.90	0.88	21.66	20.30	21.52	21.04

Table B1: Continued.

Calculated (Formula)				RELIABILITY APPROACH			Deterministic Equivalent	
Qsc	Qtc	Qc	Qtc/Qc					
77	24	101	0.24					
Predicted (Algorithm)				Known Site s	Provide β	Qa	CFS	Equiv. FS
Qsp	Qtp	Qp						
256	22	277	[2"]	0.12	2.00	**71**	1.81	**1.81**
231	20	251	[D]	0.12	3.00	**33**	2.38	**2.38**
311	22	333	[C]	0.11	2.50	**62**	1.95	**1.95**
261	21	282	[DC]	0.10	3.00	**50**	2.05	**2.05**
Measured (Loading Test)			CL Fb	Unknown Site s	Provide β	Qa	CFS	Equiv. FS
Qsm	Qtm	(Qt/Q)						
88	27	0.23	0.461	0.25	1.50	**46**	2.80	**2.80**
62	18	0.22	0.312	0.30	1.25	**26**	3.01	**3.01**
90	29	0.24	0.361	0.27	1.25	**46**	2.64	**2.64**
76	24	0.24	0.363	0.27	1.25	**39**	2.64	**2.64**

	Qp*Fb/Qm	Qm	Qp*Fb	DETERMINISTIC APPROACH Known Site Provide FS	Qa	Known Site s	Reliability-Based Equivalent CFS	Equiv. β
[2"]	1.11	*115*	**128**	2.00	**64**	0.12	2.00	**2.37**
[D]	0.98	*80*	**78**	2.50	**31**	0.12	2.50	**3.18**
[C]	1.01	*119*	**120**	2.50	**48**	0.11	2.50	**3.49**
[DC]	1.03	100	**102**	2.00	**51**	0.10	2.00	**2.90**

STEEL PIPE PILE				Unknown Site Provide FS	Qa	Unknown Site s	CFS	Equiv. β
OD	*12.75*	(in)						
t	*0.19*	(in)						
[As]i	3.34	(sq.ft)	[2"]	3.00	**43**	0.25	3.00	**1.62**
A steel	7.42	(sq in)	[D]	3.00	**26**	0.30	3.00	**1.25**
A toe	0.89	(sq.ft)	[C]	3.00	**40**	0.27	3.00	**1.46**
Le	55	(ft)	[DC]	3.00	**34**	0.27	3.00	**1.46**

NOTE:		NLT Fb	RELIABILITY APPROACH Known Site s	Provide β	Qa	Deterministic Equivalent CFS	Equiv. FS
Input	*Italics*						
Output	**Bold**						
	[2"]	1.136	0.12	2.00	**174**	1.81	**1.81**
	[D]	0.978	0.12	3.00	**103**	2.38	**2.38**
	[C]	0.863	0.11	2.50	**148**	1.95	**1.95**
	[DC]	0.952	0.10	3.00	**131**	2.05	**2.05**

APPENDIX C

Table C: Determination of Allowable Capacity

Table C24: Pipe Pile No.24 (Automatic Trip Hammer).

Calculated (Formula)

Qsc	Qtc	Qc	Qtc/Qc
64	22	85	0.25

Predicted (Algorithm) — RELIABILITY APPROACH / Deterministic Equivalent (Uniform Site)

Qsp	Qtp	Qp		Uniform Site s	Provide β	Qa	CFS	Equiv. FS
216	49	265	[2"]	0.12	2.00	**68**	1.81	**1.81**
217	38	255	[D]	0.12	3.00	**33**	2.38	**2.38**
358	54	412	[C]	0.11	2.50	**76**	1.95	**1.95**
266	48	314	[DC]	0.10	3.00	**56**	2.05	**2.05**

Measured (Loading Test)

Qsm	Qtm	(Qt/Q)	CL Fb	Non-Unif.. Site s	Provide β	Qa	CFS	Equiv. FS
59	41	0.41	0.461	0.25	1.50	**44**	2.80	**2.80**
36	24	0.39	0.312	0.30	1.25	**26**	3.01	**3.01**
70	49	0.41	0.361	0.27	1.25	**56**	2.64	**2.64**
52	37	0.41	0.363	0.27	1.25	**43**	2.64	**2.64**

DETERMINISTIC APPROACH / Reliability-Based Equivalent

	Qp*Fb/Qm	Qm	Qp*Fb	Uniform Site Provide FS	Qa	Uniform Site s	CFS	Equiv. β
[2"]	1.22	*100*	**122**	2.00	**61**	0.12	2.00	**2.37**
[D]	1.32	*60*	**79**	2.50	**32**	0.12	2.50	**3.18**
[C]	1.25	*119*	**149**	2.50	**60**	0.11	2.50	**3.49**
[DC]	1.27	90	**114**	2.00	**57**	0.10	2.00	**2.90**

	Non-Unif.. Site Provide FS	Qa	Non-Unif.. Site s	CFS	Equiv. β
[2"]	3.00	**41**	0.25	3.00	**1.62**
[D]	3.00	**26**	0.30	3.00	**1.25**
[C]	3.00	**50**	0.27	3.00	**1.46**
[DC]	3.00	**38**	0.27	3.00	**1.46**

STEEL PIPE PILE

OD	*18.00*	(in)
t	*0.38*	(in)
[As]i	4.71	(sq.ft)
A steel	20.76	(sq in)
A toe	1.77	(sq.ft)
Le	50	(ft)

NOTE:
Input *Italics*
Output **Bold**
CL -Constant Load test mtd

SPT N-VALUES ONLY (Unknown Loading Test, NLT) — RELIABILITY APPROACH / Deterministic Equivalent

	NLT Fb	Non-Unif.. Site s	Provide β	Qa	CFS	Equiv. FS
[2"]	1.136	0.25	1.50	**108**	2.80	**2.80**
[D]	0.978	0.30	1.25	**83**	3.01	**3.01**
[C]	0.863	0.27	1.25	**135**	2.64	**2.64**
[DC]	0.952	0.27	1.25	**113**	2.64	**2.64**

Table C25: Pipe Pile No.25 (Safety Hammer).

Calculated (Formula)				RELIABILITY APPROACH			Deterministic Equivalent	
Qsc	Qtc	Qc	Qtc/Qc					
65	22	87	0.25					
Predicted (Algorithm)				Uniform Site	Provide			Equiv.
Qsp	Qtp	Qp		s	β	Qa	CFS	FS
221	49	270	[2"]	0.12	2.00	**69**	1.81	**1.81**
221	38	260	[D]	0.12	3.00	**34**	2.38	**2.38**
367	54	421	[C]	0.11	2.50	**78**	1.95	**1.95**
272	48	320	[DC]	0.10	3.00	**57**	2.05	**2.05**
Measured (Loading Test)			CL	Non-Unif. Site	Provide			Equiv.
Qsm	Qtm	(Qt/Q)	Fb	s	β	Qa	CFS	FS
59	41	0.41	0.461	0.25	1.50	**44**	2.80	**2.80**
36	24	0.39	0.312	0.30	1.25	**27**	3.01	**3.01**
70	49	0.41	0.361	0.27	1.25	**58**	2.64	**2.64**
52	37	0.41	0.363	0.27	1.25	**44**	2.64	**2.64**

Qp*Fb/Qm				DETERMINISTIC APPROACH			Reliability-Based Equivalent	
				Uniform Site Provide		Uniform Site		Equiv.
		Qm	Qp*Fb	FS	Qa	s	CFS	β
[2"]	1.24	*100*	**124**	2.00	**62**	0.12	2.00	**2.37**
[D]	1.35	*60*	**81**	2.50	**32**	0.12	2.50	**3.18**
[C]	1.28	*119*	**152**	2.50	**61**	0.11	2.50	**3.49**
[DC]	1.30	90	**116**	2.00	**58**	0.10	2.00	**2.90**
STEEL PIPE PILE				Non-Unif. Site Provide		Non-Unif. Site		Equiv.
				FS	Qa	s	CFS	β
OD	*18.00*	(in)	[2"]	3.00	**41**	0.25	3.00	**1.62**
t	*0.38*	(in)	[D]	3.00	**27**	0.30	3.00	**1.25**
[As]i	4.71	(sq.ft)	[C]	3.00	**51**	0.27	3.00	**1.46**
A steel	20.76	(sq in)	[DC]	3.00	**39**	0.27	3.00	**1.46**
A toe	1.77	(sq.ft)						
Le	50	(ft)						

NOTE:		SPT N-VALUES ONLY (Unknown Loading Test, NLT)				
Input *Italics*		RELIABILITY APPROACH			Deterministic Equivalent	
Output **Bold**		Uniform Site	Provide			Equiv.
CL -Constant Load test mtd	NLT Fb	s	β	Qa	CFS	FS
[2"]	1.136	0.25	1.50	**109**	2.80	**2.80**
[D]	0.978	0.30	1.25	**84**	3.01	**3.01**
[C]	0.863	0.27	1.25	**138**	2.64	**2.64**
[DC]	0.952	0.27	1.25	**115**	2.64	**2.64**

Table C26: Pipe Pile No.26.

