FINAL REPORT

CALIBRATING RESISTANCE FACTORS IN THE LOAD AND RESISTANCE FACTOR DESIGN FOR FLORIDA FOUNDATIONS

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16. Abstract

The AASHTO LRFD Specification was approved for used in 1994, and the FDOT is planning to use it in 1998. To ensure that the LRFD Specification is adopted successfully, it is important to generate resistance factors that are suitable for the Florida geologic conditions and design methods used by FDOT.

Using the foundation database collected in the University of Florida, Florida geologic conditions and FDOT design methods, the resistant factors were calibrated by reliability analysis and Allowable Stress Design Fitting. The calibrations of the resistance factors were performed on the designs of driven pile, drilled shaft, shallow foundation and retaining wall system. The resistance factors for deep foundation load tests including static load testing and dynamic load testing using Pile driving Analyzer also were performed.

Two FDOT projects were used as design examples to compare the results of ASD and LRFD using the recommended resistance factors. The comparisons indicated that the LRFD method using the recommended resistance factors resulted compatible designs with that of ASD method.

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	l
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3

NOTE: Volumes greater than 1000 l shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	l
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
psi	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg	megagrams	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	psi

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised August 1992)

CHAPTER 1
INTRODUCTION

1.1 General

Load and Resistance Factor Design (LRFD) was adopted by the American Association of State Highway and Transportation Officials (AASHTO) as an alternative method for the design of bridge superstructures in the early 1970's. At that time, allowable stress design was the only method available in AASHTO for the design of the bridge foundations. In 1987 AASHTO initiated a research program, administered by the National Cooperative Highway Research Program to develop LRFD method for Bridge Foundations (NCHRP 24-4), and concluded with a report NCHRP 343 "Manuals for the Design of Bridge Foundations".

In LRFD method, the external loads are multiplied by load factors while the soil resistances are multiplied by resistance factors to ensure an acceptable of safety margin. The load factor is relatively less sensitive to the different environments. However, the resistance factors (performance factors) used for geotechnical engineering depend on

- geologic formation
- method of field exploration and laboratory testing
- type of foundation and earth structure
- design methodology.

The resistance factors (performance factors) used in the AASHTO LRFD Specification were calibrated through general geologic conditions and some specific design methods as described in report NCHRP 343.

The unique characteristics of Florida geology, such as soft limestone formation, limerock backfill materials which were widely used in south Florida, and the FDOT design methods of driven piles and drilled shafts developed through the FDOT research projects were not considered in the NCHRP 24-4 research project in developing the Resistant Factors. Therefore, the validity of the Resistance Factors recommended in the AASHTO LRFD Specification is questionable and may not be applicable in Florida area.

1.2 Objectives

The AASHTO LRFD Specification was approved for used in 1994, and the FDOT is planning to use it in 1998. To ensure that the LRFD Specification is adopted successfully, it is important to generate resistance factors that are suitable for the Florida geologic conditions and design methods used by FDOT. The objectives of this research program are as follows:

1. To review the assumptions and design methods used in the research project NCHRP 24-4 for calibrating the resistant factors .
2. To calibrate resistant factors using Florida geologic conditions and FDOT design methods.
3. To recommend the resistant factors for Florida foundations and retaining wall system.

1.3 Scope of Work

The overall project will be divided into five major tasks as follows:

Task 1: Literature Review

1. Review and study the state-of-the art of LRFD in geotechnical engineering and the AASHTO LRFD Specifications regarding the geotechnical and foundation

engineering applications.

2. Review the procedures used in the research project NCHRP 24-4 for calibrating the resistance factors.

Task 2: Database Collection and Analysis

1. Collect and summarize the geotechnical characteristics of typical Florida soils and limerocks through the FDOT District Offices.
2. Collect and summarize the database of driven pile and drilled shaft load tests performed in Florida.
3. Collect the case histories of FDOT projects using shallow foundations as bridge foundations, and typical retaining wall system designs.

Task 3: Calibration of Resistance Factors

1. Summarize the design methods for geotechnical and foundation engineering used in the FDOT projects.
2. Perform calibration of resistance factors using reliability theory in deep foundation designs including, but not limited to,
   - 2.1 SPT-94 analyses for driven pile
   - 2.2 Drilled shaft design by McVay's method
   - 2.3 Dynamic load testing
   - 2.4 Static load testing using conventional methods and the Osterberg Cell.
3. Perform calibration of resistant factors for shallow foundations and retaining wall systems.
4. Comply with and evaluate the design criteria for Serviceability Limit.

5. Compare the findings with AASHTO LRFD Specifications as identified in the NCHRP-343 report.

Task 4: Comparative Analysis

1. Select up to two (2) FDOT projects in various Districts throughout the State and perform geotechnical and foundation designs using the proposed LRFD guidelines.
2. Compare the results of the LRFD method and the ASD method.

Task 5: Report Preparation

1. Finalize a design procedure and guidelines for using the LRFD method in geotechnical and foundation engineering.

1.4 Literature Reviews

Over the last two decades, the strength design method in corporation of Limit States Design (LSD) concept has received increasing attention in geotechnical engineering in preference to the Allowable Stress Design (ASD). However, significant and varying degrees of uncertainty are inherently involved in geotechnical engineering and foundation design. Therefore, in recent years there has been a trend towards the use of reliability-based design and probabilistic methods, and the development of the Load and Resistant Factor Design (LRFD) method which includes Strength Limits and Service Limits Design.

Many countries are preparing for a forthcoming shift of the geotechnical engineering and foundation design from ASD to LRFD methods. In general, there are two approaches in Strength Limits Design, the factored strength approach and the factored resistant approach, to establish the geotechnical resistance used in the LRFD method. The factored strength approach which was adopted in Europe following the original work of Brinch Hansen and the Danish Code where specified partial factors are applied to the individual soil strength properties for the calculation of factored resistance. In the factored resistance approach, which was adopted in North America, an overall resistance factor is applied to the resistance for the each applicable limit state. The advantage of the factored resistance approach over the factored strength approach is that the derived resistance factors reflect not only uncertainties in soil properties, but also the uncertainties in the design methods and the extent of site investigation. The resistant factor, ϕ, from the factored resistance approach is similar in concept to the global factor of safety in ASD, and would be familiar to geotechnical engineers (Backer, 1996a, 1996b).

In the United States, the American Association of State Highway and Transportation Officials (AASHTO) initiated a research program in 1987, administered by the National Cooperative Highway Research Program, to develop a rational load factor design method for bridge foundations (NCHRP 24-4). This research provided the basis for development of Section 10- Foundations and Section 11- Abutments, Piers and Walls in the AASHTO LRFD Specification. In Canada, a task force lead by Golder Associates Ltd. (1992) to perform code calibration was introduced into Section 4.2- Foundations of the National Building Code of Canada (NBCC) in 1995. The resistance factors for deep foundations recommended in the AASHTO LRFD Specification (1994) and the NBCC Specification are summarized in Table 1.1. As shown in Table 1.1, the resistance factors in the AASHTO code for deep foundations are generally higher than those in the NBCC code. The target reliability index is on the order of 2 to 2.5 (corresponding probability of failure 10^{-1} to 10^{-2}) in AASHTO code, and 3.5 to 4.5 (corresponding probability of failure 10^{-4} to 10^{-5}) in NBCC code.

In addition to the consideration of Strength Limits which ensure that the designs provide adequate resistance against geotechnical and structural failures including bearing capacity failure, sliding and overall instability. The design of Service Limits ensures that the functions of the structures under normal service conditions perform satisfactorily. Service Limit States of structures may be reached through excessive settlement, excessive lateral deflection, or structural deterioration.

The vertical and lateral displacement of foundations must be evaluated for all applicable dead and live load combinations, and compared with tolerable movement criteria. Since the evaluations of foundation displacements by LRFD are performed in accordance with the Service I Limit State where load factor, γ and resistance factor, ϕ equal to one (1), the methodologies used to estimate settlement and lateral deflection are identical for LRFD and ASD.

Although many researchers have attempted to comply the tolerable movements of bridge foundation, the criteria of tolerable axial and lateral movement of foundations usually are developed by the structural engineer in accordance with the design characteristics of the superstructures. In general, the structural stability of the entire bridge under the external load is the major concern, and not the absolute value of the movement of bridge foundations.

Table 1.1

Resistance Factors for Geotechnical Strength Limit State
in Axial Loaded Piles

Method/Soil/Condition		Resistance Factor	
		AASHTO (1994)	NBCC (1995)
Ultimate Bearing Resistance of Single Piles	Skin Friction: Clay α-method β-method γ-method	 0.70 0.50 0.55	 -
	End Bearing: Clay and Rock Clay Rock	 0.70 0.50	 -
	Skin Friction and End Bearing: Sand SPT-method CPT-method	 0.45 0.55	 0.4 0.4
	Skin Friction and End Bearing: All Soils Load Test Pile Driving Analyzer	 0.80 0.70	 0.6 0.5
Block Failure	Clay	0.65	-
Uplift Resistance of Single Piles	α-method β-method γ-method SPT-method CPT-method Load Test	0.60 0.40 0.45 0.35 0.45 0.80	0.3 0.3 0.3 0.3 0.3 0.4
Group Uplift Resistance	Sand Clay	0.55 0.55	-

CHAPTER 2

METHODOLOGY FOR CALIBRATION OF RESISTANCE FACTOR

2.1 Design Philosophies and Methods

The basic design criteria, regardless of the method used, is that the resistances must be greater than the applied loads and provide an acceptable level of safety. Design procedures developed by engineers to provide acceptable margins of safety include the following: (1) Allowable Stress Design (ASD) using a single global factor of safety; (2) Limit State Design (LSD) using partial factors of safety; and (3) reliability-based Load and Resistance Factor Design (LRFD).

Allowable Stress Design (ASD) has been the traditional design basis in civil engineering since it was first introduced in the early 1800's. Up until today, in most of the design code, the foundation design has been based on ASD. In ASD, a global factor of safety is applied to the resistance such that the estimated stresses (or loads) do not exceed the reduced resistances. The relationship can be expressed as:

$$\frac{R_n}{FS} \geq Q_D + Q_L + Q_E \tag{2.1}$$

Where R_n is the nominal resistance, Q_D and Q_L are nominal values of dead and live load, Q_E is the environmental load such as wind, earthquake, etc. and FS is the factor of safety. Although this concept is simple and useful, the risk or level of safety associated with a value of FS depends on its definition and application , and a computed value of FS greater than one does not necessarily ensure safety (Smith 1981, 1985).

Limit State Design (LSD) has received increasing attention in geotechnical and structural engineering literature over the last 20 years. In LSD, two limit states are considered: (1) Ultimate Limit States (ULS); (2) Serviceability Limit States (SLS). ULS pertains to structural safety and applies separate partial factors of safety on loads and strengths. SLS represents the conditions which affect the function or services requirement. An advantage of LSD is that it provides a clearer methodology for the separation of the ULS and SLS. LSD is utilized to satisfy the following criteria:

ULS: Factored resistance $\geq$ Factored load effects

SLS: Deformation $\leq$ Tolerable deformation to remain serviceable

However, in the United States, geotechnical engineers practically have not adopted this concept.

Load and Resistance Factor Design (LRFD) utilizes the concept of partial factors and can be viewed as an extension of the LSD. However, the factors utilized in LSD were determined by experience and judgement, while the factors of LRFD are based on or calibrated using probability and reliability theory. The LRFD criterion is expressed in the following general form:

$$\Phi R_n \geq \sum \gamma_i Q_i \tag{2.2}$$

where ϕR_n is the factored resistance.

ϕ is the resistance factor that considers the uncertainties in resistance.

R_n is the nominal resistance estimated from engineering analyses.

$\sum \gamma_i Q_i$ is the factored loads

γ_i is a load factor that considers the uncertainties in a component of load effects.

Q_i is a component of load.

The resistance factor, ϕ, is similar in concept to the global factor of safety, and the factored resistance approach is similar to conventional ASD and may be viewed as a logical extension of ASD. The LRFD method has several advantages over the conventional ASD method including:

1. Considers the uncertainties and variability in both loads and resistances.
2. Provides more uniform levels of safety for various type of structures and materials.
3. Provides similar design concepts and procedures for various superstructures and substructures.

The primary objective of this research is to determine the resistance factor used in the LRFD method for various geotechnical engineering and foundation design application for FDOT Projects. This will provide a smooth transition for geotechnical engineers from the ASD method to the LRFD method. In general, two procedures are used to calibrate the resistance factor; fitting with ASD and using reliability. The evaluation of each procedure is as follows.

2.2 Calibration by Fitting with ASD

Because the LRFD method is a reliability-based methodology, the calibration of the resistance factor requires a sufficient database to perform probability analyses. However, when a database or case histories are not available, fitting with ASD to determine the resistance factor is an alternative. This procedure simply converts the global factor of safety to the resistance factor as follows.

If the loads consist of a dead load, Q_D and a live load, Q_L, then from Equations 2.1 and 2.2, the resistance factor, ϕ can be expressed as:

$$\Phi \geq \frac{\gamma_D Q_D + \gamma_L Q_L}{FS(Q_D + Q_L)} \tag{2.3}$$

Equation 2.3 may be written as:

$$\Phi \geq \frac{\gamma_D Q_D / Q_L + \gamma_L}{FS(Q_D / Q_L + 1)} \tag{2.4}$$

where Υ_D and Υ_L are the load factors of dead load and live load, respectively. Using Equation 2.4 and the values of Υ_D and Υ_L from the AASHTO LRFD Specification (1994), 1.25 and 1.75, respectively, the resistance factor, ϕ, for a range of safety factors and dead to live load ratios are calculated as shown in Table 2.1 and Figure 2.1. Although the fitting with ASD method is relatively simple and ensure that the results of LRFD method will be compatible with the conventional ASD, the risk or probability of failure in using this method is unknown.

2.3 Calibration using Reliability Theory

The reliability theory provides a valuable tool to compute the risk level or probability of failure in existing or new design codes. Based on the reliability theory, the calibration of the resistance factors corresponding to a given set of load factors consists of the following steps; for the detailed discussions about the reliability theory and applications refer to Barker, et al (1991) and Tobias and Trindade (1996).

(1) Estimate the level of safety or inherent reliability in current design methods.

In the reliability model, loads and resistance are considered to be random variables that can be described by probability density functions or frequency distributions. As long as the resistance, R, is greater than the load effects, Q, there exists a margin of safety for the limit states under consideration. The probability of failure or the realization of a limit state, P_f, can be expressed as

$$P_f = P(R<Q) = P[(R-Q)<0] \tag{2.5}$$

If R and Q are assumed to be log-normal distributions, Equation 2.5 can be rewritten as

$$P_f = P[\ln(R/Q)<0] = 1 - F_u\left[\frac{\ln[(\bar{R}/\bar{Q})\sqrt{(1+V_R^2)/(1+V_Q^2)}]}{\sqrt{\ln[(1+V_R^2)(1+V_Q^2)]}}\right] \tag{2.6}$$

where $\bar{R}$ and $\bar{Q}$ are the mean values, V_R and V_Q are the coefficients of variation (standard deviation divided by mean value) of R and Q, and $F_u()$ is the standard normal distribution function. Instead of specifying a probability of failure, a common approach is to express reliability in terms of a safety or reliability index, β as follows.

$$\beta = \frac{\ln[(\bar{R}/\bar{Q})\sqrt{(1+V_R^2)/(1+V_Q^2)}]}{\sqrt{\ln[(1+V_R^2)(1+V_Q^2)]}} \tag{2.7}$$

If the bias factor, λ, defined as the ratio of the mean value to the nominal value, and the loads consist of only dead and live loads, Equation 2.7 can be rewritten as:

$$\beta = \frac{\ln\left[\frac{\lambda_R \; FS \; (Q_D/Q_L+1)}{\lambda_{QD} \; Q_D/Q_L+\lambda_{QL}}\sqrt{\frac{1+V_{QD}^2+V_{QL}^2}{1+V_R^2}}\right]}{\sqrt{\ln[(1+V_R^2)(1+V_{QD}^2+V_{QL}^2)]}} \tag{2.8}$$

Where Q_D and Q_L are the nominal values of the dead and live loads, FS is the factor of safety, λ_R, λ_{QD}

and λ_{QL} are the bias factors of the resistance, dead load and live load, respectively, V_R, V_{QD} and V_{QL} are the coefficients of variation of the resistance, dead load and live load, respectively.

Table 2.2 shows the relationship between reliability index, β (for $2<\beta<6$) and probability of failure, P_f as suggested by Rosenblueth and Esteva (1972):

$$P_f = 460\exp(-4.3\beta) \tag{2.9}$$

Based on Equations 2.8 and 2.9 and the coefficients of variation of R and Q determined from probability analyses, the reliability index and probability of failure of the corresponding factor of safety, FS can be calculated.

(2) Observe the Variation of the Reliability Indices.

As shown in Equation 2.8, the reliability index is not only affected by the uncertainty of the load effects and soil resistances, also by the ratio of dead load to live load. In order to evaluate the effect of the ratio of dead to live load which depends on the type of structures, a range of ratio from one (1) to nine (9) will be used in calibration process .

(3) Select a target reliability index based on the level of safety or probability of failure used in the current design method.

Following Step (1), in which the reliability index was calculated based on the level of safety of the current design method, the target reliability index will be used for calibration of the resistance factor. This will ensure that the designs of LRFD method will not have significant deviation from the current ASD method.

(4) Calculate resistance factors consistent with the selected target reliability index.

From Equations 2.2 and 2.8, in which for a given target reliability index, β_T, and only dead and live loads are considered, the resistance factor can be expressed as

$$\phi = \frac{\lambda_R(\gamma_D Q_D/Q_L + \gamma_L)\sqrt{(1+V_{QD}^2+V_{QL}^2)/(1+V_R^2)}}{(\lambda_{QD}Q_D/Q_L+\lambda_{QL})\exp[\beta_T\sqrt{\ln((1+V_R^2)(1+V_{QD}^2+V_{QL}^2))}]} \tag{2.10}$$

The resistance factor calculated from Equation 2.10 can be used to validate or verify the new design approach by comparing designs based on calibrated resistance factors with designs obtained from conventional ASD. Recalibrating and modifying the resistance factors may be required.

Table 2.1
Resistance Factors Calibrated by Fitting with ASD
$\Upsilon_D = 1.25$ and $\Upsilon_L = 1.75$

Q_D/Q_L	Resistance Factor, ϕ			
	FS=1.5	FS=2.0	FS=2.5	FS=3.0
1	1.00	0.75	0.60	0.50
2	0.94	0.71	0.57	0.47
3	0.92	0.69	0.55	0.46
4	0.90	0.68	0.54	0.45
5	0.89	0.67	0.53	0.44
6	0.88	0.66	0.53	0.44
7	0.88	0.66	0.53	0.44
8	0.87	0.65	0.52	0.44
9	0.87	0.65	0.52	0.43
Median	0.94	0.70	0.56	0.47
Recommended	0.90	0.65	0.55	0.45

Table 2.2
Relationship between Probability of Failure and Reliability Index for Lognormal Distribution (from Barker, et al , 1991)

Reliability Index β	Probability of Failure P_f		Probability of Failure P_f	Reliability Index β
2.5	0.99×10^{-2}		1.0×10^{-1}	1.96
3.0	1.15×10^{-3}		1.0×10^{-2}	2.50
3.5	1.34×10^{-4}		1.0×10^{-3}	3.03
4.0	1.56×10^{-5}		1.0×10^{-4}	3.57
4.5	1.82×10^{-6}		1.0×10^{-5}	4.10
5.0	2.12×10^{-7}		1.0×10^{-6}	4.64
5.5	2.46×10^{-8}		1.0×10^{-7}	5.17

CHAPTER 3
DRIVEN PILE FOUNDATION DESIGN

3.1 General

In the calibration process, the load factors for dead and live loads as specified in the AASHTO LRFD Bridge Design Specification (1994) are 1.25 and 1.75, respectively. The bias factors and coefficients of variation of the load effects depend on the materials and manufacturing of the load components. Table 3.1 summarized the results of statistic study of highway structure dead and live loads by Nowak (1993). To simplify the unknown characteristics of the structure components of the database, the bias factor and coefficient of variation of the dead load for cast-in-place load components, 1.05 and 0.10, respectively, will be assumed in the calibration process. For live loads, 1.10 and 0.18 will be used for the bias factor and coefficient of variation, respectively. These conditions represent an average worse case.

3.2 Service Limits

The axial load-settlement analysis of driven pile can be performed by the Computer Program PL-AID developed by University of Florida (1990), while the lateral load-deflection analysis can be performed by the Computer Program FLPIER developed by University of Florida. The evaluation of foundation movements using LRFD method is performed in accordance to Service I Limit State with load factor and resistance factor equal to one (1).

