NCHRP

NATIONAL
COOPERATIVE
HIGHWAY
RESEARCH
PROGRAM

REPORT 507

Load and Resistance Factor Design (LRFD) for Deep Foundations

TRANSPORTATION RESEARCH BOARD
OF THE NATIONAL ACADEMIES

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

NCHRP REPORT 507

Load and Resistance Factor Design (LRFD) for Deep Foundations

SAMUEL G. PAIKOWSKY
Geotechnical Engineering Research Laboratory
Department of Civil & Environmental Engineering
University of Massachusetts
Lowell, MA

WITH CONTRIBUTIONS BY:
BJORN BIRGISSON, MICHAEL MCVAY, THAI NGUYEN
University of Florida
Gainesville, FL

CHING KUO
Geostructures, Inc.
Tampa, FL

GREGORY BAECHER
BILAL AYYUB
University of Maryland
College Park, MD

KIRK STENERSEN
KEVIN O'MALLEY
University of Massachusetts
Lowell, MA

LES CHERNAUSKAS
Geosciences Testing and Research, Inc.
N. Chelmsford, MA

MICHAEL O'NEILL
University of Houston
Houston, TX

SUBJECT AREAS
Bridges, Other Structures, and Hydraulics and Hydrology • Soils, Geology and Foundations

Research Sponsored by the American Association of State Highway and Transportation Officials
in Cooperation with the Federal Highway Administration

TRANSPORTATION RESEARCH BOARD

WASHINGTON, D.C.
2004
www.TRB.org

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

Systematic, well-designed research provides the most effective approach to the solution of many problems facing highway administrators and engineers. Often, highway problems are of local interest and can best be studied by highway departments individually or in cooperation with their state universities and others. However, the accelerating growth of highway transportation develops increasingly complex problems of wide interest to highway authorities. These problems are best studied through a coordinated program of cooperative research.

In recognition of these needs, the highway administrators of the American Association of State Highway and Transportation Officials initiated in 1962 an objective national highway research program employing modern scientific techniques. This program is supported on a continuing basis by funds from participating member states of the Association and it receives the full cooperation and support of the Federal Highway Administration, United States Department of Transportation.

The Transportation Research Board of the National Academies was requested by the Association to administer the research program because of the Board's recognized objectivity and understanding of modern research practices. The Board is uniquely suited for this purpose as it maintains an extensive committee structure from which authorities on any highway transportation subject may be drawn; it possesses avenues of communications and cooperation with federal, state and local governmental agencies, universities, and industry; its relationship to the National Research Council is an insurance of objectivity; it maintains a full-time research correlation staff of specialists in highway transportation matters to bring the findings of research directly to those who are in a position to use them.

The program is developed on the basis of research needs identified by chief administrators of the highway and transportation departments and by committees of AASHTO. Each year, specific areas of research needs to be included in the program are proposed to the National Research Council and the Board by the American Association of State Highway and Transportation Officials. Research projects to fulfill these needs are defined by the Board, and qualified research agencies are selected from those that have submitted proposals. Administration and surveillance of research contracts are the responsibilities of the National Research Council and the Transportation Research Board.

The needs for highway research are many, and the National Cooperative Highway Research Program can make significant contributions to the solution of highway transportation problems of mutual concern to many responsible groups. The program, however, is intended to complement rather than to substitute for or duplicate other highway research programs.

Note: The Transportation Research Board of the National Academies, the National Research Council, the Federal Highway Administration, the American Association of State Highway and Transportation Officials, and the individual states participating in the National Cooperative Highway Research Program do not endorse products or manufacturers. Trade or manufacturers' names appear herein solely because they are considered essential to the object of this report.