Calculated (Formula)

Qsc	Qtc	Qc	Qtc/Qc
77	24	101	0.24

Predicted (Algorithm) / RELIABILITY APPROACH — Deterministic Equivalent

Qsp	Qtp	Qp		Uniform Site s	Provide β	Qa	CFS	Equiv. FS
256	22	277	[2"]	0.12	2.00	**71**	1.81	**1.81**
231	20	251	[D]	0.12	3.00	**33**	2.38	**2.38**
311	22	333	[C]	0.11	2.50	**62**	1.95	**1.95**
261	21	282	[DC]	0.10	3.00	**50**	2.05	**2.05**

Measured (Loading Test)

Qsm	Qtm	(Qt/Q)	CL Fb	Non-Unif. Site s	Provide β	Qa	CFS	Equiv. FS
88	27	0.23	0.461	0.25	1.50	**46**	2.80	**2.80**
62	18	0.22	0.312	0.30	1.25	**26**	3.01	**3.01**
90	29	0.24	0.361	0.27	1.25	**46**	2.64	**2.64**
76	24	0.24	0.363	0.27	1.25	**39**	2.64	**2.64**

DETERMINISTIC APPROACH — Reliability-Based Equivalent

	Qp*Fb/Qm	Qm	Qp*Fb	Uniform Site Provide FS	Qa	Uniform Site s	CFS	Equiv. β
[2"]	1.11	*115*	**128**	2.00	**64**	0.12	2.00	**2.37**
[D]	0.98	*80*	**78**	2.50	**31**	0.12	2.50	**3.18**
[C]	1.01	*119*	**120**	2.50	**48**	0.11	2.50	**3.49**
[DC]	1.03	100	**102**	2.00	**51**	0.10	2.00	**2.90**

STEEL PIPE PILE

OD	*12.75*	(in)
t	*0.19*	(in)
[As]i	3.34	(sq.ft)
A steel	7.42	(sq in)
A toe	0.89	(sq.ft)
Le	55	(ft)

	Non-Unif. Site Provide FS	Qa	Non-Unif. Site s	CFS	Equiv. β
[2"]	3.00	**43**	0.25	3.00	**1.62**
[D]	3.00	**26**	0.30	3.00	**1.25**
[C]	3.00	**40**	0.27	3.00	**1.46**
[DC]	3.00	**34**	0.27	3.00	**1.46**

NOTE:
Input *Italics*
Output **Bold**

RELIABILITY APPROACH — Deterministic Equivalent

	NLT Fb	Uniform Site s	Provide β	Qa	CFS	Equiv. FS
[2"]	1.136	0.12	2.00	**174**	1.81	**1.81**
[D]	0.978	0.12	3.00	**103**	2.38	**2.38**
[C]	0.863	0.11	2.50	**148**	1.95	**1.95**
[DC]	0.952	0.10	3.00	**131**	2.05	**2.05**

Table C27: Pipe Pile No.27.

Calculated (Formula)				RELIABILITY APPROACH			Deterministic	
Qsc	Qtc	Qc	Qtc/Qc				Equivalent	
84	27	111	0.24	Uniform				
Predicted (Algorithm)				Site	Provide			Equiv.
Qsp	Qtp	Qp		s	β	Qa	CFS	FS
282	31	313	[2"]	0.12	2.00	**80**	1.81	**1.81**
261	27	288	[D]	0.12	3.00	**38**	2.38	**2.38**
368	31	399	[C]	0.11	2.50	**74**	1.95	**1.95**
300	30	330	[DC]	0.10	3.00	**58**	2.05	**2.05**

Measured (Loading Test)			CL	Non-Unif. Site	Provide			Equiv.
Qsm	Qtm	(Qt/Q)	Fb	s	β	Qa	CFS	FS
82	30	0.27	0.461	0.25	1.50	**52**	2.80	**2.80**
70	25	0.26	0.312	0.30	1.25	**30**	3.01	**3.01**
88	33	0.28	0.361	0.27	1.25	**55**	2.64	**2.64**
78	30	0.28	0.363	0.27	1.25	**45**	2.64	**2.64**

	Qp*Fb/Qm			DETERMINISTIC APPROACH			Reliability-Based Equivalent	
				Uniform Site Provide		Uniform Site		Equiv.
		Qm	Qp*Fb	FS	Qa	s	CFS	β
[2"]	1.29	*112*	**144**	2.00	**72**	0.12	2.00	**2.37**
[D]	0.95	*95*	**90**	2.50	**36**	0.12	2.50	**3.18**
[C]	1.19	*121*	**144**	2.50	**58**	0.11	2.50	**3.49**
[DC]	1.11	108	**120**	2.00	**60**	0.10	2.00	**2.90**

STEEL PIPE PILE		
OD	*14.00*	(in)
t	*0.19*	(in)
[As]i	3.67	(sq.ft)
A steel	8.16	(sq in)
A toe	1.07	(sq.ft)
Le	55	(ft)

	Non-Unif. Site Provide		Non-Unif. Site		Equiv.
	FS	Qa	s	CFS	β
[2"]	3.00	**48**	0.25	3.00	**1.62**
[D]	3.00	**30**	0.30	3.00	**1.25**
[C]	3.00	**48**	0.27	3.00	**1.46**
[DC]	3.00	**40**	0.27	3.00	**1.46**

NOTE:		NLT	RELIABILITY APPROACH Known Site	Provide		Deterministic Equivalent	Equiv.
Input	*Italics*						
Output	**Bold**	Fb	s	β	Qa	CFS	FS
	[2"]	1.136	0.12	2.00	**197**	1.81	**1.81**
	[D]	0.978	0.12	3.00	**119**	2.38	**2.38**
	[C]	0.863	0.11	2.50	**177**	1.95	**1.95**
	[DC]	0.952	0.10	3.00	**153**	2.05	**2.05**

Table C28: Pipe Pile No.28.

Calculated (Formula)

Qsc	Qtc	Qc	Qtc/Qc
85	33	118	0.28

Predicted (Algorithm) — RELIABILITY APPROACH, Uniform Site; Deterministic Equivalent

	Qsp	Qtp	Qp	Uniform Site s	Provide β	Qa	CFS	Equiv. FS
[2"]	285	37	322	0.12	2.00	**8 2**	1.81	**1.81**
[D]	263	33	297	0.12	3.00	**3 9**	2.38	**2.38**
[C]	372	38	410	0.11	2.50	**7 6**	1.95	**1.95**
[DC]	303	36	340	0.10	3.00	**6 0**	2.05	**2.05**

Measured (Loading Test) — Non-Unif. Site

Qsm	Qtm	(Qt/Q)	CL Fb	Non-Unif. Site s	Provide β	Qa	CFS	Equiv. FS
95	35	0.27	0.461	0.25	1.50	**5 3**	2.80	**2.80**
82	28	0.26	0.312	0.30	1.25	**3 1**	3.01	**3.01**
99	38	0.28	0.361	0.27	1.25	**5 6**	2.64	**2.64**
89	34	0.28	0.363	0.27	1.25	**4 7**	2.64	**2.64**

DETERMINISTIC APPROACH — Reliability-Based Equivalent (Uniform)

	Qp*Fb/Qm	Qm	Qp*Fb	Uniform Site Provide FS	Qa	Uniform Site s	CFS	Equiv. β
[2"]	1.14	*130*	**148**	2.00	**7 4**	0.12	2.00	**2.37**
[D]	0.84	*110*	**9 3**	2.50	**3 7**	0.12	2.50	**3.18**
[C]	1.08	*137*	**148**	2.50	**5 9**	0.11	2.50	**3.49**
[DC]	1.00	124	**123**	2.00	**6 2**	0.10	2.00	**2.90**

DETERMINISTIC APPROACH — Non-Unif.