The criteria of tolerable axial and lateral movement of deep foundations usually are developed by the structural engineer in accordance with the design characteristics of the superstructures. In general, the structural stability of the entire bridge under the lateral load is the major concern, and not the absolute value of the movement of bridge foundations. It is strongly recommended that a FLPIER analysis should be performed in final design to evaluate the stability of bridge structure subjected to external loads. A detailed discussion of the lateral load stability is presented in Section 3.4

3.3 Axial Load Capacity

Currently, FDOT uses computer program SPT-94 or SPT-97 to estimated the bearing capacity of driven pile in design phase, and uses static load testing and/or dynamic load testing with Pile Driving Analyzer (PDA) monitoring to determine the bearing capacity and provide quality control of pile installation in construction phase. Wave Equation Analysis predominately is used to evaluate the drivability of pile hammer in pre-construction phase.

3.3.1 SPT94/SPT97 Analysis

Although FDOT has released a new version SPT-97 in 1998, the computer program used in predicting driven pile capacity of the pile database in this research was SPT-94. The Computer program SPT94 is a DOS version program, while program SPT97 is a WINDOWS version program. The differences between two programs are the pre-and post- processors Both programs were developed based on the procedures outlined in the FDOT Research Bulletin 121 (1967) and should pose the same features to estimate the bearing capacity of driven pile.

Based on the database collected by the University of Florida and the results of statistic Analyses, the bias factor and coefficient of variation for the ratio of load test result (Davisson capacity) to SPT-94 prediction are 1.172 and 0.246, respectively. Figure 3.1 shows the comparison of the database of static load test and the corresponding Davisson capacity predicted by SPT-94 analyses. The frequence histogram of the results of SPT-94 analyses is presented in Figure 3.2. Using the above statistic parameters and a safety factor of 2 for the estimated Davisson capacity, the corresponding reliability index, β ranged from 2.45 to 2.57 for a ratio of dead load to live load ranging from 1 to 9. The detail results were shown in Table 3.2 and Figure 3.3. Table 3.3 and Figure 3.4 present the calibrated resistance factors for different reliability indices with a ratio of dead load to live load ranging from 1 to 9. According to AASHTO Specifications (1994), the ratio of dead load to live load ranged from 0.52 to 3.53 for a typical bridge span ranged from 9 to 60 meters. The median of the reliability index within this range of load ratio is approximately 2.5 which has a probability of failure of approximately 10^{-2}. This indicated that in the current FDOT practice, the reliability index of using SPT-94/SPT-97 to estimate driven pile capacity is on the order of 2.5 which represented a failure probability of approximately 1 %.

For the target reliability index of 2.5, the resistance factors ranged from 0.65 to 0.72 as shown in Table 3.3. The resistance factors calculate by fitting ASD for a safety factor of 2 ranged from 0.65 to 0.75 as shown in Table 2.1. Therefore, based on the results of the calibration by reliability analysis and ASD fitting in conjunction with the engineering judgements, a resistance factor of 0.65 is selected for using the Program SPT-94/SPT-97 to estimate the axial compression capacity of driven pile.

3.3.2 Static Load Testing

According the FDOT Standard Specification for Road and Bridge Construction Section A455, a safety factor of 2 shall be used for the project when performing conventional static load tests. Using the load factors specified in the AASHTO LRFD Bridge Design Specification (1994) and the statistic parameters of load effects proposed by Barker, et al. (1991), the corresponding reliability index of the current FDOT practice for a safety factor of 2 is 2.73 to 2.89 for a ration of dead load to live load ranging from 1 to 9 as shown in Table 3.4 and Figure 3.5. The median of the reliability index is 2.81 which represents a probability of failure of 2.6×10^{-3}. This calculation assumed that the bias factor and coefficient of variation are 1.0 and 0.1, respectively. According to Becker (1996b), a reasonable range of coefficient of variation of pile load test generally is from 0.08 to 0.25.

For the target reliability index of 2.5, the resistance factors ranged from 0.73 to 0.79 for a ratio of dead load to live load ranging from 1 to 4 as shown in Table 3.5 and Figure 3.6. The resistance factors calculated by ASD fitting for a safety factor of 2 ranged from 0.65 to 0.75 for a ratio of dead load to live load ranging from 1 to 9 as shown in Table 2.1. In general, the ratios of dead load to live load for the majority of the highway bridge are on the order of 2 to 4. In addition, according to the ASSHTO LRFD Bridge Design Specifications, a specific load combination of Strength IV is designed for load combination relating to very high dead load to live load force effect ratio. Therefore, based on the results of the calibration by reliability analysis and ASD fitting in conjunction with the engineering judgements, a resistance factor of 0.75 is selected for using the conventional static load test to estimate the axial compression capacity of driven pile.

3.3.3 Dynamic Load Test with Driving Analyzer (PDA) Monitoring

A majority of the current FDOT bridge construction projects require dynamic load testing be performed on test piles by using Pile Driving Analyzer (PDA) monitoring. The required safety factor is 2.5 according to the FDOT Standard Specifications for Road and Bridge Construction Section A455. During the test pile installation phase, two driving conditions are evaluated; the end of initial driving (EOD) and beginning of redriving (BOR).

Based on the database collected by the University of Florida and the results of probability analyses, the bias factor and coefficient of variation for the ratio of load test result (Davisson capacity) to PDA-EOD prediction are 1.355 and 0.325, respectively, for the PDA-BOR prediction are 1.052 and 0.318, respectively. Figure 3.7 and 3.8 show the comparison of the database of static load test and the corresponding PDA-EOD and PDA-BOR capacity. The frequence histogram of the results of PDA-EOD and PDA-BOR predictions are shown in Figures 3.9 and 3.10, respectively. The load factors for dead and live loads as specified in the AASHTO LRFD Bridge Design Specification (1994) are 1.25 and 1.75, respectively. Barker, et al (1991) recommended that the bias factors for dead and live loads be 1.05 and 1.15, respectively, and the coefficients of variation for dead and live loads be 0.1 and 0.18, respectively. Using the statistic parameters of load effects and a safety factor of 2.5 for the PDA capacity, the corresponding reliability index, β of the current FDOT practices ranged from 3.04 to 3.14 for the PDA-EOD condition and 2.40 to 2.50 for the PDA-BOR condition for a ratio of dead load to live load ranging from 1 to 9. The detail results were shown in Table 3.6 and Figure 3.11. The true pile bearing capacity determined by the PDA-EOD results are usually underestimates as compared to the static load test results because of lacking freeze effects. Therefore, if the PDA-EOD values are used for the design, the probability of failure will be small. However, the PDA-BOR values usually are closer to the static load test results because the pile freeze effects are included. Therefore, if the higher value of PDA-BOR results are used for design, the corresponding probability of failures will be higher than that of PDA-EOD results. Based on the results of the reliability analysis, the medians of the reliability index of PDA-EOD and PDA-BOR conditions are 3.09 and 2.45, respectively, which corresponds to a probability of failure of approximately 7.8×10^{-4} and 1.2×10^{-2}, respectively.

Using a target reliability index of 2.5, the resistance factors ranged from 0.65 to 0.70 for

PDA-EOD and 0.51 to 0.55 for PDA-BOR with a ratio of dead load to live load ranging from 1 to 4. The detail results were shown in Table 3.7 and Figure 3.12 and 3.13. The resistance factors calculated by fitting ASD for a safety factor of 2.5 ranged from 0.52 to 0.60 as shown in Table 2.1. In general, the ratios of dead load to live load for the majority of the highway bridge are on the order of 2 to 4. In addition, according to the ASSHTO LRFD Bridge Design Specifications, a specific load combination of Strength IV is designed for load combination relating to very high dead load to live load force effect ratio. Therefore, based on the results of the calibration by reliability analysis and ASD fitting in conjunction with the engineering judgements, a resistance factor of 0.65 is selected for using the PDA-EOD condition of dynamic load test to estimate the axial compression capacity of driven pile, and 0.55 for the PDA-BOR condition.

As shown in Table 3.6, the results of statistic analysis indicated that in average BOR condition provided a more accurate prediction of pile capacity with a bias factor, λ of 1.052, while the EOD condition underestimated the pile capacity with λ of 1.552. Therefore, a higher resistance of 0.65 was calculated for EOD condition compared to that of 0.55 for BOR condition. However, due to the high uncertainty of the pile freeze effect, the use of PDA-BOR condition for the quality control of piling poses a higher risk compared to the PDA-EOD condition. Therefore, it is recommended that the PDA-EOD condition with a resistance factor of 0.65 be used as the general procedure of piling control, while the PDA-BOR condition with a resistance factor of 0.55 be used as a special case where pile freeze effects were well documented.

3.3.4 Wave Equation Analysis

Wave Equation Analyses were performed on the database where pile driving data and hammer data were available. The computer program used in calibration was GRLWEAP developed by GRL, Inc. In general Wave Equation Analysis was performed in pre-construction phase where actual hammer efficiency and Smith soil parameters were not available, therefore, the analysis used the default values of the hammer efficiency and soil damping and quake specified in the Program GRLWEAP(1996). Based on the database collected by the University of Florida and the results of probability analyses, the bias factor and coefficient of variation for the ratio of load test result (Davisson capacity) to GRLWEAP prediction are 1.355 and 0.606, respectively. Figure 3.14 shows the comparison of the database of static load test and the corresponding Davisson capacity predicted by GRLWEAP analyses. Using the above statistic parameters and a safety factor of 3 for the estimated Davisson capacity, the corresponding reliability index, β of the current FDOT practice ranged from 2.22 to 2.28 for a ratio of dead load to live load ranging from 1 to 9. The detail results were shown in Table 3.8 and Figure 3.15. The median of the reliability index is 2.25 corresponding to a probability of failure of approximately 3×10^{-2}. Table 3.9 and Figure 3.16 present the calibrated resistance factors for different reliability indices with a ratio of dead load to live load ranging from 1 to 9. For a target reliability index of 2.5, the resistance factors ranged from 0.33 to 0.36. The resistance factors calculate by fitting ASD for a safety factor of 3 ranged from 0.43 to 0.50 as shown in Table 2.1.

The resistance factor calibrated by the reliability theory is about 0.1 to 0.14 less than that calibrated by ASD Fitting. This is because of the highly variation of the default values of hammer efficiency and soil damping and quake compared to that of the actual values determined from PDA monitoring. Therefore, if PDA monitoring is not performed during pile driving, a resistance factor of 0.35 is selected for using the Program GRLWEAP to estimate the axial compression capacity of driven pile.

3.3.5 Other Considerations

In addition to the axial compression capacity, uplift capacity (axial tension capacity) and group effect should be considered in driven pile foundation design as discussed in the following sections..

3.3.5.1 Uplift capacity

Since very limited tension pile load tests were performed in Florida, the database of uplift capacity of driven piles is not available. Therefore, the calibration using reliability theory for the uplift capacity predicted by Program SPT-94 can not be performed. Barker, et al (1991) recommended that the resistance factor for the uplift capacity of driven pile is 0.1 less than that for the axial compression capacity. In ASD, the uplift capacity is usually estimated as 75% of the compression side friction. Comparing the safety factor of 2 used in the Program SPT-94 on the side friction, the equivalent safety factor of uplift capacity corresponding to the ultimate side friction is 2.67 (=2.0/0.75). The resistance factor calculated by fitting ASD using a safety factor of 2.67 ranged from 0.47 to 0.56. Therefore, a resistance factor of 0.55 is selected for using the Program SPT-94 to estimate the uplift capacity of driven pile.

The resistance factor of uplift capacity is selected as 0.65 for using the conventional static load test which is 0.1 less than that for the axial compression capacity of driven pile.

3.3.5.2 Pile group

According to FDOT Structure Design Guideline (1996), the minimum center-to-center spacing between piles in a group is three (3) pile diameters, and the pile group efficiency subjected to axial load is designed as one (1). Therefore, the resistance factor for pile group is selected as the value of single pile. For the group efficiency subjected to lateral loads, it is recommended that a FLPIER analysis should be performed to evaluate the group behavior.

3.4 Lateral Load Stability

For piles subjected to lateral loadings, it requires a relatively large deflections to mobilize passive failure of soils around the pile which generally exceed tolerable movements or structure capacity. Therefore, driven pile foundation must be designed to resist lateral loads without structural failure of the pile, or without excessive lateral deflection to ensure the structural stability. The ultimate soil resistance to the lateral loads usually is not critical. In general, the design of lateral load resistance of driven piles can be accomplished using computer programs FLPIER (1996) or COM624P (1992) for single pile and FLPIER for pile group, and should include:

1. Determine the maximum pile top deflection and structure stability.

   The pile deflection should be evaluated at the Service Limit State in which Load Factor, γ and Resistance Factor, ϕ equal to one (1). However, the allowable deflection should be evaluated by Structural Engineer based on the structure stability and an arbitrary criteria of allowable lateral deflection is not acceptable according to FDOT (Potter, 1995).

   To ensure the structure stability, pile should be installed to a required minimum penetration below the final grade or design scour elevation. FDOT State Geotechnical office has suggested a methodology to determine the minimum tip elevation. This method involves constructing a curve of pile top deflections for different pile lengths using the same unfactored load (service load) condition by performing FLPIER or COM624P analyses. Then, the minimum tip elevation is determined by increasing 1.5 meters on the tip elevation corresponding to the highest tip elevation that the pile top deflection start deviating from constant, as shown in Figure 3.17.

2. Determine the maximum moment along pile.

The moment acting in the pile should be evaluated at the Strength Limit State in which Load Factor, γ should properly be selected according to the load combinations of the design conditions, while the resistance factor, ϕ equals to one (1). The estimated maximum moment should compare with the factored structure resistance to ensure that structure failure will not occur.

However, if Structural Engineer determines that the pile top deflection or the maximum moment exceed the tolerable limit at the minimum pile penetration, a trial-and-true analysis should be performed to determine the minimum pile size required for structure stability. This analysis can be performed using single pile. However, it is strongly recommended that pile group analysis should be performed for final design.

3.5 Structure Capacity

In pile design using LRFD, the pile materials should provide sufficient factored resistance against the external factored loads. The Resistance Factor, ϕ presented in AASHTO Specifications (1994) was calibrated using ASD fitting. Since there were no data available to perform calibration, it is recommended that the Resistance Factor for different pile materials shown in Table 3.10 should be used..

Table 3.1
Statistics of Bridge Load Components
(After Nowak, 1993)

Load Component	Bias, λ	COV
Dead Load		
• Factor-Made	1.03	0.08
• Cast-in-Place (CIP)	1.05	0.10
• Asphaltic Wearing Surface	1.00	0.25
Live Load (w/Dynamic Load Allowance)	1.10-1.20	0.18

Table 3.2
Reliability Index of SPT-94 Analyses

Q_D/Q_L	β	V_R	λ_R	V_{QD}	λ_D	V_{QL}	λ_L	ASD FS
1	2.45	0.246	1.172	0.1	1.05	0.18	1.15	2.0
2	2.50	0.246	1.172	0.1	1.05	0.18	1.15	2.0
3	2.52	0.246	1.172	0.1	1.05	0.18	1.15	2.0
4	2.54	0.246	1.172	0.1	1.05	0.18	1.15	2.0
5	2.55	0.246	1.172	0.1	1.05	0.18	1.15	2.0
6	2.55	0.246	1.172	0.1	1.05	0.18	1.15	2.0
7	2.56	0.246	1.172	0.1	1.05	0.18	1.15	2.0
8	2.56	0.246	1.172	0.1	1.05	0.18	1.15	2.0
9	2.57	0.246	1.172	0.1	1.05	0.18	1.15	2.0

Table 3.3
Resistance Factors for SPT-94 Analysis

Q_D/Q_L	Resistance Factor, ϕ		
	$\beta = 2.0$	$\beta = 2.5$	$\beta = 3.0$
1	0.84	0.72	0.61
2	0.81	0.69	0.59
3	0.79	0.67	0.57
4	0.78	0.66	0.57
5	0.77	0.66	0.56
6	0.77	0.65	0.56
7	0.76	0.65	0.55
8	0.76	0.65	0.55
9	0.76	0.65	0.55

Table 3.4
Reliability Index of Conventional Static Load Testing

Q_D/Q_L	β	V_R	λ_R	V_{QD}	λ_D	V_{QL}	λ_L	ASD FS
1	2.73	0.1	1.0	0.1	1.05	0.18	1.15	2.0
2	2.79	0.1	1.0	0.1	1.05	0.18	1.15	2.0
3	2.83	0.1	1.0	0.1	1.05	0.18	1.15	2.0
4	2.85	0.1	1.0	0.1	1.05	0.18	1.15	2.0
5	2.86	0.1	1.0	0.1	1.05	0.18	1.15	2.0
6	2.87	0.1	1.0	0.1	1.05	0.18	1.15	2.0
7	2.88	0.1	1.0	0.1	1.05	0.18	1.15	2.0
8	2.88	0.1	1.0	0.1	1.05	0.18	1.15	2.0
9	2.89	0.1	1.0	0.1	1.05	0.18	1.15	2.0

Table 3.5
Resistance Factors for Conventional Static Load Testing

Q_D/Q_L	Resistance Factor, ϕ		
	$\beta = 2.0$	$\beta = 2.5$	$\beta = 3.0$
1	0.88	0.79	0.70
2	0.84	0.75	0.67
3	0.83	0.74	0.66
4	0.81	0.73	0.65
5	0.81	0.72	0.64
6	0.80	0.72	0.64
7	0.80	0.71	0.64
8	0.79	0.71	0.63
9	0.79	0.71	0.63

Table 3.6
Reliability Index of PDA Prediction

Q_D/Q_L	β		V_R		λ_R		V_{QD}	λ_D	V_{QL}	λ_L	ASD FS
	EOD	BOR	EOD	BOR	EOD	BOR					
1	3.04	2.40	0.325	0.318	1.355	1.052	0.1	1.05	0.18	1.15	2.5
2	3.08	2.44	0.325	0.318	1.355	1.052	0.1	1.05	0.18	1.15	2.5
3	3.10	2.46	0.325	0.318	1.355	1.052	0.1	1.05	0.18	1.15	2.5
4	3.11	2.47	0.325	0.318	1.355	1.052	0.1	1.05	0.18	1.15	2.5
5	3.12	2.48	0.325	0.318	1.355	1.052	0.1	1.05	0.18	1.15	2.5
6	3.13	2.49	0.325	0.318	1.355	1.052	0.1	1.05	0.18	1.15	2.5
7	3.13	2.49	0.325	0.318	1.355	1.052	0.1	1.05	0.18	1.15	2.5
8	3.13	2.49	0.325	0.318	1.355	1.052	0.1	1.05	0.18	1.15	2.5
9	3.14	2.50	0.325	0.318	1.355	1.052	0.1	1.05	0.18	1.15	2.5

Table 3.7
Resistance Factor for PDA Prediction

Q_D/Q_L	Resistance Factor, ϕ					
	EOD			BOR		
	β=2.0	β=2.5	β=3.0	β=2.0	β=2.5	β=3.0
1	0.85	0.70	0.58	0.66	0.55	0.46
2	0.81	0.67	0.56	0.64	0.53	0.44
3	0.79	0.66	0.54	0.62	0.52	0.43
4	0.78	0.65	0.54	0.61	0.51	0.42
5	0.77	0.64	0.53	0.61	0.51	0.42
6	0.77	0.64	0.53	0.60	0.50	0.42
7	0.77	0.63	0.53	0.60	0.50	0.41
8	0.76	0.63	0.52	0.60	0.50	0.41
9	0.76	0.63	0.52	0.60	0.50	0.41

Table 3.8
Reliability Index of WEAP Analyses

Q_D/Q_L	β	V_R	λ_R	V_{QD}	λ_D	V_{QL}	λ_L	ASD FS
1	2.22	0.606	1.355	0.1	1.05	0.18	1.15	3.0
2	2.25	0.606	1.355	0.1	1.05	0.18	1.15	3.0
3	2.26	0.606	1.355	0.1	1.05	0.18	1.15	3.0
4	2.27	0.606	1.355	0.1	1.05	0.18	1.15	3.0
5	2.27	0.606	1.355	0.1	1.05	0.18	1.15	3.0
6	2.28	0.606	1.355	0.1	1.05	0.18	1.15	3.0
7	2.28	0.606	1.355	0.1	1.05	0.18	1.15	3.0
8	2.28	0.606	1.355	0.1	1.05	0.18	1.15	3.0
9	2.28	0.606	1.355	0.1	1.05	0.18	1.15	3.0

Table 3.9
Resistance Factors for WEAP Analysis

Q_D/Q_L	Resistance Factor, ϕ		
	$\beta = 2.0$	$\beta = 2.5$	$\beta = 3.0$
1	0.49	0.36	0.27
2	0.47	0.35	0.26
3	0.46	0.34	0.25
4	0.45	0.34	0.25
5	0.45	0.33	0.25
6	0.45	0.33	0.25
7	0.44	0.33	0.24
8	0.44	0.33	0.24
9	0.44	0.33	0.24