NCHRP REPORT 507

Project 24-17 FY'99

ISSN 0077-5614

ISBN 0-309-08796-1

Library of Congress Control Number 2004107693

Price $31.00

NOTICE

The project that is the subject of this report was a part of the National Cooperative Highway Research Program conducted by the Transportation Research Board with the approval of the Governing Board of the National Research Council. Such approval reflects the Governing Board's judgment that the program concerned is of national importance and appropriate with respect to both the purposes and resources of the National Research Council.

The members of the technical committee selected to monitor this project and to review this report were chosen for recognized scholarly competence and with due consideration for the balance of disciplines appropriate to the project. The opinions and conclusions expressed or implied are those of the research agency that performed the research, and, while they have been accepted as appropriate by the technical committee, they are not necessarily those of the Transportation Research Board, the National Research Council, the American Association of State Highway and Transportation Officials, or the Federal Highway Administration, U.S. Department of Transportation.

Each report is reviewed and accepted for publication by the technical committee according to procedures established and monitored by the Transportation Research Board Executive Committee and the Governing Board of the National Research Council.

Published reports of the

NATIONAL COOPERATIVE HIGHWAY RESEARCH PROGRAM

are available from:

Transportation Research Board
Business Office
500 Fifth Street, NW
Washington, DC 20001

and can be ordered through the Internet at:

http://www.national-academies.org/trb/bookstore

Printed in the United States of America

THE NATIONAL ACADEMIES

Advisers to the Nation on Science, Engineering, and Medicine

The **National Academy of Sciences** is a private, nonprofit, self-perpetuating society of distinguished scholars engaged in scientific and engineering research, dedicated to the furtherance of science and technology and to their use for the general welfare. On the authority of the charter granted to it by the Congress in 1863, the Academy has a mandate that requires it to advise the federal government on scientific and technical matters. Dr. Bruce M. Alberts is president of the National Academy of Sciences.

The **National Academy of Engineering** was established in 1964, under the charter of the National Academy of Sciences, as a parallel organization of outstanding engineers. It is autonomous in its administration and in the selection of its members, sharing with the National Academy of Sciences the responsibility for advising the federal government. The National Academy of Engineering also sponsors engineering programs aimed at meeting national needs, encourages education and research, and recognizes the superior achievements of engineers. Dr. William A. Wulf is president of the National Academy of Engineering.

The **Institute of Medicine** was established in 1970 by the National Academy of Sciences to secure the services of eminent members of appropriate professions in the examination of policy matters pertaining to the health of the public. The Institute acts under the responsibility given to the National Academy of Sciences by its congressional charter to be an adviser to the federal government and, on its own initiative, to identify issues of medical care, research, and education. Dr. Harvey V. Fineberg is president of the Institute of Medicine.

The **National Research Council** was organized by the National Academy of Sciences in 1916 to associate the broad community of science and technology with the Academy's purposes of furthering knowledge and advising the federal government. Functioning in accordance with general policies determined by the Academy, the Council has become the principal operating agency of both the National Academy of Sciences and the National Academy of Engineering in providing services to the government, the public, and the scientific and engineering communities. The Council is administered jointly by both the Academies and the Institute of Medicine. Dr. Bruce M. Alberts and Dr. William A. Wulf are chair and vice chair, respectively, of the National Research Council.

The **Transportation Research Board** is a division of the National Research Council, which serves the National Academy of Sciences and the National Academy of Engineering. The Board's mission is to promote innovation and progress in transportation through research. In an objective and interdisciplinary setting, the Board facilitates the sharing of information on transportation practice and policy by researchers and practitioners; stimulates research and offers research management services that promote technical excellence; provides expert advice on transportation policy and programs; and disseminates research results broadly and encourages their implementation. The Board's varied activities annually engage more than 5,000 engineers, scientists, and other transportation researchers and practitioners from the public and private sectors and academia, all of whom contribute their expertise in the public interest. The program is supported by state transportation departments, federal agencies including the component administrations of the U.S. Department of Transportation, and other organizations and individuals interested in the development of transportation. **www.TRB.org**

www.national-academies.org

COOPERATIVE RESEARCH PROGRAMS STAFF FOR NCHRP REPORT 507

ROBERT J. REILLY, *Director, Cooperative Research Programs*
CRAWFORD F. JENCKS, *Manager, NCHRP*
DAVID B. BEAL, *Senior Program Officer*
EILEEN P. DELANEY, *Managing Editor*
HILARY FREER, *Associate Editor*