	Non-Unif. Site Provide FS	Qa	Non-Unif. Site s	CFS	Equiv. β
[2"]	3.00	**4 9**	0.25	3.00	**1.62**
[D]	3.00	**3 1**	0.30	3.00	**1.25**
[C]	3.00	**4 9**	0.27	3.00	**1.46**
[DC]	3.00	**4 1**	0.27	3.00	**1.46**

STEEL PIPE PILE

OD	*14.00*	(in)
t	*0.19*	(in)
[As]i	3.67	(sq.ft)
A steel	8.16	(sq in)
A toe	1.07	(sq.ft)
Le	55	(ft)

NOTE:
Input *Italics*
Output **Bold**

RELIABILITY APPROACH — Deterministic Equivalent

	NLT Fb	Uniform Site s	Provide β	Qa	CFS	Equiv. FS
[2"]	1.136	0.12	2.00	**203**	1.81	**1.81**
[D]	0.978	0.12	3.00	**122**	2.38	**2.38**
[C]	0.863	0.11	2.50	**182**	1.95	**1.95**
[DC]	0.952	0.10	3.00	**158**	2.05	**2.05**

Table C29: Pipe Pile No.29.

Calculated (Formula)								
Qsc	Qtc	Qc	Qtc/Qc	RELIABILITY APPROACH			Deterministic	
97	43	140	0.31				Equivalent	
				Uniform				
Predicted (Algorithm)				Site	Provide			Equiv.
Qsp	Qtp	Qp		s	β	Qa	CFS	FS
327	65	393	[2"]	0.12	2.00	**100**	1.81	**1.81**
312	55	367	[D]	0.12	3.00	**48**	2.38	**2.38**
467	69	536	[C]	0.11	2.50	**99**	1.95	**1.95**
368	64	432	[DC]	0.10	3.00	**76**	2.05	**2.05**
				Non-Unif.				
Measured (Loading Test)			CL	Site	Provide			Equiv.
Qsm	Qtm	(Qt/Q)	Fb	s	β	Qa	CFS	FS
95	45	0.32	0.461	0.25	1.50	**65**	2.80	**2.80**
80	35	0.31	0.312	0.30	1.25	**38**	3.01	**3.01**
101	49	0.33	0.361	0.27	1.25	**73**	2.64	**2.64**
89	43	0.33	0.363	0.27	1.25	**59**	2.64	**2.64**

				DETERMINISTIC APPROACH			Reliability-Based	
				Uniform			Equivalent	
				Site		Uniform		Equiv.
	Qp*Fb/Qm			Provide		Site		
		Qm	Qp*Fb	FS	Qa	s	CFS	β
[2"]	1.29	*140*	**181**	2.00	**91**	0.12	2.00	**2.37**
[D]	1.00	*115*	**115**	2.50	**46**	0.12	2.50	**3.18**
[C]	1.29	*150*	**193**	2.50	**77**	0.11	2.50	**3.49**
[DC]	1.18	133	**157**	2.00	**78**	0.10	2.00	**2.90**
				Non-Unif.				
STEEL PIPE PILE				Site		Non-Unif.		
OD	*16.00*	(in)		Provide		Site		Equiv.
t	*0.19*	(in)		FS	Qa	s	CFS	β
[As]i	4.19	(sq.ft)	[2"]	3.00	**60**	0.25	3.00	**1.62**
A steel	9.34	(sq in)	[D]	3.00	**38**	0.30	3.00	**1.25**
A toe	1.40	(sq.ft)	[C]	3.00	**64**	0.27	3.00	**1.46**
Le	55	(ft)	[DC]	3.00	**52**	0.27	3.00	**1.46**

NOTE:			RELIABILITY APPROACH			Deterministic	
Input	*Italics*		Uniform			Equivalent	
Output	**Bold**	NLT	Site	Provide			Equiv.
		Fb	s	β	Qa	CFS	FS
	[2"]	1.136	0.12	2.00	**247**	1.81	**1.81**
	[D]	0.978	0.12	3.00	**151**	2.38	**2.38**
	[C]	0.863	0.11	2.50	**238**	1.95	**1.95**
	[DC]	0.952	0.10	3.00	**201**	2.05	**2.05**

Table C30: Pipe Pile No.30.

Calculated (Formula)

Qsc	Qtc	Qc	Qtc/Qc
184	120	305	0.40

RELIABILITY APPROACH — Deterministic Equivalent

Predicted (Algorithm) Qsp	Qtp	Qp		Uniform Site s	Provide β	Qa	CFS	Equiv. FS
593	24	616	[2"]	0.12	2.00	**421**	1.81	**1.81**
464	30	494	[D]	0.12	3.00	**232**	2.38	**2.38**
483	19	502	[C]	0.11	2.50	**261**	1.95	**1.95**
468	23	491	[DC]	0.10	3.00	**259**	2.05	**2.05**

Measured (Loading Test) Qsm	Qtm	(Qt/Q)	CRP Fb	Non-Unif. Site s	Provide β	Qa	CFS	Equiv. FS
438	42	0.09	1.233	0.25	1.50	**272**	2.80	**2.80**
456	44	0.09	1.117	0.30	1.25	**183**	3.01	**3.01**
446	49	0.10	1.011	0.27	1.25	**192**	2.64	**2.64**
449	48	0.10	1.082	0.27	1.25	**201**	2.64	**2.64**

DETERMINISTIC APPROACH — Reliability-Based Equivalent

	Qp*Fb/Qm	Qm	Qp*Fb	Uniform Site Provide FS	Qa	Uniform Site s	CFS	Equiv. β
[2"]	1.58	*480*	**760**	2.00	**380**	0.12	2.00	**2.37**
[D]	1.10	*500*	**551**	2.50	**221**	0.12	2.50	**3.18**
[C]	1.03	*495*	**508**	2.50	**203**	0.11	2.50	**3.49**
[DC]	1.07	498	**531**	2.00	**266**	0.10	2.00	**2.90**

STEEL PIPE PILE				Non-Unif. Site Provide FS	Qa	Non-Unif. Site s	CFS	Equiv. β
OD	*12.75*	(in)						
t	*0.38*	(in)						
[As]i	3.34	(sq.ft)	[2"]	3.00	**253**	0.25	3.00	**1.62**
A steel	14.58	(sq in)	[D]	3.00	**184**	0.30	3.00	**1.25**
A toe	0.89	(sq.ft)	[C]	3.00	**169**	0.27	3.00	**1.46**
Le	88	(ft)	[DC]	3.00	**177**	0.27	3.00	**1.46**

NOTE:
Input *Italics*
Output **Bold**

RELIABILITY APPROACH — Deterministic Equivalent

	NLT Fb	Uniform Site s	Provide β	Qa	CFS	Equiv. FS
[2"]	1.136	0.12	2.00	**388**	1.81	**1.81**
[D]	0.978	0.12	3.00	**203**	2.38	**2.38**
[C]	0.863	0.11	2.50	**223**	1.95	**1.95**
[DC]	0.952	0.10	3.00	**228**	2.05	**2.05**

Table C31: Pipe Pile No.31.