Table 3.10

Resistance Factors for Structure Design of Driven Piles
(AASHTO, 1994)

Pile Material	Resistance Factor
Prestressed Concrete	0.45
Steel	0.35 - 0.45
Concrete-Filled Pipe Steel Pipe Concrete	 0.35 0.55
Timber	0.35

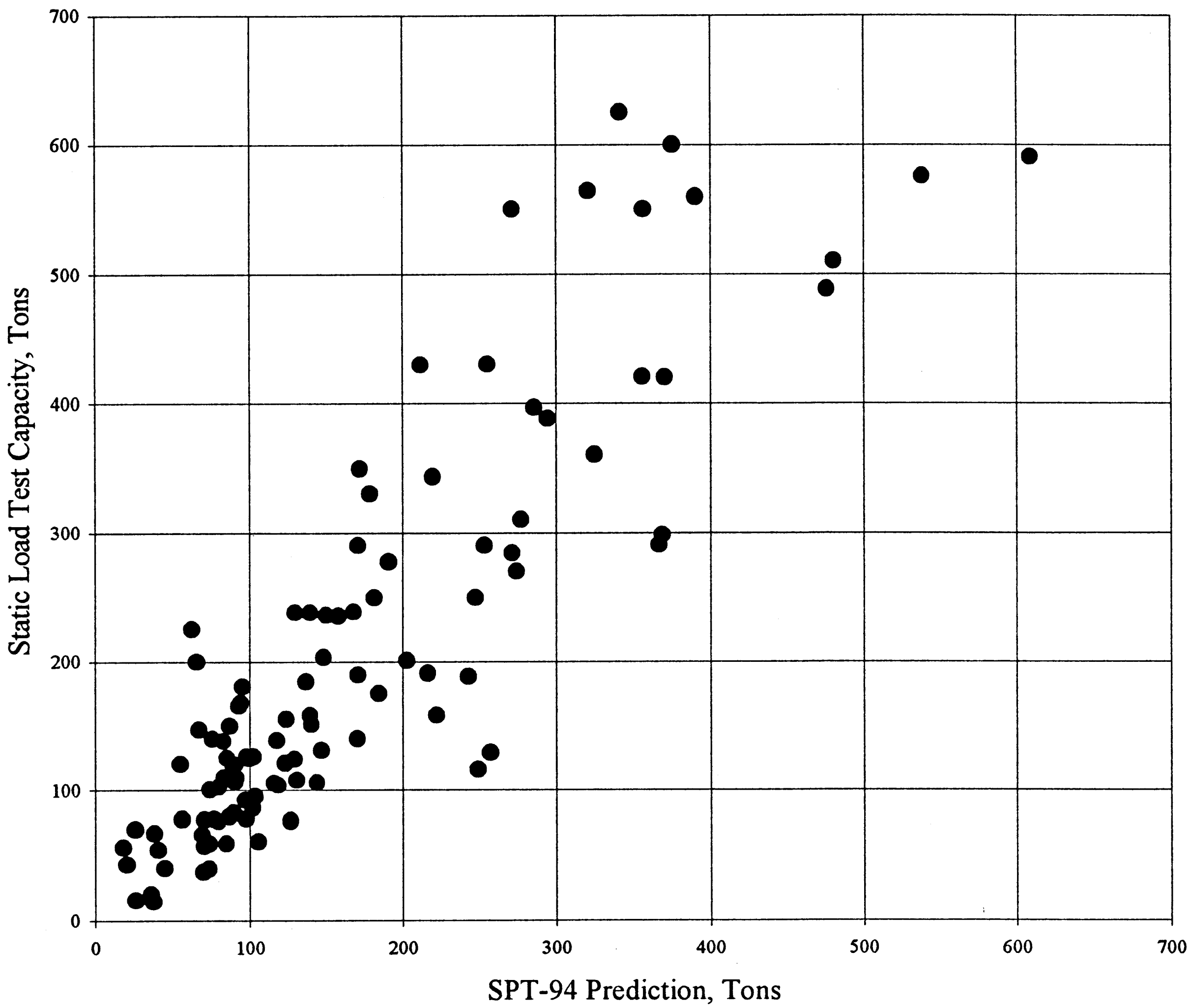


Figure 3.1 Static Load Test Capacity vs. SPT-94 Prediction

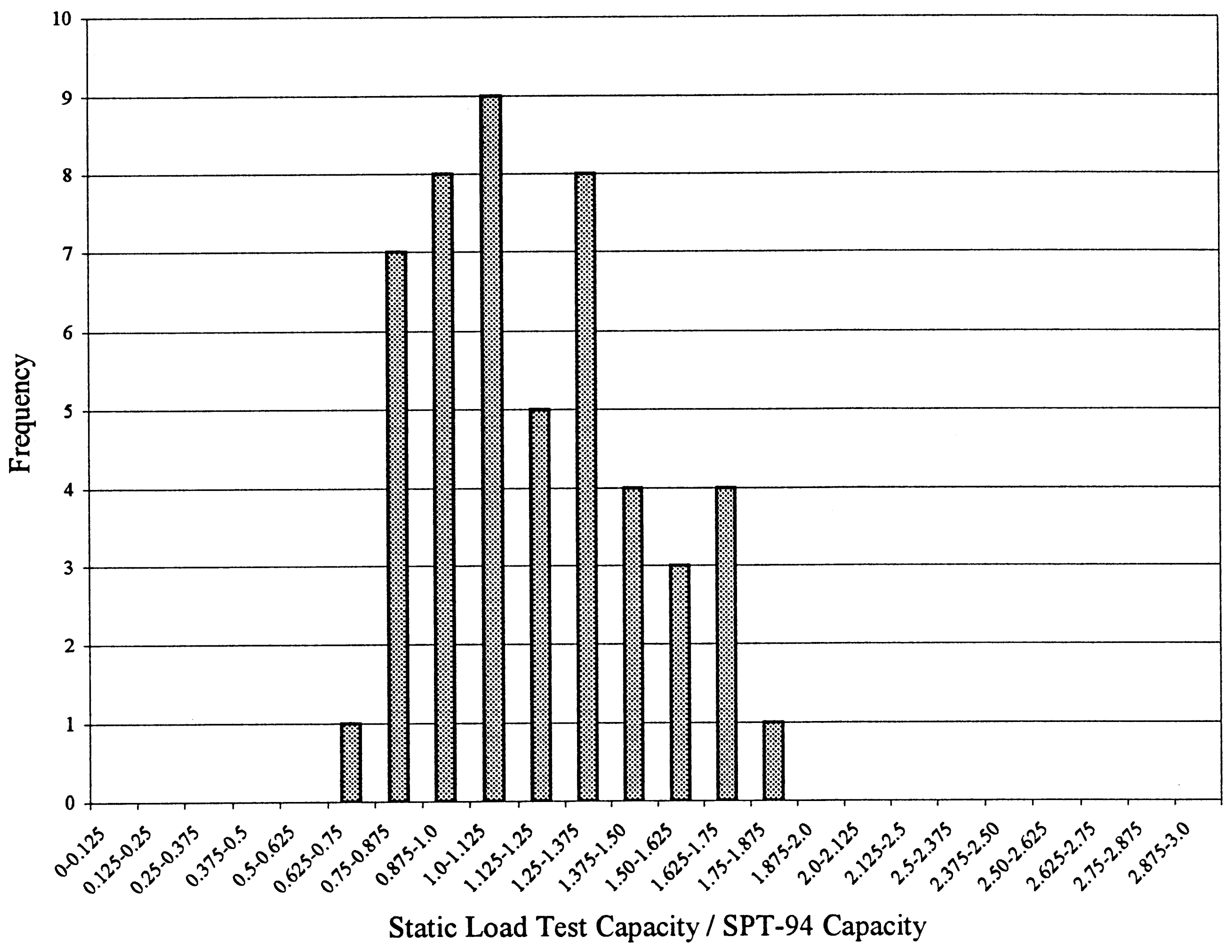


Figure 3.2 Frequence Histogram of SPT-94 Analysis

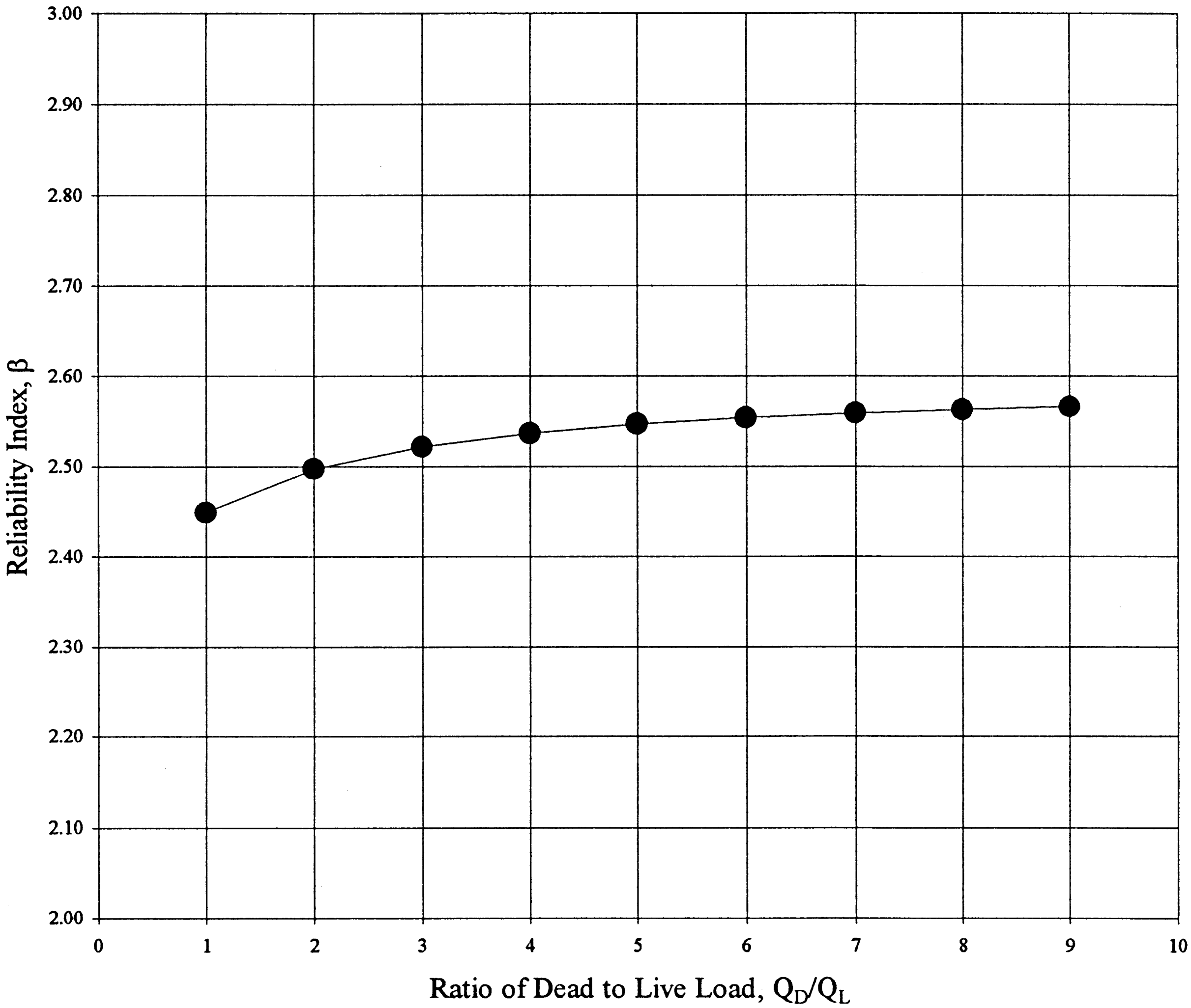


Figure 3.3 Reliability Index of SPT-94 Analysis for FS=2.0

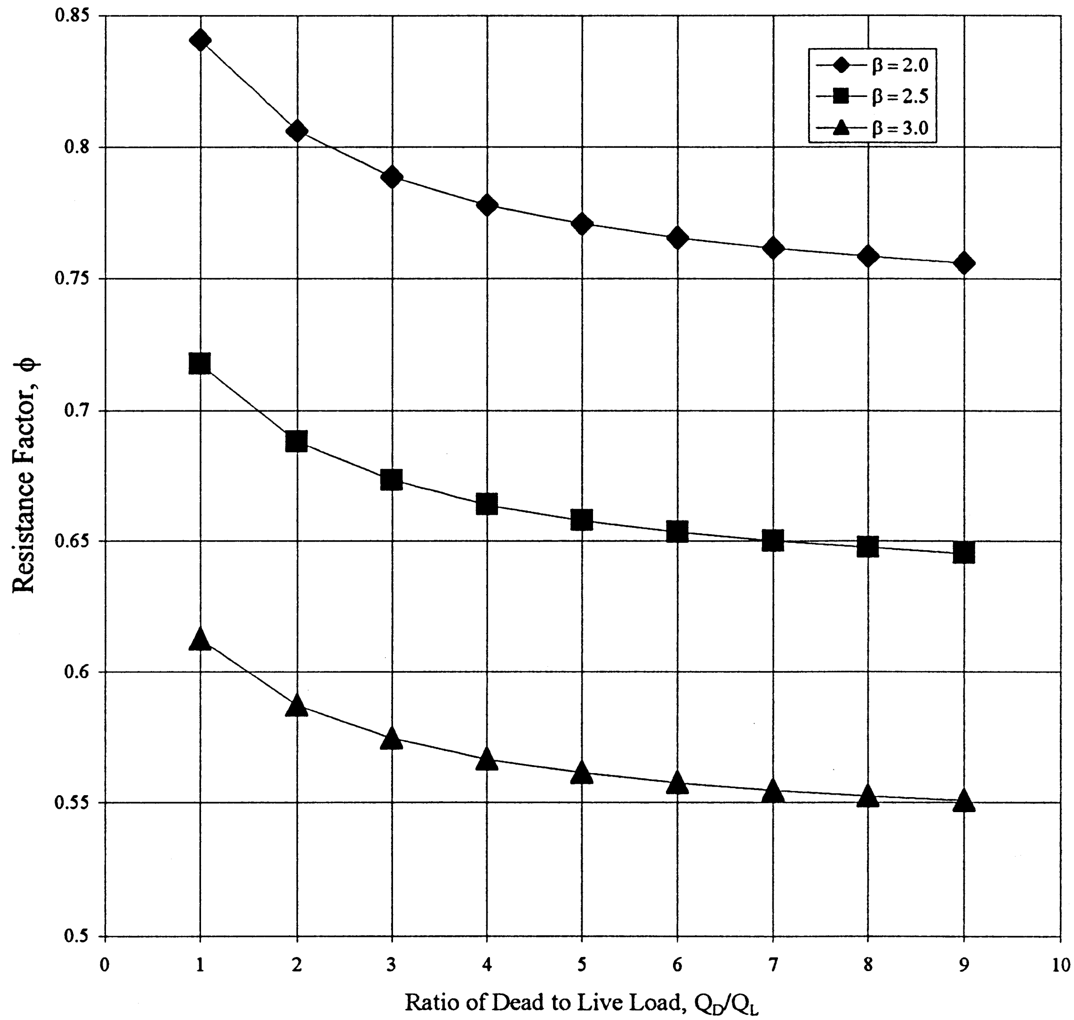


Figure 3.4 Calibrated Resistance Factor of SPT-94 Analysis

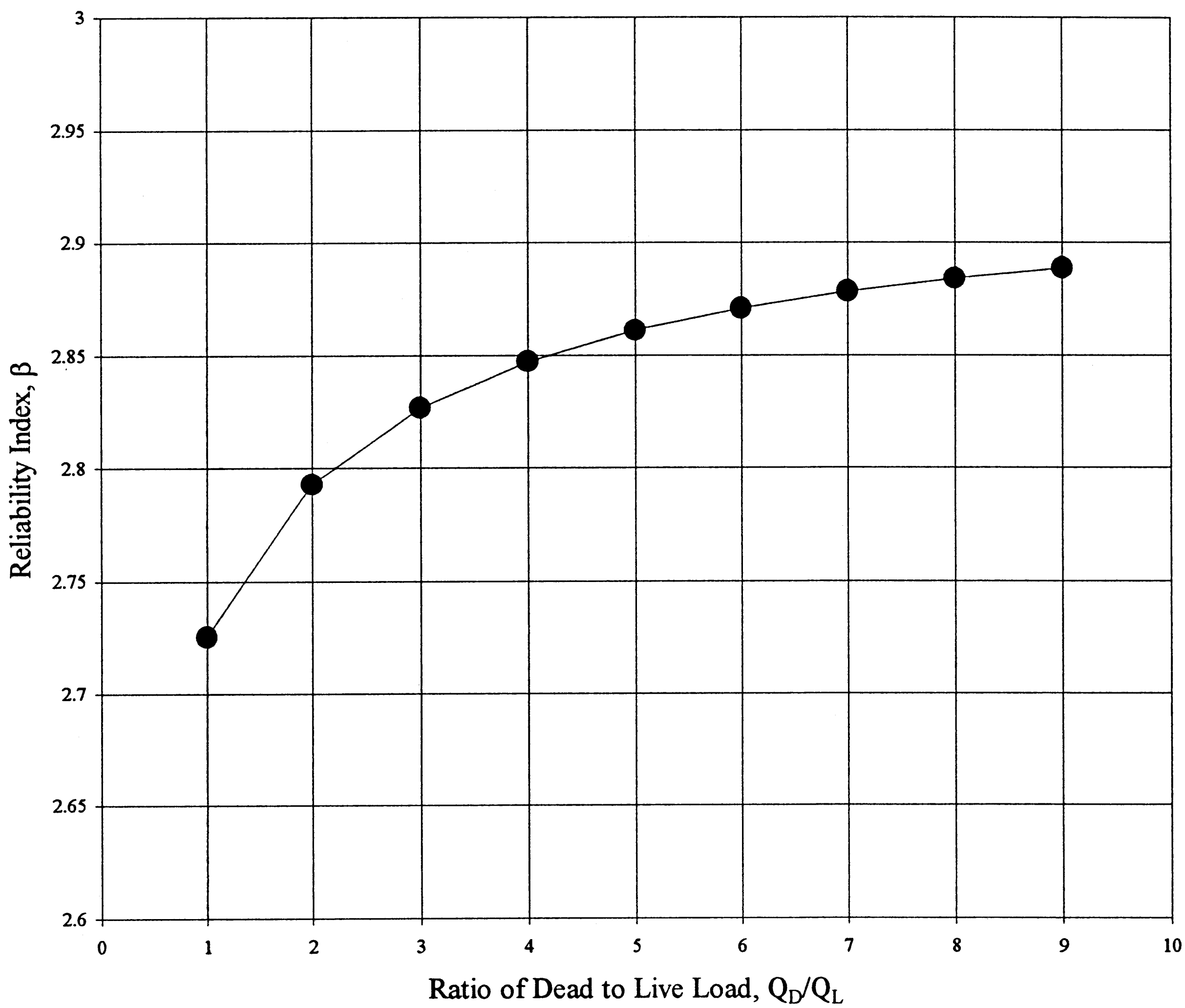


Figure 3.5 Reliability Index of Static Load Test for FS = 2.0

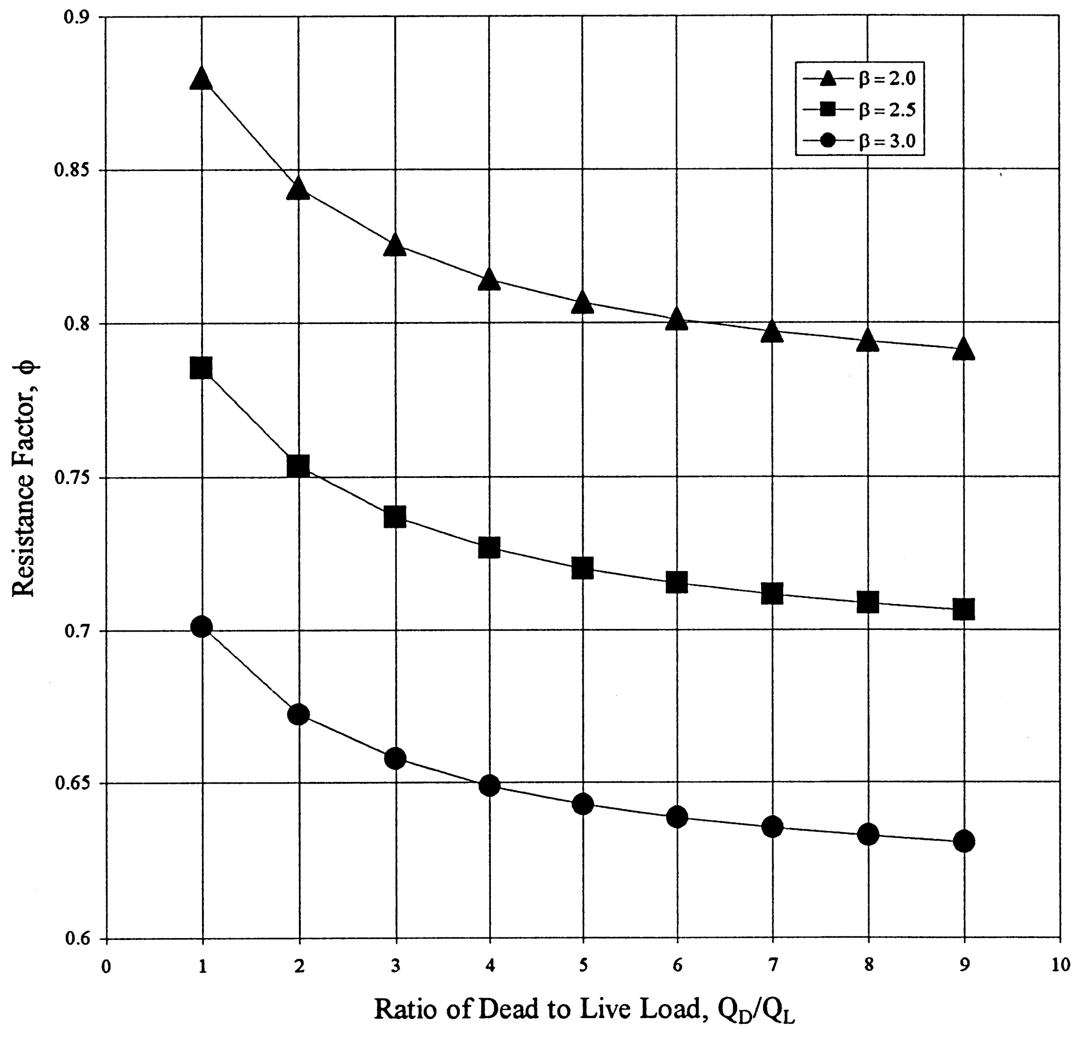


Figure 3.6 Calibrated Resistance Factor of Static Load Test

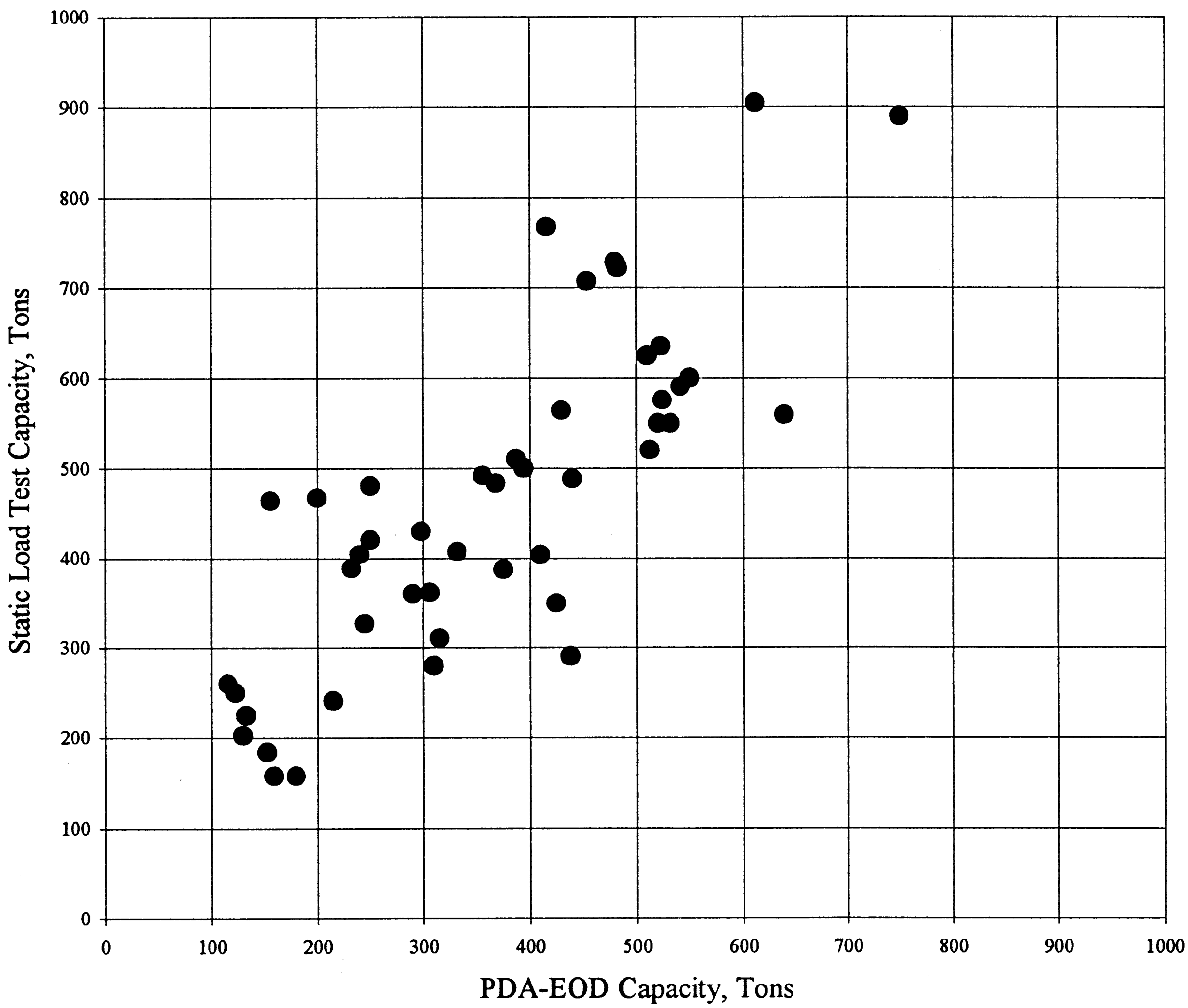


Figure 3.7 Static Load Test Results vs. PDA-EOD Capacity

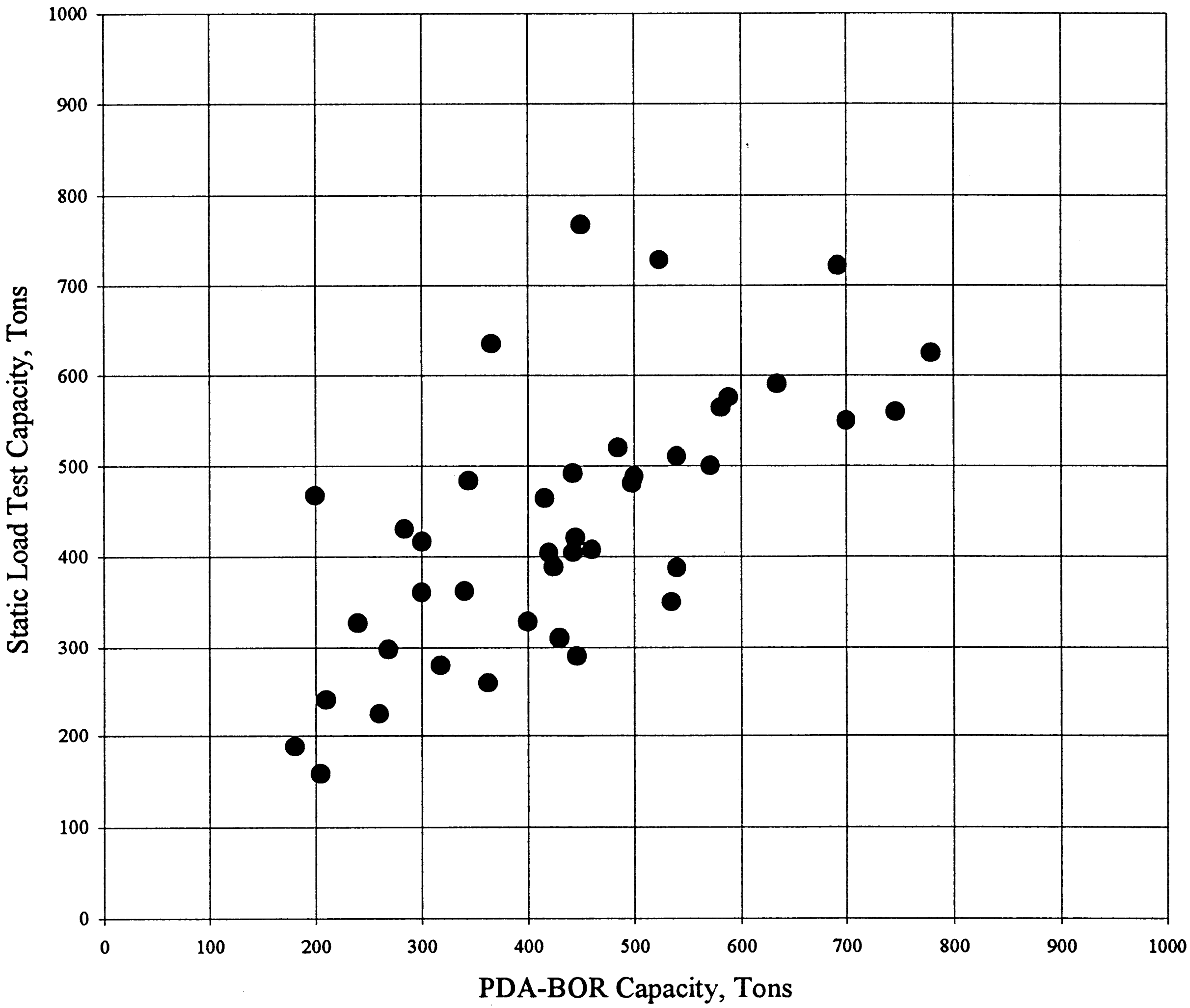