NCHRP PROJECT 24-17 PANEL
Field of Design—Area of Bridges, Other Structures, and Hydraulics and Hydrology

TERRY SHIKE, P.E., *David Evans and Associates, Inc., Salem, Oregon* (Chair)
PAUL F. BAILEY, P.E., *New York State DOT*
RICHARD BARKER, P.E., *Blacksburg, Virginia*
JERRY A. DIMAGGIO, P.E., *FHWA*
WILLIAM S. FULLERTON, P.E., *Montana DOT*
ROBERT E. KIMMERLING, P.E., *PanGeo, Inc., Seattle, Washington*
MARK J. MORVANT, P.E., *Louisiana DOTD, Louisiana Transportation Research Center*
PAUL PASSE, P.E., *PSI, Tampa, Florida*
JEFF SIZEMORE, P.E., *South Carolina DOT*
CARL EALY, *FHWA Liaison Representative*
G. P. JAYAPRAKASH, *TRB Liaison Representative*

AUTHOR ACKNOWLEDGMENTS

The presented research was sponsored by the American Association of State Highway and Transportation Officials (AASHTO), under project 24-17, in cooperation with the Federal Highway Administration (FHWA). The panel of the research project is acknowledged for their comments and suggestions. The interest, support, and suggestions of Mr. David Beal of the NCHRP are highly appreciated. Messrs. Jerry DiMaggio, Al DiMillio, and Carl Ealy of the FHWA are acknowledged for their concern and support. Dr. Gregory Baecher and Dr. Bilal Ayyub from the University of Maryland contributed to sections 1.3.1 through 1.3.4, section 1.4.3.4, sections 2.6.1 through 2.6.3, and section 3.3, and performed the calculations of the presented resistance factors based on FORM. Dr. Mike McVay, Dr. Bjorn Birgisson, and Mr. Thai Nguyen of the University of Florida, and Dr. Ching Kuo of Geostructures compiled the static analyses databases and carried out the analyses related to the material presented in sections 2.1.1, 2.1.2, 2.3.1, 2.5, 3.1.2, and 3.1.4. Dr. Frank Rausche of Goble, Rausche, Likins (GRL) and Associates provided the data pertaining to the evaluation of GRLWEAP as the WEAP method for dynamic pile capacity evaluation. Mr. Kirk Stenersen researched the performance of the dynamic analyses as part of his graduate studies at the University of Massachusetts Lowell. The help of Ms. Mary Canniff and Ms. Laural Stokes in the preparation of the manuscript is appreciated.

FOREWORD

By David B. Beal
Staff Officer
Transportation Research Board

This report contains the findings of a study to develop resistance factors for driven pile and drilled shaft foundations. These factors are recommended for inclusion in Section 10 of the AASHTO *LRFD Bridge Design Specifications* to reflect current best practice in geotechnical design and construction. The report also provides a detailed procedure for calibrating deep foundation resistance. The material in this report will be of immediate interest to bridge engineers and geotechnical engineers involved in the design of pile and drilled shaft foundations.

Full implementation of the AASHTO *LRFD Bridge Design Specifications* for deep foundations is hampered by provisions that are inconsistent with current geotechnical engineering practice. Static pile-capacity analyses are typically used to estimate required pile lengths and quantities, whereas dynamic analyses are used to determine pile capacity during pile driving. Currently, the resistance factors for static and dynamic analysis are multiplied by each other, resulting in designs that are significantly more conservative than used in past practice, increasing foundation costs.