Calculated (Formula)				RELIABILITY APPROACH			Deterministic Equivalent	
Qsc	Qtc	Qc	Qtc/Qc					
230	119	348	0.34					
Predicted (Algorithm)				Uniform Site	Provide			Equiv.
Qsp	Qtp	Qp		s	β	Qa	CFS	FS
731	17	748	[2"]	0.12	2.00	**511**	1.81	**1.81**
548	23	571	[D]	0.12	3.00	**268**	2.38	**2.38**
531	13	544	[C]	0.11	2.50	**283**	1.95	**1.95**
537	16	553	[DC]	0.10	3.00	**292**	2.05	**2.05**
Measured (Loading Test)			CRP	Non-Unif. Site	Provide			Equiv.
Qsm	Qtm	(Qt/Q)	Fb	s	β	Qa	CFS	FS
539	41	0.07	1.233	0.25	1.50	**330**	2.80	**2.80**
594	46	0.07	1.117	0.30	1.25	**212**	3.01	**3.01**
760	67	0.08	1.011	0.27	1.25	**209**	2.64	**2.64**
675	58	0.08	1.082	0.27	1.25	**227**	2.64	**2.64**

	Qp*Fb/Qm			DETERMINISTIC APPROACH			Reliability-Based Equivalent	
				Uniform Site Provide		Uniform Site		Equiv.
		Qm	Qp*Fb	FS	Qa	s	CFS	β
[2"]	1.59	*580*	**923**	2.00	**461**	0.12	2.00	**2.37**
[D]	1.00	*640*	**638**	2.50	**255**	0.12	2.50	**3.18**
[C]	0.67	*827*	**550**	2.50	**220**	0.11	2.50	**3.49**
[DC]	0.82	734	**598**	2.00	**299**	0.10	2.00	**2.90**

STEEL PIPE PILE				Non-Unif. Site Provide		Non-Unif. Site		Equiv.
				FS	Qa	s	CFS	β
OD	*12.75*	(in)						
t	*0.38*	(in)						
[As]i	3.34	(sq.ft)	[2"]	3.00	**308**	0.25	3.00	**1.62**
A steel	14.58	(sq in)	[D]	3.00	**213**	0.30	3.00	**1.25**
A toe	0.89	(sq.ft)	[C]	3.00	**183**	0.27	3.00	**1.46**
Le	95	(ft)	[DC]	3.00	**199**	0.27	3.00	**1.46**

NOTE:		RELIABILITY APPROACH			Deterministic Equivalent	
Input *Italics*		Uniform				
Output **Bold**	NLT	Site	Provide			Equiv.
	Fb	s	β	Qa	CFS	FS
[2"]	1.136	0.12	2.00	**471**	1.81	**1.81**
[D]	0.978	0.12	3.00	**235**	2.38	**2.38**
[C]	0.863	0.11	2.50	**241**	1.95	**1.95**
[DC]	0.952	0.10	3.00	**257**	2.05	**2.05**

Table C32: H-Pile No.32 (Automatic Trip Hammer).

Calculated (Formula)				
Qsc	Qtc	Qc		Qtc/Qc
56	17	73		0.23

RELIABILITY APPROACH — Deterministic Equivalent

Predicted (Algorithm) Qsp	Qtp	Qp		Uniform Site s	Provide β	Qa	CFS	Equiv. FS
190	31	221	[2"]	0.12	2.00	**56**	1.81	**1.81**
187	25	211	[D]	0.12	3.00	**28**	2.38	**2.38**
295	33	329	[C]	0.11	2.50	**61**	1.95	**1.95**
225	30	255	[DC]	0.10	3.00	**45**	2.05	**2.05**

Measured (Loading Test) Qsm	Qtm	(Qt/Q)	CL Fb	Non-Unif. Site s	Provide β	Qa	CFS	Equiv. FS
66	37	0.36	0.461	0.25	1.50	**36**	2.80	**2.80**
59	31	0.34	0.312	0.30	1.25	**22**	3.01	**3.01**
69	39	0.36	0.361	0.27	1.25	**45**	2.64	**2.64**
63	36	0.36	0.363	0.27	1.25	**35**	2.64	**2.64**

DETERMINISTIC APPROACH — Reliability-Based Equivalent

	Qp*Fb/Qm	Qm	Qp*Fb	Uniform Site Provide FS	Qa	Uniform Site s	CFS	Equiv. β
[2"]	0.99	*103*	**102**	2.00	**51**	0.12	2.00	**2.37**
[D]	0.73	*90*	**66**	2.50	**26**	0.12	2.50	**3.18**
[C]	1.10	*108*	**119**	2.50	**47**	0.11	2.50	**3.49**
[DC]	0.94	99	**93**	2.00	**46**	0.10	2.00	**2.90**

H14x73 PILE				Non-Unif. Site Provide FS	Qa	Non-Unif. Site s	CFS	Equiv. β
de	15.90	(in)						
w	*14.59*	(in)						
h	13.61	(in)						
[As]i	4.16	(sq.ft)	[2"]	3.00	**34**	0.25	3.00	**1.62**
A steel	*21.40*	(sq in)	[D]	3.00	**22**	0.30	3.00	**1.25**
A toe	1.38	(sq.ft)	[C]	3.00	**40**	0.27	3.00	**1.46**
Le	50	(ft)	[DC]	3.00	**31**	0.27	3.00	**1.46**

NOTE:
Input *Italics*
Output **Bold**
CL -Constant Load test mtd

SPT N-VALUES ONLY (Without Loading Test, NLT)
RELIABILITY APPROACH — Deterministic Equivalent

	NLT Fb	Non-Unif. Site s	Provide β	Qa	CFS	Equiv. FS
[2"]	1.136	0.25	1.50	**90**	2.80	**2.80**
[D]	0.978	0.30	1.25	**69**	3.01	**3.01**
[C]	0.863	0.27	1.25	**107**	2.64	**2.64**
[DC]	0.952	0.27	1.25	**92**	2.64	**2.64**

Table C33: H-Pile No.33 (Safety Hammer).

Calculated (Formula)

Qsc	Qtc	Qc	Qtc/Qc
57	17	74	0.23

Predicted (Algorithm) — RELIABILITY APPROACH / Deterministic Equivalent

Qsp	Qtp	Qp		Uniform Site s	Provide β	Qa	CFS	Equiv. FS
194	31	225	[2"]	0.12	2.00	**57**	1.81	**1.81**
191	25	216	[D]	0.12	3.00	**28**	2.38	**2.38**
303	33	337	[C]	0.11	2.50	**62**	1.95	**1.95**
230	30	260	[DC]	0.10	3.00	**46**	2.05	**2.05**

Measured (Loading Test)

Qsm	Qtm	(Qt/Q)	CL Fb	Non-Unif. Site s	Provide β	Qa	CFS	Equiv. FS
66	37	0.36	0.461	0.25	1.50	**37**	2.80	**2.80**
59	31	0.34	0.312	0.30	1.25	**22**	3.01	**3.01**
69	39	0.36	0.361	0.27	1.25	**46**	2.64	**2.64**
63	36	0.36	0.363	0.27	1.25	**36**	2.64	**2.64**

DETERMINISTIC APPROACH / Reliability-Based Equivalent

	Qp*Fb/Qm	Qm	Qp*Fb	Uniform Site Provide FS	Qa	Uniform Site s	CFS	Equiv. β
[2"]	1.01	*103*	**104**	2.00	**52**	0.12	2.00	**2.37**
[D]	0.75	*90*	**67**	2.50	**27**	0.12	2.50	**3.18**
[C]	1.13	*108*	**122**	2.50	**49**	0.11	2.50	**3.49**
[DC]	0.96	99	**95**	2.00	**47**	0.10	2.00	**2.90**

H14x73 PILE

de	15.90	(in)
w	*14.59*	(in)
h	*13.61*	(in)
[As]i	4.16	(sq.ft)
A steel	*21.40*	(sq in)
A toe	1.38	(sq.ft)
Le	50	(ft)

	Non-Unif. Site Provide FS	Qa	Non-Unif. Site s	CFS	Equiv. β
[2"]	3.00	**35**	0.25	3.00	**1.62**
[D]	3.00	**22**	0.30	3.00	**1.25**
[C]	3.00	**41**	0.27	3.00	**1.46**
[DC]	3.00	**32**	0.27	3.00	**1.46**

NOTE:
Input *Italics*
Output **Bold**
CL -Constant Load test mtd

SPT N-VALUES ONLY (Without Loading Test, NLT)
RELIABILITY APPROACH / Deterministic Equivalent

	NLT Fb	Non-Unif. Site s	Provide β	Qa	CFS	Equiv. FS
[2"]	1.136	0.25	1.50	**91**	2.80	**2.80**
[D]	0.978	0.30	1.25	**70**	3.01	**3.01**
[C]	0.863	0.27	1.25	**110**	2.64	**2.64**
[DC]	0.952	0.27	1.25	**94**	2.64	**2.64**

Table C34: H-Pile No.34.