Figure 3.8 Static Load Test Results vs. PDA-BOR Capacity

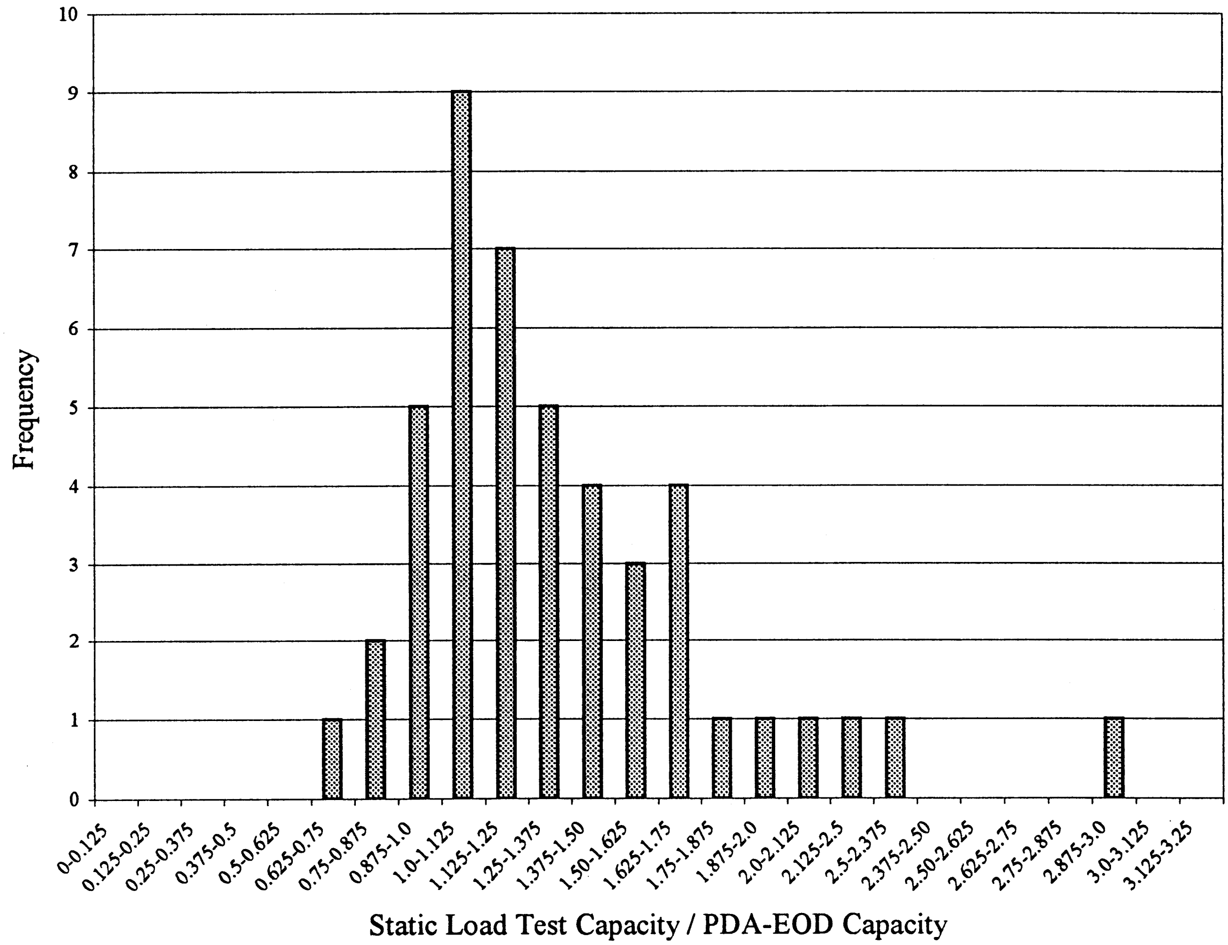


Figure 3.9 Frequence Histogram of PDA-EOD Prediction

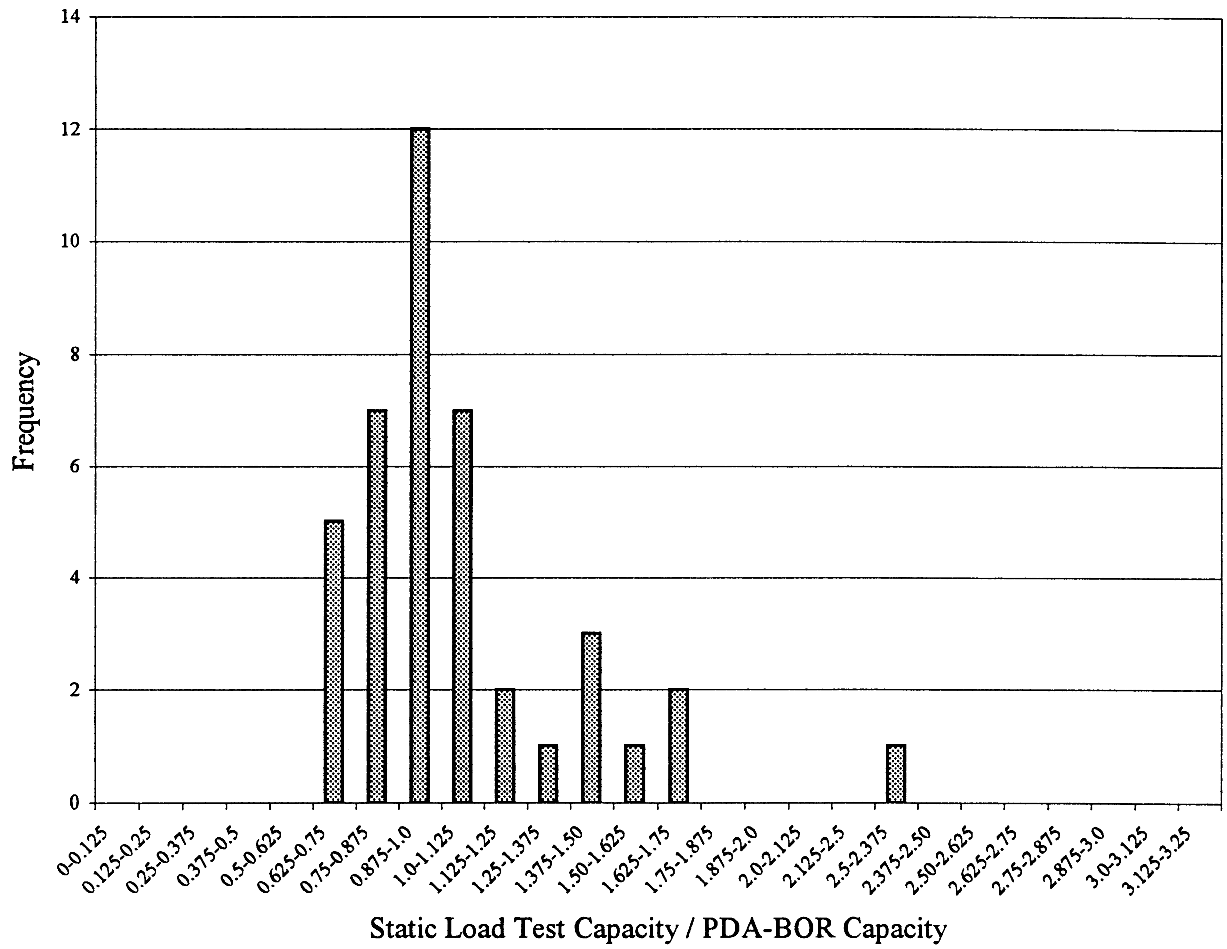


Figure 3.10 Frequence Histogram of PDA-BOR Prediction

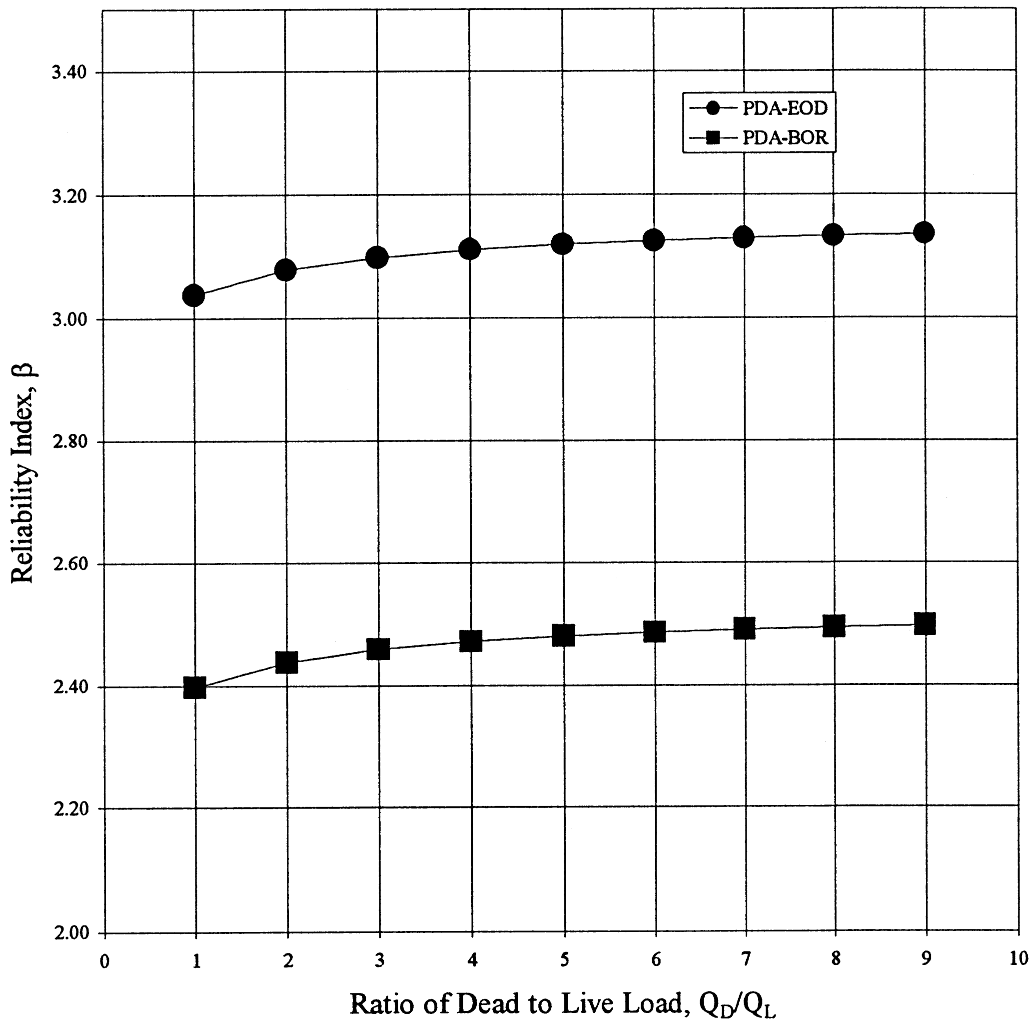


Figure 3.11 Reliability Index of PDA Prediction for FS = 2.5

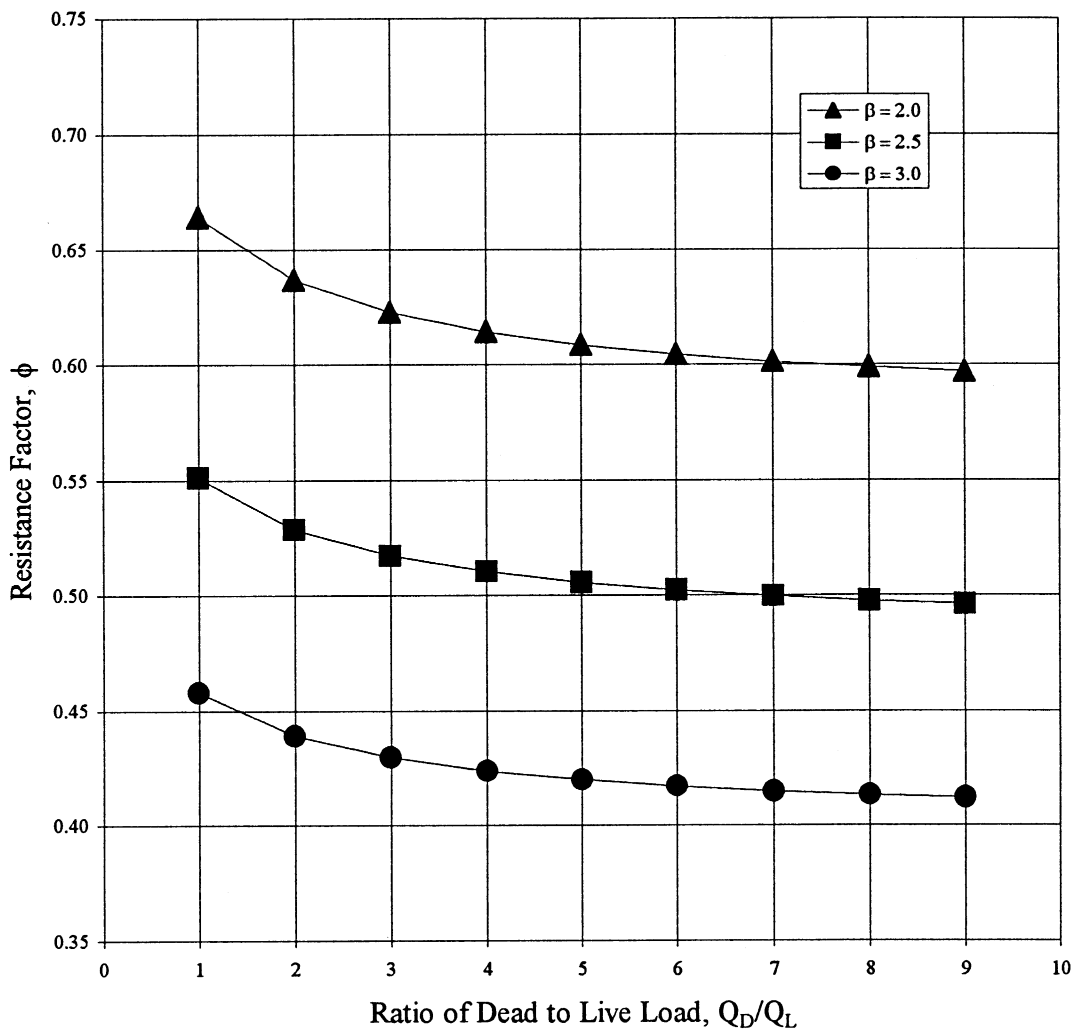


Figure 3.12 Calibrated Resistance Factor of PDA-BOR Prediction

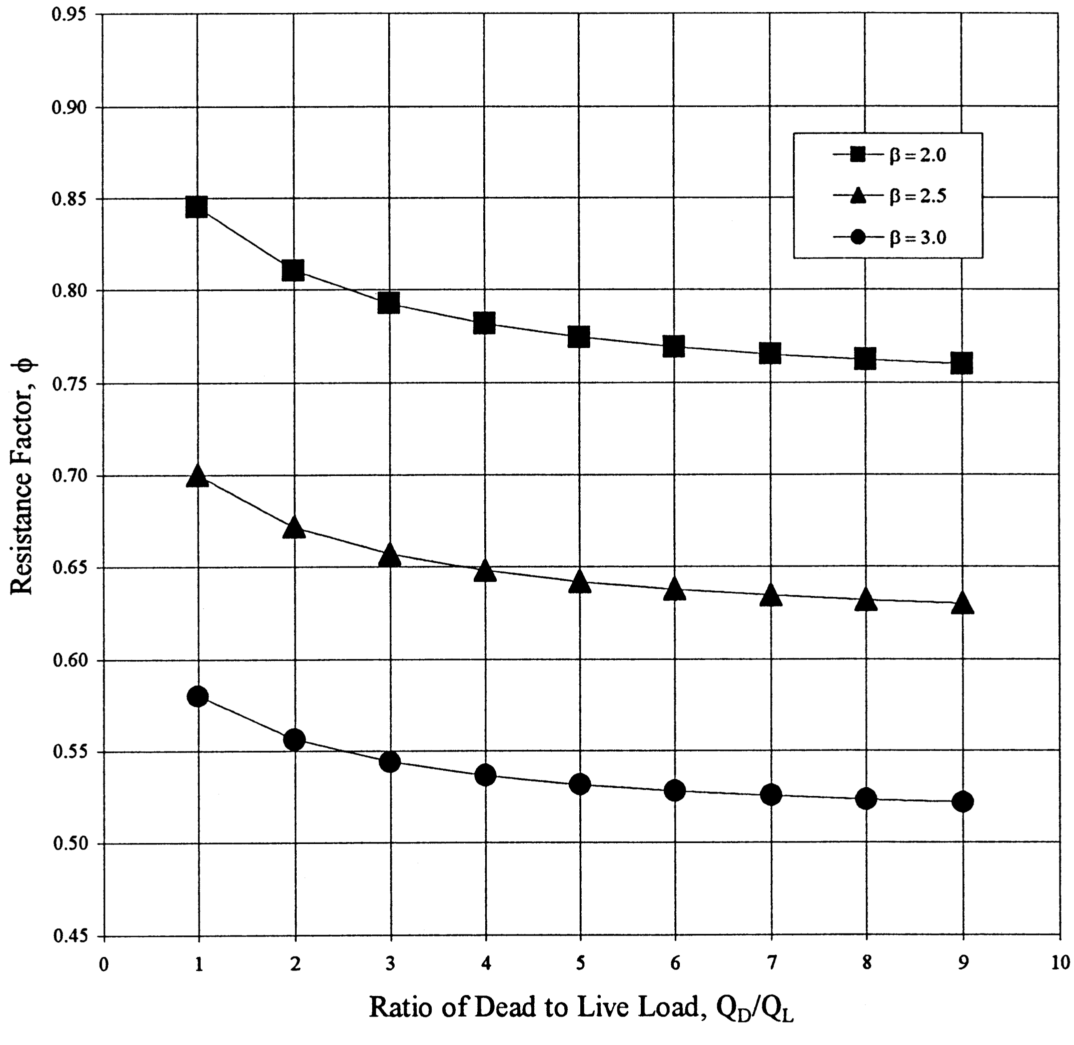


Figure 3.13 Calibrated Resistance Factor of PDA-EOD Prediction

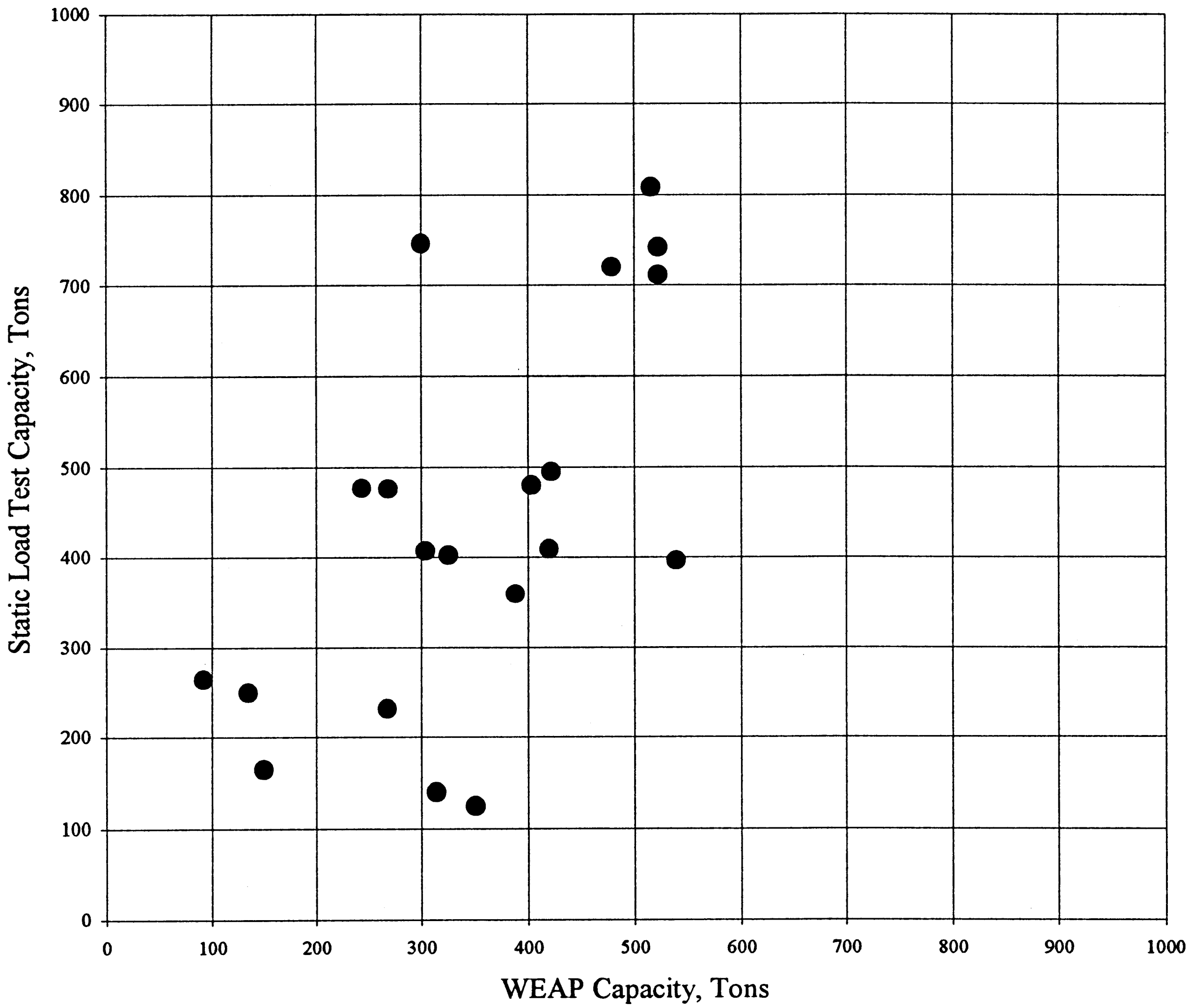


Figure 3.14 Static Load Test Capacity vs. WEAP Analysis

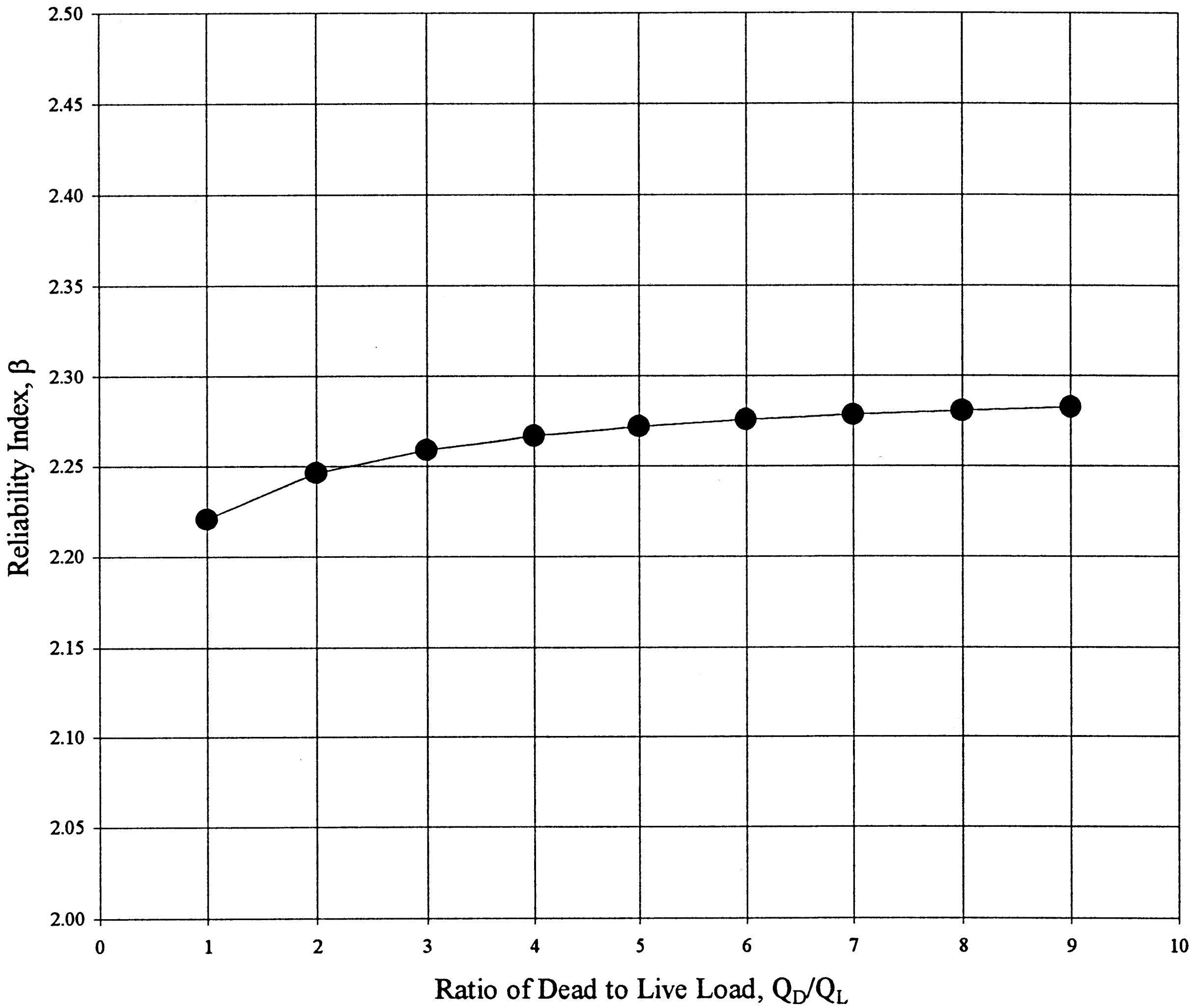


Figure 3.15 Reliability Index of WEAP Analysis

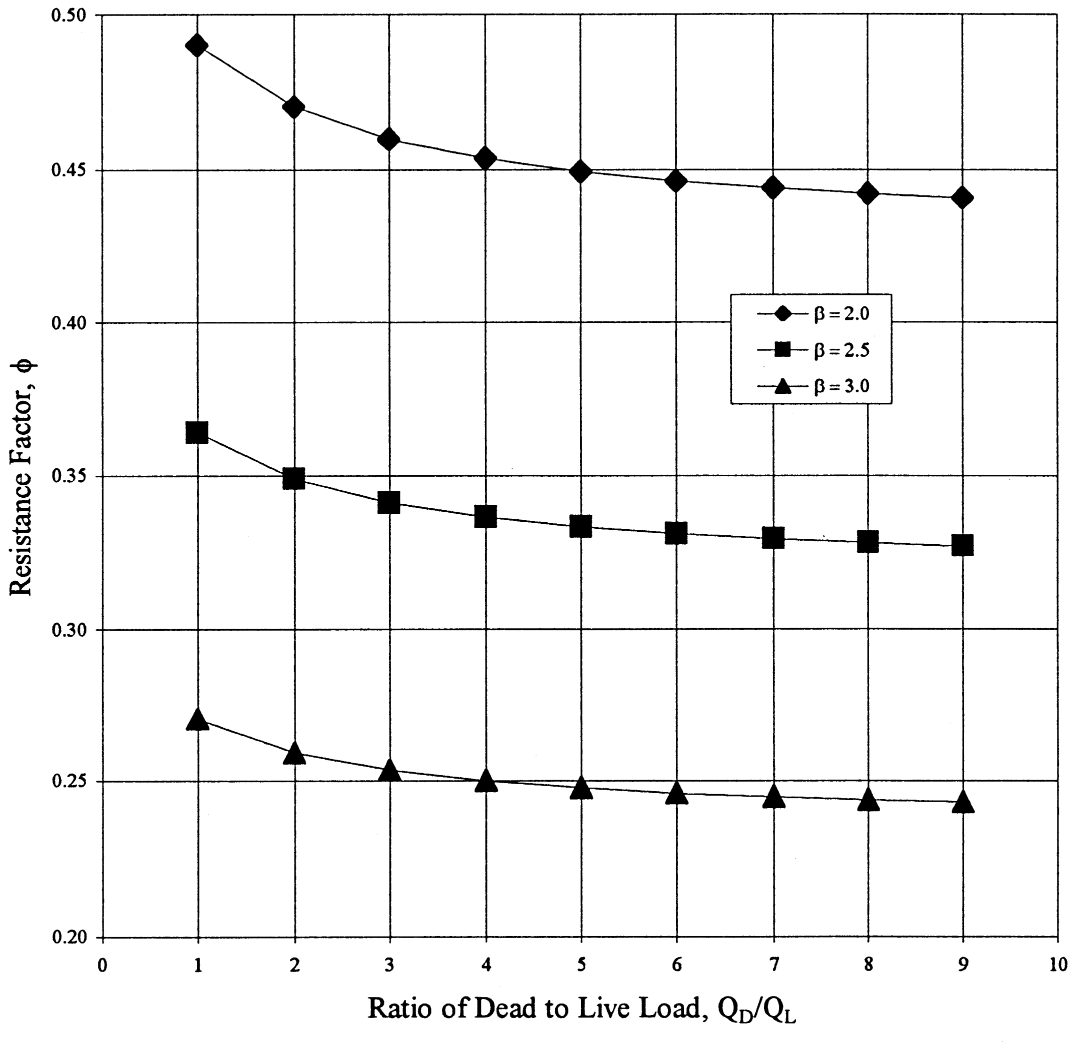
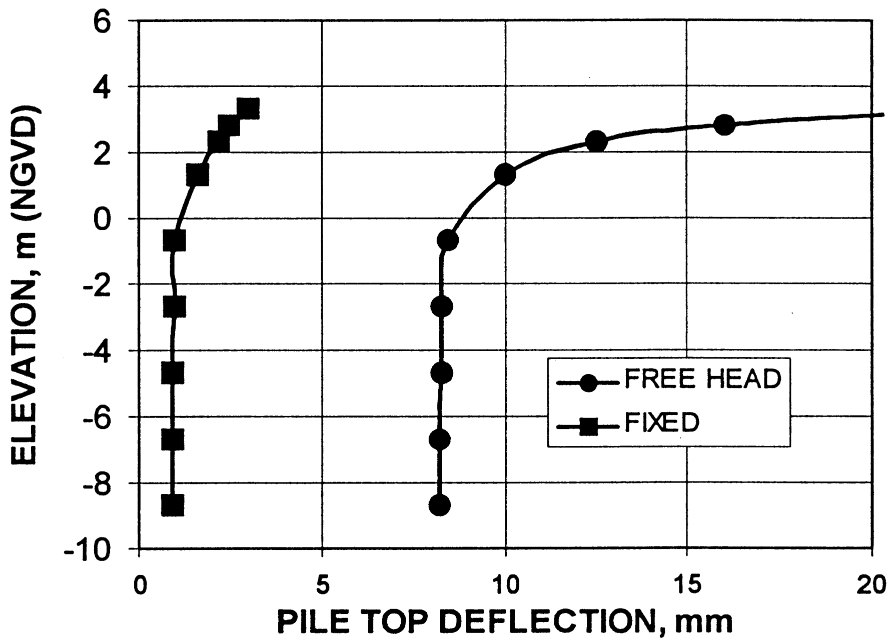


Figure 3.16 Calibrated Resistance Factor of WEAP Analysis



PILE SIZE	610 mm
UNSUPPORTED LENGTH	2.0 m
CUTOFF ELEVATION	10.3 m (NGVD)
UNFACTORED AXIAL LOAD	1400 kN
UNFACTORED LATERAL LOAD	35 kN

FIGURE 3.17 DETERMINATION OF MINIMUM TIP ELEVATION

CHAPTER 4

DRILLED SHAFT FOUNDATION DESIGN

4.1 General

In the calibration process, the load factors for dead and live loads as specified in the AASHTO LRFD Bridge Design Specification (1994) are 1.25 and 1.75, respectively. The bias factors and coefficients of variation of the load effects depend on the materials and manufacturing of the load components. Table 3.1 summarized the results of statistic study of highway structure dead and live loads by Nowak (1993). To simplify the unknown characteristics of the structure components of the database, the bias factor and coefficient of variation of the dead load for cast-in-place load components, 1.05 and 0.10, respectively, will be assumed in the calibration process. For live loads, 1.10 and 0.18 will be used for the bias factor and coefficient of variation, respectively. These conditions represent an average worse case.