Resistance factors for drilled shafts in sand or gravel are not provided in the LRFD Specifications, and many of the state departments of transportation do not have the data or the resources to do their own calibrations as recommended in the specification. The effect of various construction techniques on drilled shaft resistance factors also is not addressed in the LRFD Specifications.

The resistance factors for deep foundations were not calibrated for the LRFD load factors. In addition, the resistance factors do not account for the variability of the site conditions and the number of load tests conducted. Another shortcoming is that many accepted design procedures, some of which are commonly recommended by FHWA, are not supported by the LRFD Specifications.

The objective of this research was to address the aforementioned issues and to provide resistance factors for the load and resistance factor design of deep foundations. Under NCHRP Project 24-17, the University of Massachusetts at Lowell with the assistance of D'Appolonia, the University of Maryland, the University of Florida, and the University of Houston assembled databases for static analysis of drilled shafts and driven piles and for dynamic analysis of driven piles. These databases were used for the statistical evaluation of resistance factors. Extensive appendices providing detailed information on the development and application of the resistance factors are included on NCHRP CD-39 bound with the report.

CONTENTS

LOAD AND RESISTANCE FACTOR DESIGN (LRFD) FOR DEEP FOUNDATIONS

SUMMARY

NCHRP Project 24-17 was aimed at rewriting AASHTO's Deep Foundation Specifications. The AASHTO specifications are traditionally observed on all federally aided projects and generally viewed as a national code of U.S. highway practice; hence they influence the construction of all the deep foundations of highway bridges throughout the United States. This report presents the results of the studies and analyses conducted for that project.

The development of load and resistance factors for deep foundations design is presented. The existing AASHTO specifications, similar to others worldwide, are based on Load and Resistance Factor Design (LRFD) principles. The presented research is the first, however, to use reliability-based calibration-utilizing databases. Large databases containing case histories of piles tested to failure were compiled and analyzed.

The state of the art was examined via a literature review of design methodologies, LRFD principles, and deep foundation codes. The state of the practice was established via a questionnaire, distributed to and gathered from state and federal transportation officials. Large databases were gathered and provided. Analyses of the data, guided by the state of practice led to findings detailing the performance of various static and dynamic analyses methods when compared to recorded pile performance. Static capacity evaluation methods used in common design practices were found overall to over-predict the observed pile capacities. Common dynamic capacity evaluation methods used for quality control were found overall to under-predict the observed pile capacities. Both findings demonstrate the shortcoming of safety parameter evaluation based on absolute values (i.e., resistance factors or factors of safety) and the need for an efficiency parameter to allow for an objective measure to assess the performance of methods of analysis.

The parameters that control the accuracy of the predictions were researched and analyzed for the dynamic methods. A set of controlling parameters was established to allow calibration of the prediction methods.

Target reliability magnitudes were researched and values were recommended considering the action of piles in a redundant or non-redundant form. Statistical analyses compatible with common practice in the structural area were utilized for the development of LRFD resistance factors. Parameters that control the size of a testing sample

and site variability were researched and incorporated. Recommended design parameters offering a consistent reliability in design were then presented and discussed.

The need for the modification of LRFD for use in geotechnical applications through knowledge-based parameters accounting for subsurface variability, quality of soil parameters estimation, and previous experience as well as amount and type of testing during construction is presented.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