Calculated (Formula)

Qsc	Qtc	Qc	Qtc/Qc
54	43	97	0.44

Predicted (Algorithm) / RELIABILITY APPROACH (Uniform Site) / Deterministic Equivalent

Qsp	Qtp	Qp		Uniform Site s	Provide β	Qa	CFS	Equiv. FS
181	72	253	[2"]	0.12	2.00	**65**	1.81	**1.81**
175	59	234	[D]	0.12	3.00	**31**	2.38	**2.38**
266	77	342	[C]	0.11	2.50	**64**	1.95	**1.95**
207	70	277	[DC]	0.10	3.00	**49**	2.05	**2.05**

Measured (Loading Test) / RELIABILITY APPROACH (Non-Unif. Site) / Deterministic Equivalent

Qsm	Qtm	(Qt/Q)	CL Fb	Non-Unif. Site s	Provide β	Qa	CFS	Equiv. FS
164	86	0.34	0.461	0.25	1.50	**42**	2.80	**2.80**
148	72	0.33	0.312	0.30	1.25	**24**	3.01	**3.01**
194	103	0.35	0.361	0.27	1.25	**47**	2.64	**2.64**
169	90	0.35	0.363	0.27	1.25	**38**	2.64	**2.64**

DETERMINISTIC APPROACH (Known Site) / Reliability-Based Equivalent

	Qp*Fb/Qm	Qm	Qp*Fb	Known Site Provide FS	Qa	Known Site s	CFS	Equiv. β
[2"]	0.47	*250*	**117**	2.00	**58**	0.12	2.00	**2.37**
[D]	0.33	*220*	**73**	2.50	**29**	0.12	2.50	**3.18**
[C]	0.42	*297*	**124**	2.50	**49**	0.11	2.50	**3.49**
[DC]	0.39	259	**101**	2.00	**50**	0.10	2.00	**2.90**

DETERMINISTIC APPROACH (Non-Unif. Site) / Reliability-Based Equivalent

	Non-Unif. Site Provide FS	Qa	Non-Unif. Site s	CFS	Equiv. β
[2"]	3.00	**39**	0.25	3.00	**1.62**
[D]	3.00	**24**	0.30	3.00	**1.25**
[C]	3.00	**41**	0.27	3.00	**1.46**
[DC]	3.00	**34**	0.27	3.00	**1.46**

H14x73 PILE

de	15.90	(in)
w	*14.59*	(in)
h	*13.61*	(in)
[As]i	4.16	(sq.ft)
A steel	*21.40*	(sq in)
A toe	1.38	(sq.ft)
Le	52	(ft)

NOTE:
Input *Italics*
Output **Bold**

RELIABILITY APPROACH (Non-Unif. Site) / Deterministic Equivalent

	NLT Fb	Non-Unif. Site s	Provide β	Qa	CFS	Equiv. FS
[2"]	1.136	0.25	1.50	**103**	2.80	**2.80**
[D]	0.978	0.30	1.25	**76**	3.01	**3.01**
[C]	0.863	0.27	1.25	**112**	2.64	**2.64**
[DC]	0.952	0.27	1.25	**100**	2.64	**2.64**

Table C35: H-Pile No.35.

Calculated (Formula)				RELIABILITY APPROACH			Deterministic Equivalent	
Qsc	Qtc	Qc	Qtc/Qc					
58	53	111	0.48					
Predicted (Algorithm)				Uniform Site	Provide			Equiv.
Qsp	Qtp	Qp		s	β	Qa	CFS	FS
196	82	278	[2"]	0.12	2.00	**71**	1.81	**1.81**
187	69	256	[D]	0.12	3.00	**34**	2.38	**2.38**
281	87	368	[C]	0.11	2.50	**68**	1.95	**1.95**
221	80	301	[DC]	0.10	3.00	**53**	2.05	**2.05**
Measured (Loading Test)			CL	Non-Unif. Site	Provide			Equiv.
Qsm	Qtm	(Qt/Q)	Fb	s	β	Qa	CFS	FS
175	85	0.33	0.461	0.25	1.50	**46**	2.80	**2.80**
165	75	0.31	0.312	0.30	1.25	**27**	3.01	**3.01**
211	105	0.33	0.361	0.27	1.25	**50**	2.64	**2.64**
185	93	0.33	0.363	0.27	1.25	**41**	2.64	**2.64**

	Qp*Fb/Qm	Qm	Qp*Fb	DETERMINISTIC APPROACH Uniform Site Provide FS	Qa	Uniform Site s	CFS	Reliability-Based Equivalent Equiv. β
[2"]	0.49	*260*	**128**	2.00	**64**	0.12	2.00	**2.37**
[D]	0.33	*240*	**80**	2.50	**32**	0.12	2.50	**3.18**
[C]	0.42	*316*	**133**	2.50	**53**	0.11	2.50	**3.49**
[DC]	0.39	278	**109**	2.00	**55**	0.10	2.00	**2.90**

H14x73 PILE				Non-Unif. Site Provide FS	Qa	Non-Unif. Site s	CFS	Equiv. β
de	15.90	(in)						
w	*14.59*	(in)						
h	*13.61*	(in)						
[As]i	4.16	(sq.ft)	[2"]	3.00	**43**	0.25	3.00	**1.62**
A steel	*21.40*	(sq in)	[D]	3.00	**27**	0.30	3.00	**1.25**
A toe	1.38	(sq.ft)	[C]	3.00	**44**	0.27	3.00	**1.46**
Le	54	(ft)	[DC]	3.00	**36**	0.27	3.00	**1.46**

NOTE:		NLT	RELIABILITY APPROACH Unknown Site	Provide		Deterministic Equivalent	Equiv.
Input	*Italics*						
Output	**Bold**						
		Fb	s	β	Qa	CFS	FS
	[2"]	1.136	0.25	1.50	**113**	2.80	**2.80**
	[D]	0.978	0.30	1.25	**83**	3.01	**3.01**
	[C]	0.863	0.27	1.25	**120**	2.64	**2.64**
	[DC]	0.952	0.27	1.25	**109**	2.64	**2.64**

Table C36: H-Pile No.36.

Calculated (Formula)				
Qsc	Qtc	Qc		Qtc/Qc
81	24	105		0.23

RELIABILITY APPROACH — Deterministic Equivalent

Predicted (Algorithm)				Uniform Site	Provide			Equiv.
Qsp	Qtp	Qp		s	β	Qa	CFS	FS
270	25	295	[2"]	0.12	2.00	**75**	1.81	**1.81**
248	23	270	[D]	0.12	3.00	**35**	2.38	**2.38**
342	25	367	[C]	0.11	2.50	**68**	1.95	**1.95**
283	24	307	[DC]	0.10	3.00	**54**	2.05	**2.05**

Measured (Loading Test)			CL	Non-Unif. Site	Provide			Equiv.
Qsm	Qtm	(Qt/Q)	Fb	s	β	Qa	CFS	FS
93	31	0.25	0.461	0.25	1.50	**49**	2.80	**2.80**
68	22	0.24	0.312	0.30	1.25	**28**	3.01	**3.01**
106	37	0.26	0.361	0.27	1.25	**50**	2.64	**2.64**
86	30	0.26	0.363	0.27	1.25	**42**	2.64	**2.64**

DETERMINISTIC APPROACH — Reliability-Based Equivalent

	Qp*Fb/Qm	Qm	Qp*Fb	Uniform Site Provide FS	Qa	Uniform Site s	CFS	Equiv. β
[2"]	1.10	*124*	**136**	2.00	**68**	0.12	2.00	**2.37**
[D]	0.94	*90*	**84**	2.50	**34**	0.12	2.50	**3.18**
[C]	0.93	*143*	**133**	2.50	**53**	0.11	2.50	**3.49**
[DC]	0.96	117	**111**	2.00	**56**	0.10	2.00	**2.90**

12BP53 H - PILE		
de	13.44	(in)
w	*12.05*	(in)
h	*11.78*	(in)
[As]i	3.52	(sq.ft)
A steel	*15.50*	(sq in)
A toe	0.99	(sq.ft)
Le	55	(ft)

	Non-Unif. Site Provide FS	Qa	Non-Unif. Site s	CFS	Equiv. β
[2"]	3.00	**45**	0.25	3.00	**1.62**
[D]	3.00	**28**	0.30	3.00	**1.25**
[C]	3.00	**44**	0.27	3.00	**1.46**
[DC]	3.00	**37**	0.27	3.00	**1.46**

NOTE:
Input *Italics*
Output **Bold**

RELIABILITY APPROACH — Deterministic Equivalent

	NLT Fb	Non-Unif. Site s	Provide β	Qa	CFS	Equiv. FS
[2"]	1.136	0.25	1.50	**120**	2.80	**2.80**
[D]	0.978	0.30	1.25	**88**	3.01	**3.01**
[C]	0.863	0.27	1.25	**120**	2.64	**2.64**
[DC]	0.952	0.27	1.25	**111**	2.64	**2.64**

Table C37: H-Pile No.37.