4.2 Service Limits

The axial load-settlement analysis of drilled shaft can be performed by the Computer Program PL-AID (1990), while the lateral load-deflection analysis can be performed by the Computer Program FLPIER. The evaluation of foundation movements using LRFD method is performed in accordance to Service I Limit State with load factor and resistance factor equal to one (1).

The criteria of tolerable axial and lateral movement of drilled shaft usually are developed by the structural engineer in accordance with the design characteristics of the superstructures. In general, the structural stability of the entire bridge under the lateral load is the major concern, and not the absolute value of the lateral deflection of drilled shaft. Therefore, a series of lateral load analyses using FLPIER should be performed to determine the minimum penetration of deep foundation in which the entire structure will be in stable conditions.

4.3 Axial Load Capacity

Drilled shafts capacities were evaluated according to the procedures outlined in the FHWA publication "Drilled Shafts: Construction Procedures and Design Methods (1988)" and the technical publication by McVay, et al (1992). Currently, three load test methods on drilled shaft are approved by the FDOT, i.e., conventional static load test, Osterberg cell load test and Statnamic load test.

4.3.1 Bridge Foundation

A very limited drilled shaft load testing database in which the loading reached the failure load condition is available, and most of the load test results do not separate the skin friction contributed from overburden soils and rocks and the end bearings. In addition, different approaches were used to determine the total capacity of drilled shaft. A total of five (5) approaches were used to estimated the drilled shaft capacity consisting of the side friction and end bearing by using the Program SHAFTUF:

1. Soil Side Friction + Rock Side Friction + End Bearing
2. Soil Side Friction + Rock Side Friction
3. Soil Side Friction + Rock Side Friction + 1/3 End Bering
4. Rock Side Friction
5. Rock Side Friction + 1/3 End Bearing

Based on the database collected by the University of Florida and the results of statistic analysis, the bias factor and coefficient of variation for the ratio of load test result (FHWA failure criteria) to SHAFTUF prediction are presented in Table 4.1. Using the above statistic parameters and a safety factor of 2.5 for the estimated FHWA capacity, the corresponding reliability index, β of current FDOT practice ranged from 3.49 to 4.60 as presented in Table 4.1 and Figure 4.1 for a ratio of dead load to live load ranging from 1 to 9.

As shown in Table 4.1, the approaches including the side friction of the overburden soil have relatively low bias factor and coefficient of variation, which indicated a better prediction of the drilled shaft capacity. However, the side friction from overburden soils greatly affected by the construction method. Therefore, a target reliability index of 4.0 was selected for the approaches including the soil side friction, and 3.5 was selected for the approaches neglecting the soil side friction in determining the resistance factor of drilled shaft design. This is consistence with the inherent reliability index of the current ASD design methodology. Table 4.2 presents the range of the calibrated resistance factors of different design approaches for different reliability indices with a ratio of dead load to live load ranging from 1 to 9.

Using the target reliability index of 3.5 and 4.0 for drilled shaft design without and with consideration of soil friction, respectively, the resistance factors ranged from 0.51 to 0.67 for a ratio of dead load to live load ranging from 1 to 4 as shown in and Figures 4.2 to 4.6 The resistance factors calculated by ASD fitting for a safety factor of 2.5 ranged from 0.52 to 0.60 as shown in Table 2.1. In general, the ratios of dead load to live load for the majority of the highway bridge are on the order of 2 to 4. In addition, according to the ASSHTO LRFD Bridge Design Specifications, a specific load combination of Strength IV is designed for load combination relating to very high dead load to live load force effect ratio. Therefore, based on the results of the calibration by reliability analysis and ASD fitting in conjunction with the engineering judgements, the resistance factors of different design methodologies are presented in Table 4.2.

4.3.2 Miscellaneous Structure Foundation

Currently FDOT uses Drilled shaft as the foundation to support the highmast lighting pole , sign structure and signal structure. For this type of structure, because of the relatively low dead load and high wind load, lateral load and torsion resistance usually control the design of the substructures. The load test database of drilled shafts utilized in this application is not available, therefore the calibration of the resistance factor using reliability analysis is not feasible. A calibration by ASD Fitting is used to determine the resistance factor for lateral load and torsion resistance of the drilled shaft.

The Program MASTARM developed by FDOT Structure Design Office and the methodology described in FDOT Research Report D647F "Drilled Shaft Foundation for Highway Sign Structure (1974)" are used to estimate the lateral load and torsion resistance of drilled shaft. A safety factor of 1.5 and 1.0 is applied to the lateral load and torsion resistance, respectively, and a load factor of 1.4 for wind load, the resistance factor was calculated as 1.0 for both the lateral load and the torsion resistance.

4.3.3 Load Testing

Currently FDOT utilize conventional static load testing, Osterberg Cell load testing and

Statnamic load testing on the drilled shaft. However, very limited drilled shaft load tests were performed by using Osterberg cell and Statnamic load testing to date. Therefore, it is not feasible to performed separate calibrations for Osterberg Cell load testing and Statnamic load testing. It is recommended that the resistance factor for the conventional static load testing can be used for Osterberg Cell load testing and Statnamic load testing.

According the FDOT Standard Specification for Road and Bridge Construction Section B455, a safety factor of 2 shall be used for the project when performing conventional static load tests. Based on the results of reliability analysis, for a target reliability index of 2.5, the resistance factors ranged from 0.71 to 0.79 for a ratio of dead load to live load ranging from 1 to 9. The resistance factors calculated by ASD fitting for a safety factor of 2 ranged from 0.65 to 0.75. In general, the ratios of dead load to live load for the majority of the highway bridge are on the order of 2 to 4. In addition, according to the ASSHTO LRFD Bridge Design Specifications, a specific load combination of Strength IV is designed for load combination relating to very high dead load to live load force effect ratio. Therefore, based on the results of the calibration by reliability analysis and ASD fitting in conjunction with the engineering judgements, a resistance factor of 0.75 is selected for using the conventional static load test to estimate the axial compression capacity of drilled shaft.

4.3.4 Other considerations

In addition to the axial compression capacity, uplift capacity (axial tension capacity) and group effect should be considered in drilled shaft foundation design as discussed in the following sections..

4.3.4.1 Uplift capacity

Database of drilled shaft uplift capacity is not available. Therefore, the calibration using reliability theory for the uplift capacity can not be performed. Barker, et al (1991) recommended that the resistance factor for the uplift capacity of drilled shaft is 0.1 less than that for the axial compression capacity. Based on the results of reliability analysis on axial compression capacity, the resistance factor of uplift capacity ranged from 0.55 to 0.60. In ASD, the uplift capacity is usually estimated as 75% of the compression side friction. Comparing the safety factor of 2.5 used in the

Program SHAFTUF on the side friction, the equivalent safety factor of uplift capacity corresponding to the ultimate side friction is 3.33 (=2.5/0.75). Using this safety factor, the resistance factor calculated by fitting ASD using a safety factor of 3.33 ranged from 0.40 to 0.45. Therefore, based on the results of the calibration by reliability analysis and ASD fitting in conjunction with the engineering judgements, the resistance factor of 0.50 is selected for using the SHAFTUF to estimate the uplift capacity of drilled shaft.

The resistance factor of uplift capacity is selected as 0.65 for using the conventional static load test which is 0.1 less than that for the axial compression capacity of drilled shaft.

4.3.4.2 Drilled Shaft group

According to FDOT Structure Design Guideline (1996), the minimum center-to-center spacing between shafts in a group is three (3) shaft diameters, and the shaft group efficiency subjected to axial loads is designed as one (1). Therefore, the resistance factor for shaft group is the same as that for a single shaft. For the group efficiency subjected to lateral loads, it is recommended that a FLPIER analysis should be performed to evaluate the group behavior.

4.4 Lateral Load Resistance

For drilled shaft subjected to lateral loadings, it requires a relatively large deflections to mobilize passive failure of soils around the drilled shaft which generally exceed tolerable movements or structure capacity. Therefore, drilled shaft foundation must be designed to resist lateral loads without structural failure of the drilled shaft, or without excessive lateral deflection to ensure the structural stability. The ultimate soil resistance to the lateral loads usually is not critical. In general, the design of lateral load resistance of drilled shaft can be accomplished using computer programs FLPIER or COM624P for single shaft and FLPIER for shaft group, and should include:

1. Determine the maximum shaft top deflection and structure stability.

   The shaft deflection should be evaluated at the Service Limit State in which both Load Factor, γ and Resistance Factor, ϕ equal to one (1). However, the allowable deflection should be evaluated by Structural Engineer based on the structure stability and an arbitrary criteria of an allowable lateral deflection is not acceptable according to FDOT (Potter, 1995).

To ensure the structure stability, drilled shaft should be installed to a required minimum penetration below the final grade or design scour elevation. The method of determining the minimum tip elevation is the same as that for driven pile as discussed in Chapter 3.

2. Determine the maximum moment along pile.

The moment acting in the drilled shaft should be evaluated at the Strength Limit State in which Load Factor, γ should properly be selected according to the load combinations of the design conditions, while the resistance factor, ϕ equals to one (1). The estimated maximum moment should compare with the factored structure resistance to ensure that structure failure will not occur.

However, if Structural Engineer determines that the shaft top deflection or the maximum moment exceed the tolerable limit at the minimum shaft penetration, a trial-and-true analysis should be performed to determine the required minimum shaft size and/or reinforcement. This analysis can be performed using single shaft. However, it is strongly recommended that shaft group analysis should be performed for final design.

4.5 Structure Capacity

In shaft design using LRFD, the shaft materials should provide sufficient factored resistance against the external factored loads. The Resistance Factor, ϕ presented in AASHTO Specifications (1994) was calibrated using ASD fitting. Since there were no data available to perform calibration, it is recommended that the Resistance Factor for different reinforcement design shown in Table 4.3. be used.

Table 4.1

Reliability Index of SHAFTUF Analysis
Ratio of Dead Load to Live Load (Q_D/Q_L) = 1 to 9
FHWA Failure Criteria

Design Methodology	β	ASD FS	V_R	λ_R	V_{QD}	λ_D	V_{QL}	λ_L
Soil Side Friction + Rock Side Friction + Rock End Bearing	3.72-3.86	2.5	0.168	1.148	0.1	1.05	0.18	1.15
Soil Side Friction + Rock Side Friction	4.45-4.60	2.5	0.155	1.344	0.1	1.05	0.18	1.15
Rock Side Friction	3.73-3.83	2.5	0.296	1.618	0.1	1.05	0.18	1.15
Soil Side Friction + Rock Side Friction + 1/3 Rock End Bearing	4.23-4.38	2.5	0.152	1.263	0.1	1.05	0.18	1.15
Rock Side Friction + 1/3 Rock End Bearing	3.49-3.59	2.5	0.305	1.524	0.1	1.05	0.18	1.15

Table 4.2

Resistance Factor of SHAFTUF Analysis
Ratio of Dead Load to Live Load (Q_D/Q_L) = 1 to 9
FHWA Failure Criteria

Design Methodology	Resistance Factor, ϕ				
	$\beta_T = 2.5$	$\beta_T = 3.0$	$\beta_T = 3.5$	$\beta_T = 4.0$	Recommended
Soil Side Friction + Rock Side Friction + Rock End Bearing	0.73-0.82	0.64-0.72	0.56-0.63	0.49-0.55	0.50
Soil Side Friction + Rock Side Friction	0.87-0.97	0.77-0.85	0.68-0.75	0.59-0.66	0.60
Rock Side Friction	0.80-0.89	0.67-0.75	0.56-0.62	0.47-0.52	0.65
Soil Side Friction + Rock Side Friction + 1/3 Rock End Bearing	0.83-0.92	0.73-0.81	0.64-0.71	0.57-0.63	0.55
Rock Side Friction + 1/3 Rock End Bearing	0.74-0.82	0.62-0.69	0.51-0.57	0.43-0.48	0.60

Table 4.3

Resistance Factor for Structural Design of Drilled Shaft
(AASHTO, 1994)

Type of Reinforcement	Resistance Factor
Reinforced Concrete with Spiral Reinforcement	0.75
Reinforced Concrete with Tie Reinforcement	0.75

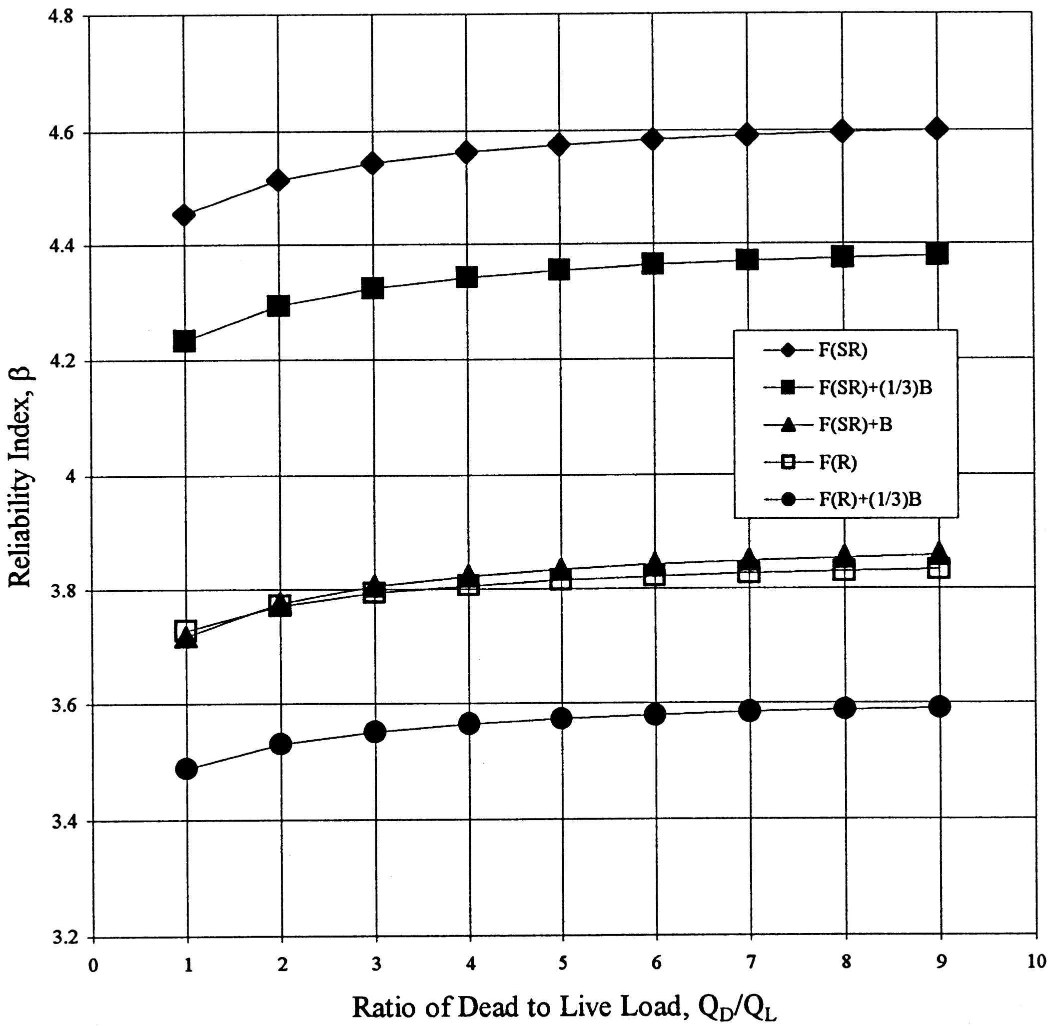


Figure 4.1 Reliability Index of SHAFTUF Prediction

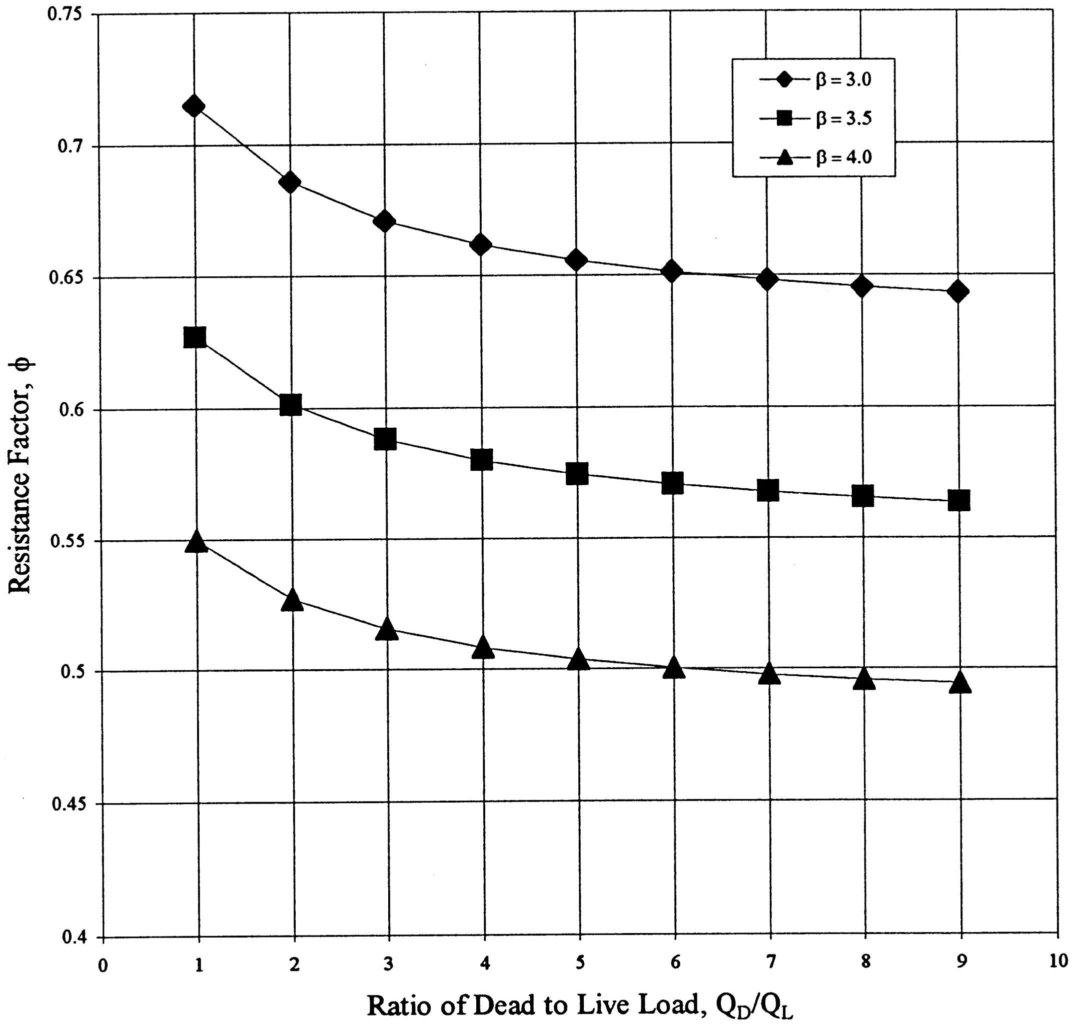


Figure 4.2 Calibrated Resistance Factor of SHAFTUF Prediction
(Soil Side Friction +Rock Side Friction + End Bearing)