1.1 BACKGROUND

National Cooperative Highway Research Program Project NCHRP 24-17, "LRFD Deep Foundations Design," was initiated to provide (1) recommended revisions to the driven pile and drilled shaft portions of section 10 of the AASHTO *LRFD Bridge Design Specifications* (AASHTO, 2001) and (2) a detailed procedure for calibrating deep foundation resistance factors. The current AASHTO specifications, as well as other existing codes based on Load and Resistance Factor Design (LRFD) principles, were calibrated using a combination of reliability theory, fitting to Allowable Stress Design (ASD—also called Working Stress Design, or WSD), and engineering judgment. The main challenges of the project were, therefore, the compilation of large, high-quality databases and the development of a procedural and data management framework that would enable LRFD parameter evaluation and future updates. Meeting these challenges required (1) organizing the resistance factors into a design-construction-quality-control sequence (i.e., independence in resistance factors according to the chronological stage and the evaluation procedure) and (2) overcoming the generic difficulties of applying the LRFD methodology to geotechnical applications, i.e., incorporation of indirect variability (e.g., site or parameters interpretation), judgment based on previous experience, and similar factors into the methodology. The project team, headed by the author, was divided into three groups dealing respectively with static analyses (University of Florida), probabilistic and structural analyses (University of Maryland), and dynamic analyses (University of Massachusetts Lowell).

This chapter provides a background for design methodologies and LRFD principles and usage. In Chapter 2, following a discussion of the major findings from a questionnaire and survey designed to discover the state of current practice, the databases that were developed for the project are presented and analyzed. Selected design methods are described, followed by an in-depth evaluation of the dynamic methods for the evaluation of the capacity of driven piles and an examination of their controlling parameters. The performance of different prediction methods, categorized according to the examined methods of analysis and controlling parameters, are also discussed in Chapter 2. In Chapter 3, the results of these analyses are used for the development of the resistance factors recommended for the revision of the AASHTO *LRFD Bridge Design Specifications*. Statistical methods are used for the development of recommendations for number of piles to be tested in quality assurance. Chapter 4 presents the conclusions supported by the study, suggestions for additional research, and a framework for LRFD for deep foundations that incorporates knowledge-based design. Detailed data and analyses are provided in the appendices available on the accompanying CD.

1.2 STRESS DESIGN METHODOLOGIES

1.2.1 Working Stress Design

The working Stress Design (WSD) method, also called Allowable Stress Design (ASD), has been used in Civil Engineering since the early 1800s. Under WSD, the design loads (Q), which consist of the actual forces estimated to be applied to the structure (or a particular element of the structure), are compared to resistance, or strength (R_n) through a factor of safety (FS):

$$Q \le Q_{all} = \frac{R_n}{FS} = \frac{Q_{ult}}{FS} \tag{1}$$

Where Q = design load; Q_{all} = allowable design load; R_n = resistance of the element or the structure, and Q_{ult} = ultimate geotechnical pile resistance.

Table 1, from *Standard Specifications for Highway Bridges* (AASHTO, 1997), presents common practice, the traditional factors of safety used in conjunction with different levels of control in analysis and construction. Presumably, when a more reliable and consistent level of control is used, a smaller FS can be used, which leads to more economical design. Practically, however, the factors of safety in Table 1 do not necessarily consider the bias, in particular, the conservatism (i.e., underprediction) of the methods listed; hence, the validity of their assumed effect on the economics of design is questionable. (These traditional factors of safety are further discussed and evaluated in section 3.5.2)

1.2.2 Limit States Design

In the 1950s, the demand for more economical design of piles brought about the use of Limit States Design (LSD).

TABLE 1 Factor of safety on ultimate axial geotechnical capacity based on level of construction control (AASHTO, 1997)

Basis for Design and Type of Construction Control	Increasing Design/Construction Control				
Subsurface Exploration	X	X	X	X	X
Static Calculation	X	X	X	X	X
Dynamic Formula	X				
Wave Equation		X	X	X	X
CAPWAP Analysis			X		X
Static Load Test				X	X
Factor of Safety (FS)	3.50	2.75	2.25	2.00*	1.90

*For any combination of construction control that includes a static load test, FS =2.0.

Within LSD, two types of limit states are usually considered, Ultimate Limit State (ULS), and Serviceability Limit State (SLS). ULS pertains to structural safety and involves structural collapse or, in relation to piles, the ultimate bearing capacity of the soil. SLS pertains to conditions, such as excessive deformations and settlement or deterioration of the structure that would affect the performance of the structure under expected working loads.