Calculated (Formula)								
Qsc	Qtc	Qc	Qtc/Qc	RELIABILITY APPROACH			Deterministic	
97	42	139	0.30				Equivalent	
				Uniform				
Predicted (Algorithm)				Site	Provide			Equiv.
Qsp	Qtp	Qp		s	β	Qa	CFS	FS
325	64	389	[2"]	0.12	2.00	**99**	1.81	**1.81**
310	54	364	[D]	0.12	3.00	**48**	2.38	**2.38**
462	67	529	[C]	0.11	2.50	**98**	1.95	**1.95**
365	62	427	[DC]	0.10	3.00	**76**	2.05	**2.05**
				Non-Unif.				
Measured (Loading Test)			CL	Site	Provide			Equiv.
Qsm	Qtm	(Qt/Q)	Fb	s	β	Qa	CFS	FS
83	39	0.32	0.461	0.25	1.50	**64**	2.80	**2.80**
71	31	0.31	0.312	0.30	1.25	**38**	3.01	**3.01**
89	43	0.32	0.361	0.27	1.25	**72**	2.64	**2.64**
79	38	0.33	0.363	0.27	1.25	**59**	2.64	**2.64**
				DETERMINISTIC APPROACH			Reliability-Based	
				Uniform			Equivalent	
				Site		Uniform		Equiv.
	Qp*Fb/Qm			Provide		Site		
		Qm	Qp*Fb	FS	Qa	s	CFS	β
[2"]	1.47	*122*	**179**	2.00	**90**	0.12	2.00	**2.37**
[D]	1.11	*102*	**113**	2.50	**45**	0.12	2.50	**3.18**
[C]	1.45	*132*	**191**	2.50	**76**	0.11	2.50	**3.49**
[DC]	1.32	117	**155**	2.00	**77**	0.10	2.00	**2.90**
14BP73 H - PILE				Non-Unif.				
de	15.90	(in)		Site		Non-Unif.		
w	*14.59*	(in)		Provide		Site		Equiv.
h	13.61	(in)		FS	Qa	s	CFS	β
[As]i	4.16	(sq.ft)	[2"]	3.00	**60**	0.25	3.00	**1.62**
A steel	*21.40*	(sq in)	[D]	3.00	**38**	0.30	3.00	**1.25**
A toe	1.38	(sq.ft)	[C]	3.00	**64**	0.27	3.00	**1.46**
Le	55	(ft)	[DC]	3.00	**52**	0.27	3.00	**1.46**

NOTE:				RELIABILITY APPROACH			Deterministic	
Input	*Italics*			Non-Unif.			Equivalent	
Output	**Bold**		NLT	Site	Provide			Equiv.
			Fb	s	β	Qa	CFS	FS
		[2"]	1.136	0.25	1.50	**158**	2.80	**2.80**
		[D]	0.978	0.30	1.25	**118**	3.01	**3.01**
		[C]	0.863	0.27	1.25	**173**	2.64	**2.64**
		[D&C]	0.952	0.27	1.25	**154**	2.64	**2.64**

Table C38: H-Pile No.38.

Calculated (Formula)			
	Qsc	Qtc	Qc
	176	139	316
Predicted (Algorithm)	Qsp	Qtp	Qp
[2"]	584	105	689
[D]	521	102	623
[C]	688	102	789
[DC]	583	102	685
Measured (Loading Test)	Qsm	Qtm	(Qt/Q)
[2"]	437	113	0.21
[D]	352	88	0.20
[C]	773	213	0.22
[DC]	559	154	0.22

Qtc/Qc: 0.44

RELIABILITY APPROACH — Deterministic Equivalent

Uniform Site		Uniform Site s	Provide β	Qa	CFS	Equiv. FS
[2"]		0.12	2.00	**176**	1.81	**1.81**
[D]		0.12	3.00	**82**	2.38	**2.38**
[C]		0.11	2.50	**147**	1.95	**1.95**
[DC]		0.10	3.00	**121**	2.05	**2.05**

Non-Unif. Site	CL Fb	Non-Unif. Site s	Provide β	Qa	CFS	Equiv. FS
[2"]	0.461	0.25	1.50	**113**	2.80	**2.80**
[D]	0.312	0.30	1.25	**65**	3.01	**3.01**
[C]	0.361	0.27	1.25	**108**	2.64	**2.64**
[DC]	0.363	0.27	1.25	**94**	2.64	**2.64**

DETERMINISTIC APPROACH — Reliability-Based Equivalent

	Qp*Fb/Qm	Qm	Qp*Fb	Uniform Site Provide FS	Qa	Uniform Site s	CFS	Equiv. β
[2"]	0.58	*550*	**318**	2.00	**159**	0.12	2.00	**2.37**
[D]	0.44	*440*	**195**	2.50	**78**	0.12	2.50	**3.18**
[C]	0.29	*986*	**285**	2.50	**114**	0.11	2.50	**3.49**
[DC]	0.35	713	**249**	2.00	**124**	0.10	2.00	**2.90**

W14X102 PILE		
de	16.21	(in)
w	*14.57*	(in)
h	*14.16*	(in)
[As]i	4.24	(sq.ft)
A steel	*29.10*	(sq in)
A toe	1.43	(sq.ft)
Le	75	(ft)

	Non-Unif. Site Provide FS	Qa	Unknown Site s	CFS	Equiv. β
[2"]	3.00	**106**	0.25	3.00	**1.62**
[D]	3.00	**65**	0.30	3.00	**1.25**
[C]	3.00	**95**	0.27	3.00	**1.46**
[DC]	3.00	**83**	0.27	3.00	**1.46**

NOTE:
Input *Italics*
Output **Bold**

RELIABILITY APPROACH — Deterministic Equivalent

	NLT Fb	Non-Unif. Site s	Provide β	Qa	CFS	Equiv. FS
[2"]	1.136	0.25	1.50	**280**	2.80	**2.80**
[D]	0.978	0.30	1.25	**203**	3.01	**3.01**
[C]	0.863	0.27	1.25	**258**	2.64	**2.64**
[DC]	0.952	0.27	1.25	**247**	2.64	**2.64**

Table C39: H-Pile No.39.

Calculated (Formula)

Qsc	Qtc	Qc	Qtc/Qc
205	1630	1836	0.89

Predicted (Algorithm) / RELIABILITY APPROACH – Uniform Site / Deterministic Equivalent

	Qsp	Qtp	Qp	Uniform Site s	Provide β	Qa	CFS	Equiv. FS
[2"]	676	1040	1716	0.12	2.00	**438**	1.81	**1.81**
[D]	588	1046	1634	0.12	3.00	**214**	2.38	**2.38**
[C]	737	989	1726	0.11	2.50	**320**	1.95	**1.95**
[DC]	643	1007	1651	0.10	3.00	**292**	2.05	**2.05**

Measured (Loading Test) / RELIABILITY APPROACH – Non-Unif. Site / Deterministic Equivalent

Qsm	Qtm	(Qt/Q)	CL Fb	Non-Unif. Site s	Provide β	Qa	CFS	Equiv. FS
306	69	0.19	0.461	0.25	1.50	**283**	2.80	**2.80**
278	62	0.18	0.312	0.30	1.25	**169**	3.01	**3.01**
387	95	0.20	0.361	0.27	1.25	**236**	2.64	**2.64**
331	80	0.20	0.363	0.27	1.25	**227**	2.64	**2.64**