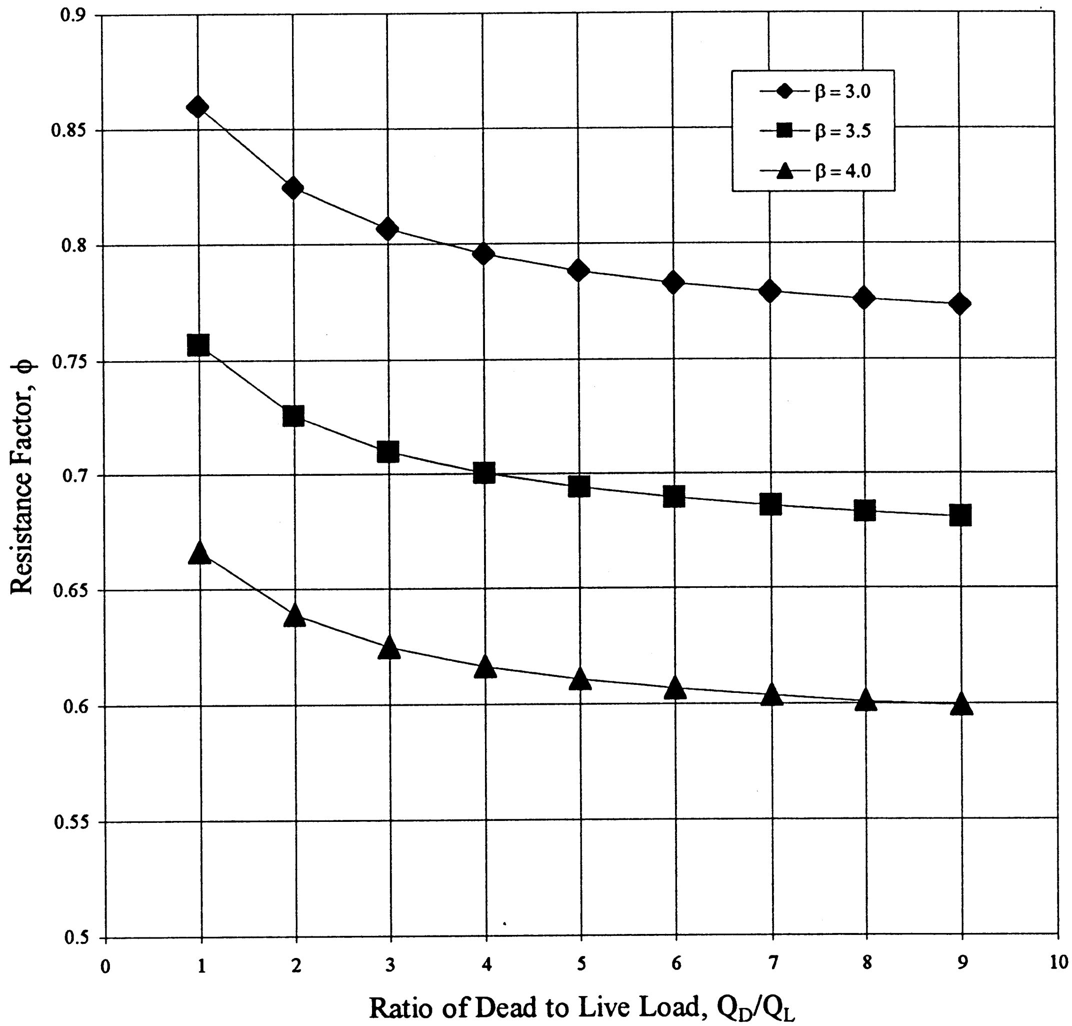


Figure 4.3 Calibrated Resistance Factor of SHAFTUF Prediction (Soil Side Friction + Rock Side Friction)

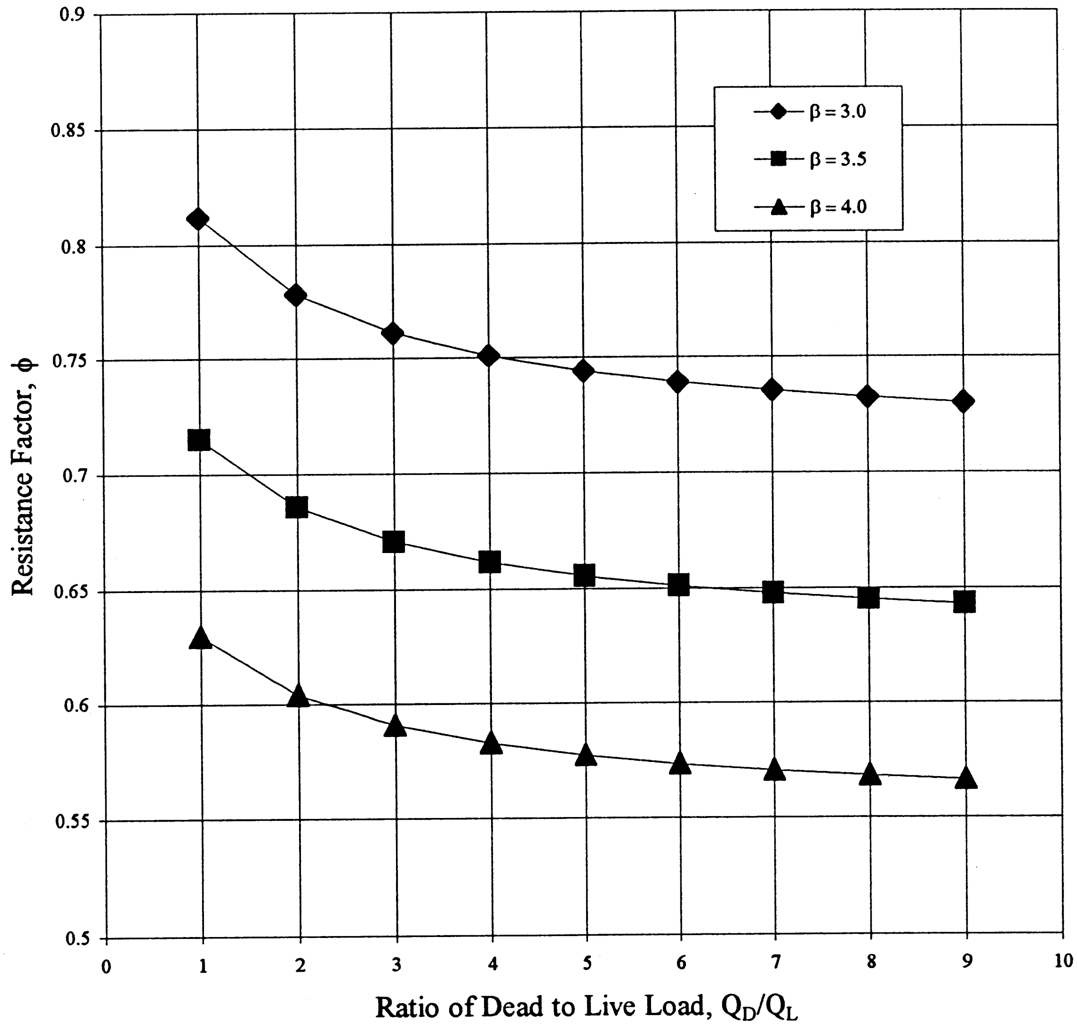


Figure 4.4 Calibrated Resistance Factor of SHAFTUF Prediction (Soil Side Friction + Rock Side Friction + 1/3 End Bearing)

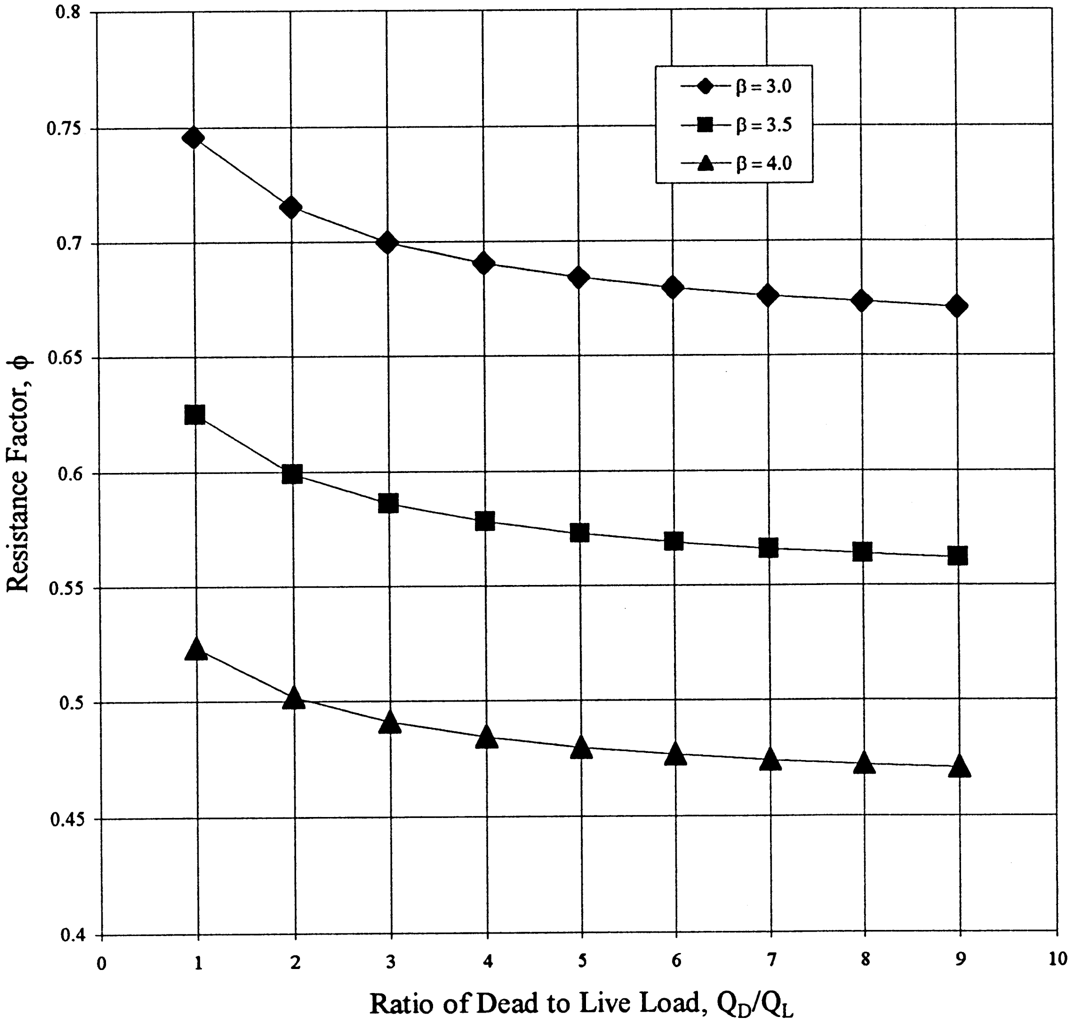


Figure 4.5 Calibrated Resistance Factor of SHAFTUF Prediction (Rock Side Friction)

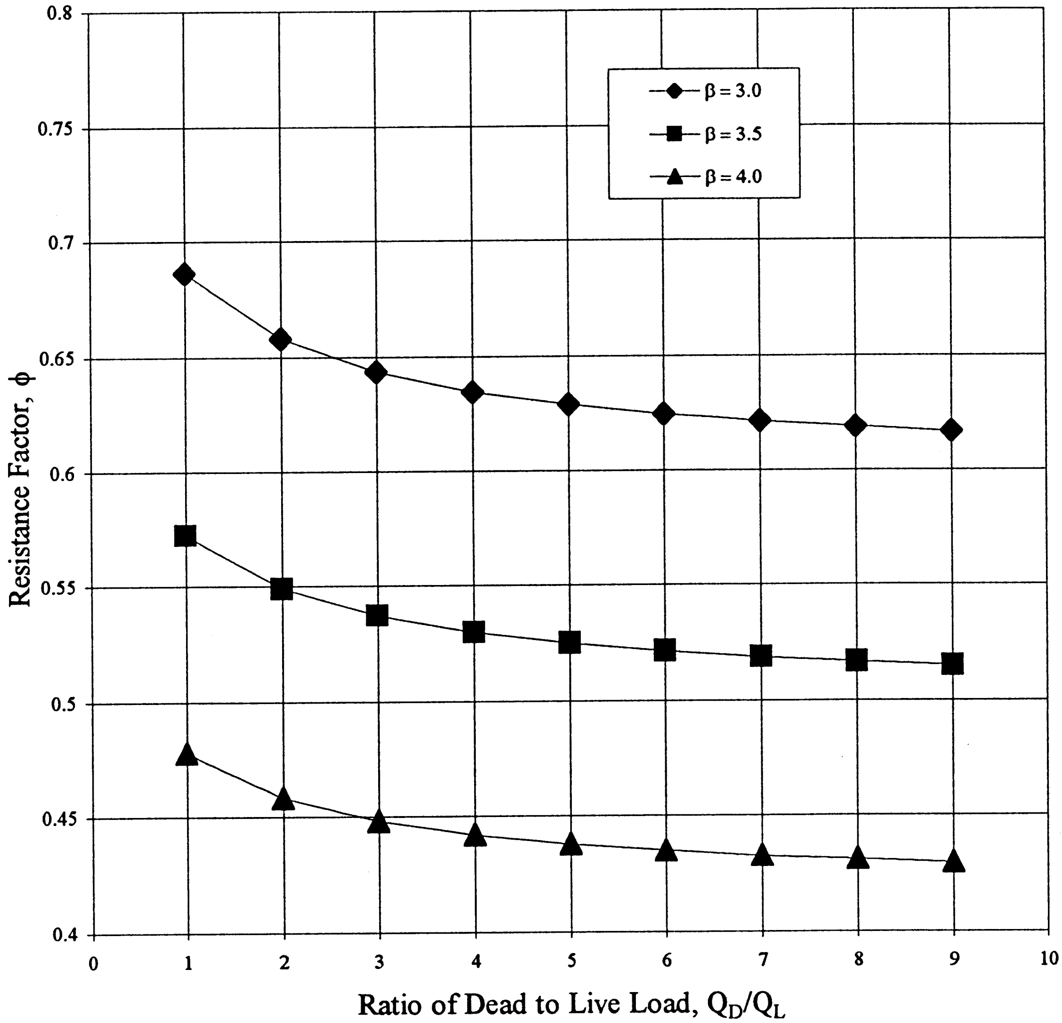


Figure 4.6 Calibrated Resistance Factor of SHAFTUF Prediction (Rock Side Friction + 1/3 End Bearing)

CHAPTER 5
SHALLOW FOUNDATION

5.1 General

There is no shallow foundation database available in Florida, resistance factors for shallow foundation design are developed based on the ASD Fitting performed by Barker, et al. (1991a, 1991b).

5.2 Service Limits

The settlement of shallow foundation can be estimated by conventional methods using semi-empirical correlations with in-situ test results. Currently, FDOT does not have preferred methods for shallow foundation design, therefore the methods for estimating settlement as described in Gifford, et al (1987), Cheney and Chassie (1993) and Barker, et al (1991a) can be used in conjunction with engineering judgement.

The criteria of tolerable settlement of shallow foundation are developed by the structural engineer in accordance with the design characteristics of the superstructures. In general practice, a limit of 25 mm has been set based on empirical assumptions without consideration of the structural performance. According to Moulton, et al (1985) who conducted a performance survey of more than 200 bridges supported on shallow foundations, the angular distortions of 0.008 or less for simple-span structures and 0.004 or less for continuous-span structures are acceptable. The maximum tolerable settlement is 50 mm for both simple-span and continuous-span bridges.

5.3 Strength Limits

The stability evaluation in the design of shallow foundation includes bearing capacity, sliding resistance, overturning (load eccentricity), and overall stability. The following ASD Fitting equation discussed in Chapter 2 was used to determine the resistance factors for shallow foundation design.

$$\Phi \geq \frac{\gamma_D Q_D + \gamma_L Q_L}{FS(Q_D + Q_L)} \tag{2.3}$$

where ϕ is resistance factor, FS is safety factor of ASD, Q_D and Q_L are dead and live load, respectively, Υ_D and Υ_L are the load factors of dead load and live load, respectively.

Using safety factors of 3.0, 1.5, 2.0 and 1.5 for the bearing capacity, sliding resistance, overturning and overall stability of ASD shallow foundation design, the resistance factors were calibrated as 0.45, 0.90, 0.65 and 0.85, respectively. For overturning stability, the location of the factored bearing pressure resultant must be within 1/4 and 3/8 B (footing width) of the center of the footing for footing on soil and rock, respectively. Table 5.1 summarizes the resistance factor for shallow foundation design.

In addition, the resistance factor , 0.45, of bearing capacity may result a larger footing size compared to that of ASD design if the footing was subjected to horizontal loads. This is because the reduced effective area of footing due to load eccentricity in LRFD design is smaller than that in ASD design. The factored bearing pressure is not only increased by load factor, but also by the reduced effective area. Therefore, during design if shallow foundation is subjected to horizontal loads, the resistance factor of bearing capacity should be increased to 0.60.

TABLE 5.1

CALIBRATED RESISTANCE FACTORS FOR SHALLOW FOUNDATION DESIGN

DESIGN CONSIDERATION	DESIGN METHODOLOGY	ASD FS	RESISTANCE FACTOR, ϕ
BEARING CAPACITY	RATIONAL METHOD USING FRICTION ANGLE AND COHESION ESTIMATED FROM FIELD TESTS	3.0	0.45 0.60*
SLIDING		1.5	0.90
OVERTURNING		2.0	0.65 e<B/4 (SOIL) e<3B/8 (ROCK)
OVERALL STABILITY		1.5	0.85

* If significant horizontal loads are expected.

CHAPTER 6
RETAINING WALL SYSTEMS

6.1 General

Because no retaining wall database was available in Florida, the resistance factors for the retaining wall design were developed based on the ASD Fitting performed by Barker, et al. (1991a, 1991b). Currently, there are three (3) major types of wall systems utilized in Florida, i.e., rigid retaining walls including precast and cast-in-place walls, flexible sheet pile walls including cantilever and anchored walls, and Mechanical Stabilized Earthen wall (MSE wall).

6.2 Rigid Retaining Wall

6.2.1 Service Limits

The methods of estimating settlement and the criteria of tolerable settlement for rigid retaining walls generally follow the guidelines for the shallow foundations.

6.2.2 Strength Limits

The stability evaluation in the design of rigid retaining wall include bearing capacity, sliding resistance, overturning (load eccentricity), and overall stability. The resistance factors were determined by ASD Fitting as discussed in Chapter 2. Using load factors of 1.35 and 1.5 for vertical earth pressure and lateral active earth pressure, respectively, the resistance factors of shallow foundation design for the sliding resistance, overturning and overall stability are 1.0, 0.75 and 0.85 corresponding the safety factor 1.5, 2.0 and 1.5, respectively, in ASD method. The resistance factor of bearing capacity was determined as 0.60 corresponding to the safety factor of 3.0 due to significant horizontal earth pressures acted on the retaining wall.

6.3 Flexible Sheet Pile Wall

6.3.1 Service Limits

In general, the vertical settlement of the sheet pile wall is not a concern due to very light vertical loads. The tolerable horizontal deflection of the sheet pile wall is set to make sure that the wall movement will not affect the performance of other structures or facilities near the wall.

6.3.2 Strength Limits

The resistance factors were determined by ASD Fitting as discussed in Chapter 2. Using load factors of 1.35 and 1.5 for vertical earth pressure and lateral active earth pressure, respectively, the resistance factors of sheet pile wall design for the overturning (passive resistance) and overall stability are 1.0 and 0.85 corresponding the safety factor 1.5 in ASD method.

The pullout resistance of anchor for the anchor wall design also was calibrated by ASD Fitting and with the reference of ASSHTO Specification (1994). The resistance factors of soil pullout resistance are 0.65 without pullout load test and 0.7 with pull out load test. The resistance factors of rock pullout resistance are 0.55 to 0.75 depending on the design method and 0.8 with pullout load test.

6.4 Mechanical Stabilized Earth Wall

6.4.1 Service Limits

Due to the special design of the slip joint and wall panel, MSE wall can tolerate larger settlement and distortion than that of rigid retaining wall. According Reinforcing Technology (1997), MSE wall can tolerate a settlement up to 300 mm and 1% of vertical distortion with appropriate design. The tolerable horizontal displacement is on the order of 6.2 mm per meter of wall height for precast wall facing and 16.7 mm per meter of wall height for flexible wire facing as recommended by FHWA (1990).

6.4.2 Strength Limits

The resistance factors were determined by ASD Fitting using equation 2.3 discussed in Chapter 2. Using load factors of 1.35 and 1.5 for vertical earth pressure and lateral active earth pressure, respectively, the resistance factors for the bearing capacity, sliding resistance, overturning and overall stability are 0.70, 1.0, 0.75 and 0.85 corresponding safety factors of 2.5, 1.5, 2.0 and 1.5, respectively, in ASD method. For overturning stability, the location of the factored bearing pressure resultant must be within 1/4 and 3/8 B (footing width) of the center of the footing for footing on soil and rock, respectively, for both rigid retaining wall and MSE wall systems.

6.5 Summary

Table 6.1 summarizes the resistance factor for wall system design. However, for the overall stability, the current available computer program are not readily in LRFD method. The computer program used for sheet pile wall design also is not readily available in LRFD method.