DETERMINISTIC APPROACH – Uniform Site / Reliability-Based Equivalent

	Qp*Fb/Qm	Qm	Qp*Fb	Uniform Site Provide FS	Qa	Uniform Site s	CFS	Equiv. β
[2"]	2.11	*375*	**791**	2.00	**396**	0.12	2.00	**2.37**
[D]	1.50	*340*	**510**	2.50	**204**	0.12	2.50	**3.18**
[C]	1.29	*482*	**623**	2.50	**249**	0.11	2.50	**3.49**
[DC]	1.46	411	**599**	2.00	**300**	0.10	2.00	**2.90**

H14x73 PILE

de	15.90	(in)
w	*14.59*	(in)
h	*13.61*	(in)
[As]i	4.16	(sq.ft)
A steel	*21.40*	(sq in)
A toe	1.38	(sq.ft)
Le	59	(ft)

DETERMINISTIC APPROACH – Non-Unif. Site / Reliability-Based Equivalent

	Non-Unif. Site Provide FS	Qa	Non-Unif. Site s	CFS	Equiv. β
[2"]	3.00	**264**	0.25	3.00	**1.62**
[D]	3.00	**170**	0.30	3.00	**1.25**
[C]	3.00	**208**	0.27	3.00	**1.46**
[DC]	3.00	**200**	0.27	3.00	**1.46**

NOTE:
Input *Italics*
Output **Bold**

RELIABILITY APPROACH – Non-Unif. Site / Deterministic Equivalent

	NLT Fb	Non-Unif. Site s	Provide β	Qa	CFS	Equiv. FS
[2"]	1.136	0.25	1.50	**697**	2.80	**2.80**
[D]	0.978	0.30	1.25	**531**	3.01	**3.01**
[C]	0.863	0.27	1.25	**564**	2.64	**2.64**
[DC]	0.952	0.27	1.25	**595**	2.64	**2.64**

Table C40: H-Pile No.40.

Calculated (Formula)								
Qsc	Qtc	Qc	Qtc/Qc	RELIABILITY APPROACH			Deterministic Equivalent	
283	105	387	0.27					
Predicted (Algorithm)				Uniform Site	Provide			Equiv.
Qsp	Qtp	Qp		s	β	Qa	CFS	FS
900	14	914	[2"]	0.12	2.00	**233**	1.81	**1.81**
676	19	695	[D]	0.12	3.00	**91**	2.38	**2.38**
662	11	673	[C]	0.11	2.50	**125**	1.95	**1.95**
664	13	677	[DC]	0.10	3.00	**120**	2.05	**2.05**
Measured (Loading Test)			CL	Non-Unif. Site	Provide			Equiv.
Qsm	Qtm	(Qt/Q)	Fb	s	β	Qa	CFS	FS
261	19	0.07	0.461	0.25	1.50	**151**	2.80	**2.80**
242	18	0.07	0.312	0.30	1.25	**72**	3.01	**3.01**
313	26	0.08	0.361	0.27	1.25	**92**	2.64	**2.64**
277	23	0.08	0.363	0.27	1.25	**93**	2.64	**2.64**

	Qp*Fb/Qm			DETERMINISTIC APPROACH			Reliability-Based Equivalent	
				Uniform Site Provide		Uniform Site		Equiv.
		Qm	Qp*Fb	FS	Qa	s	CFS	β
[2"]	1.50	*280*	**421**	2.00	**211**	0.12	2.00	**2.37**
[D]	0.83	*260*	**217**	2.50	**87**	0.12	2.50	**3.18**
[C]	0.72	*339*	**243**	2.50	**97**	0.11	2.50	**3.49**
[DC]	0.82	300	**246**	2.00	**123**	0.10	2.00	**2.90**

H14x73 PILE		
de	13.44	(in)
w	*12.05*	(in)
h	*11.78*	(in)
[As]i	3.52	(sq.ft)
A steel	*15.50*	(sq in)
A toe	0.99	(sq.ft)
Le	102	(ft)

	Non-Unif. Site Provide		Non-Unif. Site		Equiv.
	FS	Qa	s	CFS	β
[2"]	3.00	**140**	0.25	3.00	**1.62**
[D]	3.00	**72**	0.30	3.00	**1.25**
[C]	3.00	**81**	0.27	3.00	**1.46**
[DC]	3.00	**82**	0.27	3.00	**1.46**

NOTE:
Input *Italics*
Output **Bold**

	NLT	RELIABILITY APPROACH Non-Unif. Site	Provide		Deterministic Equivalent	Equiv.
	Fb	s	β	Qa	CFS	FS
[2"]	1.136	0.25	1.50	**371**	2.80	**2.80**
[D]	0.978	0.30	1.25	**226**	3.01	**3.01**
[C]	0.863	0.27	1.25	**220**	2.64	**2.64**
[DC]	0.952	0.27	1.25	**244**	2.64	**2.64**

Table C41: Concrete Pile No.41.

Calculated (Formula)		
Qsc	Qtc	Qc
36	36	73

Qtc/Qc
0.50

RELIABILITY APPROACH — Uniform Site (Deterministic Equivalent)

Predicted (Algorithm) Qsp	Qtp	Qp		Uniform Site s	Provide β	Qa	CFS	Equiv. FS
124	113	238	[2"]	0.12	2.00	**61**	1.81	**1.81**
128	83	210	[D]	0.12	3.00	**28**	2.38	**2.38**
221	132	352	[C]	0.11	2.50	**65**	1.95	**1.95**
160	111	271	[DC]	0.10	3.00	**48**	2.05	**2.05**

Non-Unif. Site

Measured (Loading Test) Qsm	Qtm	(Qt/Q)	CL Fb	Non-Unif. Site s	Provide β	Qa	CFS	Equiv. FS
110	115	0.51	0.461	0.25	1.50	**39**	2.80	**2.80**
99	91	0.48	0.312	0.30	1.25	**22**	3.01	**3.01**
114	114	0.50	0.361	0.27	1.25	**48**	2.64	**2.64**
103	106	0.51	0.363	0.27	1.25	**37**	2.64	**2.64**

DETERMINISTIC APPROACH — Uniform Site (Reliability-Based Equivalent)

	Qp*Fb/Qm	Qm	Qp*Fb	Uniform Site Provide FS	Qa	Uniform Site s	CFS	Equiv. β
[2"]	0.49	*225*	**110**	2.00	**55**	0.12	2.00	**2.37**
[D]	0.35	*190*	**66**	2.50	**26**	0.12	2.50	**3.18**
[C]	0.56	*228*	**127**	2.50	**51**	0.11	2.50	**3.49**
[DC]	0.47	209	**98**	2.00	**49**	0.10	2.00	**2.90**

Non-Unif. Site

	Non-Unif. Site Provide FS	Qa	Non-Unif. Site s	CFS	Equiv. β
[2"]	3.00	**37**	0.25	3.00	**1.62**
[D]	3.00	**22**	0.30	3.00	**1.25**
[C]	3.00	**42**	0.27	3.00	**1.46**
[DC]	3.00	**33**	0.27	3.00	**1.46**

16" SQUARE CONCRETE PILE

de	18.05	(in)
w	*16.00*	(in)
h	*16.00*	(in)
[As]i	4.73	(sq.ft)
A conc	*256*	(sq in)
A toe	1.78	(sq.ft)
Le	40	(ft)
E	6.30	(ksi)

NOTE:
Input *Italics*
Output **Bold**

RELIABILITY APPROACH — Non-Unif. Site (Deterministic Equivalent)

	NLT Fb	Non-Unif. Site s	Provide β	Qa	CFS	Equiv. FS
[2"]	1.136	0.25	1.50	**96**	2.80	**2.80**
[D]	0.978	0.30	1.25	**68**	3.01	**3.01**
[C]	0.863	0.27	1.25	**115**	2.64	**2.64**
[DC]	0.952	0.27	1.25	**98**	2.64	**2.64**

Table C42: Concrete Pile No.42.