Table 6.1

Calibrated Resistance Factors for
Wall System Design

Wall System	Design Consideration	Design Methodology	FS (ASD)	Resistance Factor, ϕ
Rigid Retaining Wall	Bearing Capacity	Rational Method Using Friction Angle and Cohesion Estimated from Field Tests	3.0	0.60
	Sliding		1.5	1.0
	Overturning		2.0	0.75 $e<B/4$ (Soil) $e<3B/8$ (Rock)
	Overall Stability		1.5	0.85
MSE Wall	Bearing Capacity	Rational Method (FHWA-RD-89-043) Using Friction Angle and Cohesion Estimated from Field Tests	2.5	0.70
	Sliding		1.5	1.00
	Overturning		2.0	0.75 $e<B/4$ (Soil) $e<3B/8$ (Rock)
	Overall Stability		1.5	0.85
Flexible Sheet Pile Wall	Overturning	Rational Method (Passive Resistance)	1.5	1.0
	Overall Stability	Rational Method	1.5	0.85
	Pullout Resistance of Anchor	Rational Method/All Soils	2.5	0.65
		Soil Pullout Load test	2.5	0.70
		Rational Method/Rocks		
		Correction w/Rock type only	3.0	0.55
		Lab Compression Test	3.0	0.60
		Lab Rock-Grout Bond Test	3.0	0.75
		Rock Pullout Load Test	3.0	0.80

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

The calibration of resistance factors for Florida foundation performed in this research consisted of:

1. Deep foundations including driven pile and drilled shaft.
2. Shallow foundation design.
3. Retaining wall system design.

The deep foundation database collected to date for the Florida area indicated that the sample size of driven piles is sufficient to perform reliability analysis. Although, the sample size of drilled shaft is relatively small for reliability analysis, the calibrated resistance factors are reasonable comparing to the results from ASD Fitting and the recommendations in AASHOT LFRD Specifications (1994)

The target reliability index used in the NCHRP24-4 (1991) research project for the driven pile was on the order of 2.0 to 2.5 which is significantly lower than the values of 3.5 to 4.5 used in the NBCC code. Table 7.1 shows the resistance factors calibrated for the Florida soils compared to that specified in the AASHTO LRFD Specifications. The target reliability index of 2.5 used in this research is based on the reliability index inherent in the current ASD method and the FDOT current practice.

The calibrations for shallow foundation and retaining wall system were performed by ASD Fitting. The resistance factors recommended in ASSHTO Specifications also were used as reference to determine the resistance factors.

Table 7.2 summarized the resistance factors calibrated through this research project. Two FDOT projects were redesign using LRFD method and the resistance factors presented in Table 7.2. The designs are presented in Appendix A including the comparisons with ASD method. The comparisons indicated that the LRFD method using the recommended resistance factors resulted compatible designs with that of ASD method.

There are several design concerns as discussed in the following were not addressed in this research because of insufficient information or not included in the scope of services. It is recommended that a continuous research on these design concern should be performed.

1. Design criteria of service limits. Service limits design is one of the major role in LRFD methodology. However, the AASHTO LRFD Bridge Design Specifications only provide a very limit information regarding the service limits and without design guideline. Therefore, it is necessary to conduct a joint research of geotechnical engineer and structural engineer to provide a design guideline for the service limits.
2. Drilled shaft database and design methodology including the selection of design parameters. Currently, FDOT has not expressed any preferred methodology for drilled shaft design. In addition, due to the difficulty of obtaining a good quality of rock core in the soft limerock layer, the selection of the design parameters became a major problem in drilled shaft design. A methodology to determine the design parameters by in-situ testing in conjunction a standard design method will be the key to provide an economic and safe design for drilled shaft foundation.
3. Resistance factor for lateral load analysis. Currently the resistance factors used in the lateral load analysis are equal to one for both strength limits and service limits designs. Therefore, the relation between the induced maximum moment and the structure capacity is not as clear as that in the axial load analysis. Therefore, a detail reliability analysis should be performed to determine the suitable resistance factor for lateral load analysis.
4. LRFD based computer programs. The majority of the geotechnical computer programs are coded in ASD method. Some critical programs, such as slope stability analysis and retaining wall design should be modified to LRFD method to ensure a smooth and successful transition from ASD to LRFD methodology.

Table 7.1

Comparison of Resistance Factors for Geotechnical Strength Limit State in Axial Loaded Piles and Drilled Shafts

Method/Soil/Condition		Resistance Factor			
		AASHTO (1994)	FDOT RESEARCH		NBCC (1995)
			Reliability	Fitting	
Ultimate Bearing Resistance of Single Piles	Skin Friction and End Bearing: Sand SPT-method	0.45	0.65	0.70	0.40
	Skin Friction and End Bearing: All Soils Load Test Pile Driving Analyzer	0.80 0.70	0.75 0.65	0.70 0.55	0.60 0.50
Drilled Shaft	All Soils	0.45-0.65	0.50-0.65	0.55	-

Table 7.2

Summary of Calibrated Resistance Factors
for Foundation Design

Foundation	Application	Design Consideration	Design Methodology	ASD Fitting			Target Reliability Index, β_T	Resistance Factor, ϕ
				FS	β_{ASD}	ϕ_{ASD}		
Drilled Shaft	Bridge Foundation	Compression	FHWA-HI-88-042 on soils with N<15 correction suggested by O'Nell. For rock socket, using McVay's method to determine unit skin friction.					
			1. F(S) + F(R) + B	2.5	3.72-3.86	0.55	4.0	0.50
			2. F(S) + F (R)	2.5	4.45-4.60	0.55	4.0	0.60
			3. F(R)	2.5	3.73-3.83	0.55	3.5	0.65
			4. F(S) + F(R) + 1/3 B	2.5	4.23-4.38	0.55	4.0	0.55
			5. F(R) + 1/3 B	2.5	3.49-3.59	0.55	3.5	0.60
			Load testing	2.0	-	0.70	2.5	0.75
		Uplift	Same as above for side friction	2.5	-	0.55	-	0.50
			Load testing	2.0	-	0.70	-	0.65
		Lateral Load	Structure Stability Consideration	-	-	-	-	1.0
	Misc. Structure	Compression	Same as above	2.5	-	-	-	0.55
		Uplift	Same as above	2.5	-	-	-	0.50
		Lateral Load	Brom's Method (FDOT D647F205)	1.5	-	-	-	1.0
		Torsion	FDOT Structure Office	1.0	-	-	-	1.0

Table 7.2 (Continued)

Summary of Calibrated Resistance Factors
for Foundation Design

Foundation	Application	Design Consideration	Design Methodology	ASD Fitting			Target Reliability Index, β_T	Resistance Factor, ϕ
				FS	β_{ASD}	ϕ_{ASD}		
Shallow Foundation	Bridge Foundation	Bearing Capacity	Rational Method	3.0	-	0.45	-	0.45
		Sliding		1.5	-	0.90	-	0.90
		Overturning		2.0	-	0.65	-	0.65 e<B/4 (soil) e<3B/8 (rock)
		Overall Stability		1.5	-	0.85	-	0.85
Driven Pile	Bridge Foundation	Compression	Modified RB-121 (SPT-94)	2.0	2.45-2.57	0.70	2.5	0.65
			PDA (EOD)	2.5	3.04-3.14	0.55	2.5	0.65
			PDA (BOR)	2.5	2.40-2.50	0.55	2.5	0.55
			Wave Equation Analysis	3.0	2.28-3.11	0.45	2.5	0.35
			Static Load Testing	2.0	-	0.70	2.5	0.75
		Uplift	Modified RB-121 (SPT-94)	2.0	-	0.70	-	0.55
			Static Load Testing	2.0	-	0.70	-	0.65
		Lateral Load	Structure Stability Consideration	-	-	-	-	1.0

Table 7.2 (Continued)

Summary of Calibrated Resistance Factors
for Foundation Design

Foundation	Application	Design Consideration	Design Methodology	ASD Fitting			Target Reliability Index, β_T	Resistance Factor, ϕ
				FS	β_{ASD}	ϕ_{ASD}		
Rigid Retaining Wall	Earth Structure	Bearing Capacity	Rational Method	3.0	-	0.60	-	0.60
		Sliding		1.5	-	1.0	-	1.0
		Overturning		2.0	-	0.75	-	0.75 e<B/4 (soil) e<3B/8 (rock)
		Overall Stability		1.5	-	0.85	-	085
MSE Wall	Earth Structure	Bearing Capacity	FHWA -RD-89-043 (Rational Method)	2.5	-	0.70	-	0.70
		Sliding		1.5	-	1.0	-	1.0
		Overturning		2.0	-	0.75	-	0.75 e<B/4 (soil) e<3B/8 (rock)
		Overall Stability	Rational Method	1.5	-	0.85	-	0.85
Flexible Sheet Pile Wall	Earth Structure	Overturning	Rational Method (Passive Resistance)	1.5	-	1.0	-	1.0
		Overall Stability	Rational Method	1.5	-	0.85	-	0.85
		Pullout Resistance of Anchor	Rational Method/All Soils	2.5	-	0.65	-	0.65
			Soil Pullout Load Test	2.5	-	0.70	-	0.70
			Rational Method/Rock::					
			Correction w/Rock Type only	3.0	-	0.55	-	0.55
			Lab Compression Test	3.0	-	0.60	-	0.60
			Lab Rock-Grout Bond Test	3.0	-	0.75	-	0.75
			Rock Pullout Load Test	3.0	-	0.80	-	0.80

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APPENDIX A
DESIGN AND COMPARISON OF LRFD AND ASD METHODS

Two FDOT projects were used as design examples to compare the results of ASD and LRFD using the recommended resistance factors. The general descriptions and design conditions of the example projects were summarized as follows. Because the original Allowable Stress Design (ASD) of these projects were in English units, the Load and Resistance Factor Design (LRFD) will also be performed in English.

A.1 First Design Example

The first example project was located on SR 60 in Bartow, Florida. The proposed improvements included a widening of the existing bridge structure and new construction of Mechanically Stabilized Earth (MSE) Wall. The structure was approximately 188 feet in length with two end bents and four intermediate bents. The foundation type selected for the bridge widening was 18-inch square prestress concrete pile.

A.1.1 Driven Pile Design

According to Structural Engineer, the estimated live loads and dead loads acting on each pile at different bent locations were shown on Table A.1. The load factors of live load and dead load were 1.75 and 1.25, respectively. The resistance factor of driven pile design using computer program SPT-94 was 0.65, while the resistance factor of the Required Driving Resistance (RDR) with PDA monitoring were 0.65.

The results of SPT-94 analysis performed by the original Geotechnical Designer (FDOT, 1997) were used in LRFD design. The anticipated tip elevations were determined at the elevation where the Davisson capacity from SPT-94 output was equal to the factored load divided by the resistance factor of 0.65. Since the resistance factor of PDA monitoring was the same as that of SPT-94 analysis, RDR should be the same as that of factored load divided by the resistance of 0.65. The results of LRFD design were shown in Table A.1 in conjunction with the results of ASD design. As shown in Table A.1, the difference of anticipate tip elevation and RDR for ASD and LRFD were within 5%. For ASD design, a safety factor of 2.0 was used in SPT-94 analysis and 2.5 for RDR with PDA monitoring.

A.1.2 Mechanically Stabilized Earth Wall

A Lotus 1-2-3 spread sheet developed by FDOT District 5 Geotechnical office to performed external stability analysis of MSE wall. This spread sheet was originally coded in ASD and was modified to LRFD using the recommended resistance factors as follows:

	ASD/F.S.	LRFD/ϕ
Overturning Stability	2.0	0.75
Sliding Stability	1.5	1.0
Bearing Capacity	2.5	0.7

The design parameters used in the MSE wall No. 4C design were as follows:

Slope of soil surface = 26.6 degrees

type of reinforcement = Inextensible

Backfill unit weight = 105 pcf; Angle of internal friction = 30 degrees

Reinforced soil mass unit weight = 105 pcf; Angle of internal friction = 30 degrees

Foundation soil unit weight = 100 pcf; Angle of internal friction = 29 degrees

The results of LRFD external stability analysis were shown in Table A.2 in conjunction with the ASD results. As can be seen in the table, the required lengths at different wall heights were at approximately the same for both design methods. It should be noted that currently no computer program is available to perform LRFD overall stability analysis, therefore, this analysis did not perform.

A.2 Second Design Example

The second example project was located on SR 688 in Pinellas County, Florida. The project included a reconstruction of the existing bridge structure, new construction of Mechanically Stabilized Earth (MSE) Wall and mast arm pole foundations. The structure is a bascule bridge over Intracoastal Waterway with two abutment end bents and two intermediate piers. The foundation type selected was 36-inch drilled shaft for the end bents, and 60-inch drilled shaft for intermediate pier. The foundation type of mast arm pole was 48-inch drilled shaft.

A.2.1 Drilled Shaft Design

According to Structural Engineer, the estimated live loads and dead loads acting on each drilled shaft at different bent locations were shown on Table A.3. The load factors of live load and dead load were 1.75 and 1.25, respectively. The resistance factor of drilled shaft design using computer program SHAFT 98 was 0.55 which considered the skin friction contributed by soil and rock layers and 1/3 of ultimate end bearing.

The results of SHAFTUF analysis performed by the original Geotechnical Designer (PSI, 1996) were used in LRFD design. The anticipated tip elevations were determined at the elevation where the Ultimate capacity from SHAFT98 output was equal to the factored load divided by the resistance factor of 0.55. The results of LRFD design were shown in Table A.3 in conjunction with the results of ASD design. As shown in Table A.3, the difference of anticipate tip elevation for ASD and LRFD were within 5%. For ASD design, a safety factor of 2.5 was used in design. It should be noted that since the LFRD lateral load analysis used service load and resistance factor of 1.0 which will result the same design as ASD, therefore, this analysis did not perform in this exercise.

A.2.2 Mechanically Stabilized Earth Wall

A Lotus 1-2-3 spread sheet developed by FDOT District 5 Geotechnical office to performed external stability analysis of MSE wall. This spread sheet was originally coded in ASD and was modified to LRFD using the recommended resistance factors as follows:

	ASD/F.S.	LRFD/ϕ
Overturning Stability	2.0	0.75
Sliding Stability	1.5	1.0
Bearing Capacity	2.5	0.7

The design parameters used in the MSE wall design were as follows:

Slope of soil surface = 26.6 degrees

type of reinforcement = Extensible (Geogride)

Backfill unit weight = 105 pcf; Angle of internal friction = 30 degrees

Reinforced soil mass unit weight = 105 pcf; Angle of internal friction = 30 degrees

Foundation soil unit weight = 52.6 pcf; Angle of internal friction = 30 degrees

The results of LRFD external stability analysis were shown in Table A.4 in conjunction with the ASD results. As can be seen in the table, the required lengths at different wall heights were at approximately the same for both design methods. It should be noted that currently no computer program is available to perform LRFD overall stability analysis, therefore, this analysis did not perform.

A.2.3. Mast Arm Pole Foundation Design

For the mast arm pole foundation design, the required drilled shaft length was founded to be the greatest length to provide required capacities for axial, lateral and torsion loads. The program SHAFT98 was used for axial load design with a resistance factor of 0.55. For torsion design, the skin friction calculated from SHAFT98 was used to estimate the torsion resistance with a resistance factor of 1.0. For lateral analysis, the recommended design procedure from the FDOT report D647F, entitled 'Drilled Shaft Foundation for Highway Sign Structures' was used with a resistance factor of 1.0. The majority of the axial load was from dead load, therefore, a load factor of 1.25 was selected, while the majority of later load and torsion were from wind load, a resistance factor of 1.40 was selected.

The results of LRFD design in conjunction with that of ASD were shown in Table A.5. As can be seen in the table, the required drilled shaft lengths determined from ASD and LRFD were in excellent agreement.

Table A.1

Comparison of Driven Pile Designs by ASD and LRFD Methods

Bent No.	Pile Size (in.)	Live Load (tons)	Dead Load (tons)	ASD			LRFD		
				Design Load (tons)	Anticipated Tip. EL. (ft, NGVD)	RDR (tons)	Factored Load (tons)	Anticipated Tip EL. (ft, NGVD)	RDR (tons)
1	18	33	19	52	73	130	82	72	126
2	18	38	22	60	70	150	94	70	145
3	18	52	29	81	68	203	128	65	197
4	18	52	29	81	68	203	128	65	197
5	18	38	22	60	70	150	94	70	145
6	18	33	19	52	73	130	82	72	126

Table A.2

Comparison of MSE wall Design by ASD and LRFD Methods

H (ft)	E/I	ASD					LRFD				
		L (ft)	Overturning F.S.	Sliding F.S.	Bearing Capacity F.S.	Allowable Bearing Capacity (psf)	L (ft)	Overturning Ratio	Sliding Ratio	Bearing Capacity Ratio	Ultimate Bearing Capacity (psf)
5	I	8	4.29	1.70	4.41	1996	8	2.86	1.13	2.67	4508
7	I	8	3.29	1.53	2.90	1827	8	2.19	1.02	1.66	4016
9	I	10	3.33	1.54	2.96	2295	10	2.22	1.03	1.70	5052
11	I	12	3.36	1.54	3.00	2762	12	2.24	1.03	1.73	6089
13	I	13	3.12	1.50	2.66	2907	13	2.08	1.00	1.50	6346
15	I	15	3.17	1.51	2.72	3375	15	2.11	1.01	1.55	7383
17	I	17	3.20	1.52	2.78	3843	17	2.14	1.01	1.58	8420
19	I	19	3.23	1.52	2.82	4311	19	2.16	1.01	1.61	9457
21	I	21	3.26	1.53	2.86	4779	21	2.17	1.02	1.63	10493
23	I	22	3.13	1.50	2.66	4922	22	2.09	1.00	1.51	10749
25	I	24	3.16	1.51	2.71	5391	24	2.10	1.00	1.53	11786
27	I	26	3.18	1.51	2.74	5859	26	2.12	1.01	1.56	12823
29	I	28	3.20	1.52	2.77	6327	28	2.13	1.01	1.58	13860
31	I	30	3.22	1.52	2.80	6795	30	2.15	1.01	1.60	14897

Table A.3

Comparison of Drilled Shaft Designs by ASD and LRFD Methods

No.	Shaft Size (in.)	Live Load (tons)	Dead Load (tons)	ASD		LRFD	
				Design Load (tons)	Anticipated Tip. EL. (ft, NGVD)	Factored Load (tons)	Anticipated Tip EL. (ft, NGVD)
Bent 1	36	27	168	195	-48	257	-47
Pier 1	60	53	697	750	-66	964	-64
Pier 2	60	53	697	750	-70	964	-68
Bent 2	36	27	168	196	-55	257	-54

Table A.4

Comparison of MSE wall Design by ASD and LRFD Methods

H (ft)	E/I	ASD					LRFD				
		L (ft)	Overturning F.S.	Sliding F.S.	Bearing Capacity F.S.	Allowable Bearing Capacity (psf)	L (ft)	Overturning Ratio	Sliding Ratio	Bearing Capacity Ratio	Ultimate Bearing Capacity (psf)
3.5	E	8	4.2	1.74	2.97	1207	8	2.35	1.32	1.19	2686
5.5	E	10	3.97	1.71	2.68	1487	10	2.17	1.26	1.07	3296
7.5	E	12	3.84	1.69	2.52	1770	12	2.07	1.23	1.01	3917
9.5	E	15	4.04	1.72	2.70	2261	15	2.15	1.23	1.10	5040
11.5	E	17	3.95	1.71	2.59	2547	17	2.08	1.21	1.06	5670
13.5	E	19	3.89	1.70	2.51	2833	19	2.04	1.20	1.02	6302
15.5	E	22	4.02	1.72	2.64	3326	22	2.10	1.21	1.08	7429
17.5	E	24	3.97	1.71	2.57	3613	24	2.06	1.20	1.05	8063
19.5	E	26	3.92	1.70	2.51	3900	26	2.03	1.19	1.03	8699
21.5	E	29	4.02	1.72	2.61	4394	29	2.08	1.19	1.08	9826

Table A.5

Comparison of Mast Arm Pole Foundation Design by ASD and LRFD Methods

Pole No.	ASD				LRFD			
	Axial Load (kips)	Lateral Load (kips)	Torsion (kip-ft)	Required Shaft Length (ft)	Factored Axial Load (kips)	Factored Lateral Load (kips)	Factored Torsion (ft)	Required Shaft Length (ft)
M-1	4.8	5.6	90	15	6.0	7.8	126	14
M-2	5.8	5.5	105	16	7.2	7.7	146	16
M-3	5.6	5.5	117	23	7.0	7.7	163	22
M-4	4.0	5.3	107	15	5.0	7.4	150	15
M-5	3.6	4.8	92	15	4.5	6.7	129	15
M-6	2.2	5.7	140	18	5.5	8.0	196	19
M-7	3.6	4.8	93	14	4.5	6.7	130	14