Calculated (Formula)				RELIABILITY APPROACH			Deterministic Equivalent	
Qsc	Qtc	Qc	Qtc/Qc					
59	56	115	0.49					
Predicted (Algorithm)				Uniform Site	Provide			Equiv.
Qsp	Qtp	Qp		s	β	Qa	CFS	FS
199	122	322	[2"]	0.12	2.00	**82**	1.81	**1.81**
197	96	293	[D]	0.12	3.00	**38**	2.38	**2.38**
317	136	452	[C]	0.11	2.50	**84**	1.95	**1.95**
239	120	359	[DC]	0.10	3.00	**64**	2.05	**2.05**

Measured (Loading Test)			CL	Non-Unif. Site	Provide			Equiv.
Qsm	Qtm	(Qt/Q)	Fb	s	β	Qa	CFS	FS
175	120	0.41	0.461	0.25	1.50	**53**	2.80	**2.80**
151	94	0.39	0.312	0.30	1.25	**30**	3.01	**3.01**
191	130	0.40	0.361	0.27	1.25	**62**	2.64	**2.64**
168	115	0.41	0.363	0.27	1.25	**49**	2.64	**2.64**

	Qp*Fb/Qm			DETERMINISTIC APPROACH Uniform Site Provide		Uniform Site	Reliability-Based Equivalent	Equiv.
		Qm	Qp*Fb	FS	Qa	s	CFS	β
[2"]	0.50	*295*	**148**	2.00	**74**	0.12	2.00	**2.37**
[D]	0.37	*245*	**91**	2.50	**37**	0.12	2.50	**3.18**
[C]	0.51	*321*	**163**	2.50	**65**	0.11	2.50	**3.49**
[DC]	0.46	283	**130**	2.00	**65**	0.10	2.00	**2.90**

16" SQUARE CONCRETE PILE				Non-Unif. Site Provide		Unknown Site		Equiv.
				FS	Qa	s	CFS	β
de	18.05	(in)	[2"]	3.00	**49**	0.25	3.00	**1.62**
w	*16.00*	(in)	[D]	3.00	**30**	0.30	3.00	**1.25**
h	*16.00*	(in)	[C]	3.00	**54**	0.27	3.00	**1.46**
[As]i	4.73	(sq.ft)	[DC]	3.00	**43**	0.27	3.00	**1.46**
A conc	*256*	(sq in)						
A toe	1.78	(sq.ft)						
Le	51	(ft)						
E	6.30	(ksi)						

NOTE:		NLT	RELIABILITY APPROACH Non-Unif. Site	Provide		Deterministic Equivalent	Equiv.
Input	*Italics*						
Output	**Bold**	Fb	s	β	Qa	CFS	FS
	[2"]	1.136	0.25	1.50	**131**	2.80	**2.80**
	[D]	0.978	0.30	1.25	**95**	3.01	**3.01**
	[C]	0.863	0.27	1.25	**148**	2.64	**2.64**
	[DC]	0.952	0.27	1.25	**130**	2.64	**2.64**

Table C43: Concrete Pile No.43.

Calculated (Formula)									
Qsc	Qtc	Qc	Qtc/Qc	RELIABILITY APPROACH			Deterministic Equivalent		
4	45	48	0.93	Uniform					
Predicted (Algorithm)				Site	Provide			Equiv.	
Qsp	Qtp	Qp		s	β	Qa	CFS	FS	
12	320	332	[2"]	0.12	2.00	**85**	1.81	**1.81**	
14	197	211	[D]	0.12	3.00	**28**	2.38	**2.38**	
29	411	440	[C]	0.11	2.50	**82**	1.95	**1.95**	
19	317	335	[DC]	0.10	3.00	**59**	2.05	**2.05**	
				Non-Unif.					
Measured (Loading Test)			CL	Site	Provide			Equiv.	
Qsm	Qtm	(Qt/Q)	Fb	s	β	Qa	CFS	FS	
####	####	0.87	0.461	0.25	1.50	**55**	2.80	**2.80**	
####	####	0.80	0.312	0.30	1.25	**22**	3.01	**3.01**	
####	####	0.81	0.361	0.27	1.25	**60**	2.64	**2.64**	
####	####	0.83	0.363	0.27	1.25	**46**	2.64	**2.64**	

				DETERMINISTIC APPROACH			Reliability-Based Equivalent	
				Uniform Site		Uniform		Equiv.
	Qp*Fb/Qm			Provide		Site		
		Qm	Qp*Fb	FS	Qa	s	CFS	β
[2"]	####	*?*	**153**	2.00	**77**	0.12	2.00	**2.37**
[D]	####	*?*	**66**	2.50	**26**	0.12	2.50	**3.18**
[C]	####	*?*	**159**	2.50	**64**	0.11	2.50	**3.49**
[DC]	####	#VALUE!	**122**	2.00	**61**	0.10	2.00	**2.90**
16" OCTAGONAL CONCRETE PILE				Non-Unif.				
de	*16.00*	(in)		Site		Non-Unif.		
w		(in)		Provide		Site		Equiv.
h		(in)		FS	Qa	s	CFS	β
[As]i	4.19	(sq.ft)	[2"]	3.00	**51**	0.25	3.00	**1.62**
A conc	*201*	(sq in)	[D]	3.00	**22**	0.30	3.00	**1.25**
A toe	1.40	(sq.ft)	[C]	3.00	**53**	0.27	3.00	**1.46**
Le	13	(ft)	[DC]	3.00	**41**	0.27	3.00	**1.46**
E	6.30	(ksi)						

NOTE:			RELIABILITY APPROACH			Deterministic Equivalent	
Input	*Italics*		Non-Unif.				
Output	**Bold**	NLT	Site	Provide			Equiv.
		Fb	s	β	Qa	CFS	FS
	[2"]	1.136	0.25	1.50	**135**	2.80	**2.80**
	[D]	0.978	0.30	1.25	**69**	3.01	**3.01**
	[C]	0.863	0.27	1.25	**144**	2.64	**2.64**
	[DC]	0.952	0.27	1.25	**121**	2.64	**2.64**

APPENDIX D

Table D: Measured Vs Predicted Capacities, Pipe Pile

Student t-test

$$(\mu_m - \mu_p) = \bar{p} \pm t_{\alpha/2} \frac{s_p}{\sqrt{n}} \quad ; \quad \bar{p} = \frac{\sum p_i}{n} \quad ; \quad s_p = \sqrt{\frac{\sum (p_i - \bar{p})^2}{n-1}} \quad ; \quad t_{0.005} = 4.032$$

Table D1: Comparing Measured and Predicted Capacities.

PILE NO.	PIPE PILE	[2"] CRITERIA			[D] CRITERIA		
		Q_m (tons)	Q_p (tons)	Q_m-Q_p	Q_m (tons)	Q_p (tons)	Q_m-Q_p
26	No.3, Kansas City	115	127	-12	80	78	2
27	No.3A, Kansas City	112	144	-32	95	90	5
28	No.7, Kansas City	130	148	-18	110	93	17
29	No.7A, Kansas City	140	181	-41	115	115	0
30	No.8, Aliquippa	480	760	-280	500	551	-51
31	No.28, Hamilton	580	923	-343	640	638	2
			$\bar{p}$	-121.00		$\bar{p}$	-4.17
			s_p	136.24		s_p	21.67

PILE NO.	PIPE PILE	[C] CRITERIA			[DC] CRITERIA		
		Q_m (tons)	Q_p (tons)	Q_m-Q_p	Q_m (tons)	Q_p (tons)	Q_m-Q_p
26	No.3, Kansas City	119	120	-1	100	102	-2
27	No.3A, Kansas City	121	144	-23	108	120	-12
28	No.7, Kansas City	137	148	-11	124	123	1
29	No.7A, Kansas City	150	193	-43	133	157	-24
30	No.8, Aliquippa	495	508	-13	498	531	-33
31	No.28, Hamilton	827	550	277	734	598	136
			$\bar{p}$	31.00		$\bar{p}$	11.00
			s_p	110.78		s_p	57.13

(i) [2"] Criteria

$$-345.26 \leq (\mu_m - \mu_p) \leq 103.26$$

(ii) [D] Criteria

$$-39.84 \leq (\mu_m - \mu_p) \leq 31.5$$

(iii) [C] Criteria

$$-151.35 \leq (\mu_m - \mu_p) \leq 213.35$$

(iv) [DC] Criteria

$$-83.04 \leq (\mu_m - \mu_p) \leq 105.04$$

Therefore, there is no significant difference at the 99% Confidence Interval between measured and predicted capacities for all criteria (pipe pile only).

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