The formula for ULS is

$$\text{Factored resistance} \geq \text{Factored load effects} \tag{2}$$

The formula for SLS is

$$\text{Deformation} \leq \text{Tolerable deformation to remain serviceable} \tag{3}$$

1.3 LOAD AND RESISTANCE FACTOR DESIGN (LRFD)

1.3.1 Principles

The design of a pile depends upon predicted loads and the pile's capacity to resist them. Both loads and capacity have various sources and levels of uncertainty. Engineering design has historically compensated for these uncertainties by using experience and subjective judgment. On the other hand, these uncertainties can be quantified using probability-based methods aimed at achieving engineered designs with consistent levels of reliability. The intent of Load and Resistance Factor Design (LRFD) is to separate uncertainties in loading from uncertainties in resistance and then to use procedures from probability theory to ensure a prescribed margin of safety.

Figure 1 shows probability density functions (PDFs) for load effect, Q, and resistance, R. "Load effect" is the load calculated to act on a particular element, (e.g., a specific pile). As loads are usually better known than are resistances, the load effect typically has smaller variability than resistance (i.e., a smaller coefficient of variation, translating to a narrower probability density function). Since failure is defined as the load effect exceeding the resistance, the probability of

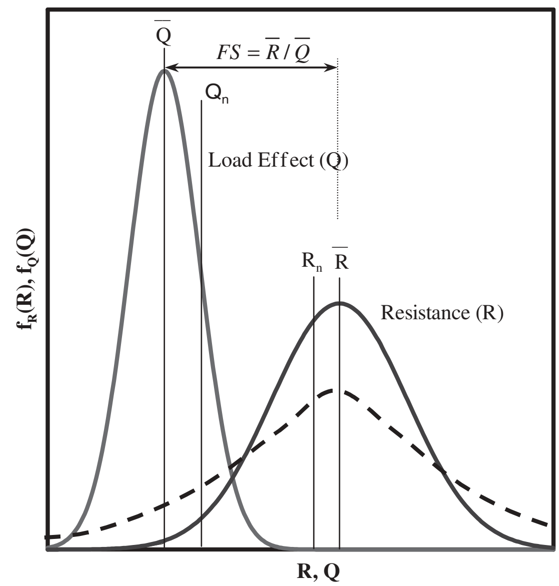


Figure 1. An illustration of probability density functions for load effect and resistance.

failure ($P_f = P(R < \overline{Q})$) is related to the extent to which the two probability density functions overlap (although not simply to the area of overlap).

In LRFD, partial safety factors are applied separately to the load effect and resistance. Strength is reduced and load effects are increased, by multiplying the corresponding characteristic (or nominal) values by factors called strength (resistance) and load factors, respectively. Using this approach, the factored (i.e., reduced) strength of a pile must be larger than a linear combination of the factored (i.e., increased) load effects. The nominal values (e.g., the nominal strength, R_n) are those calculated by the specific calibrated design method and are not necessarily the means (i.e., the mean loads, $\overline{Q}$, or mean resistance, $\overline{R}$ (Figure 1). For example, $\overline{R}$ might be the mean of dynamic signal matching analysis predictions calculated in many case histories, while R_n is the predicted value for the specific analyzed pile.

Based on considerations ranging from case histories to existing design practice, a prescribed value is chosen for probability of failure. Then, for a given pile design based on the application of resistance and load factors, the probability for failure, that is, the probability that the factored loads exceed the factored resistances, should be smaller than the prescribed value. In foundation practice, the factors applied to load effects are typically transferred from structural codes, and then resistance factors are specifically calculated to provide the prescribed probability of failure.

The importance of uncertainty regarding resistance can be seen by reference to Figure 1. In this figure, the mean factor of safety is $\overline{FS} = \overline{R}/\overline{Q}$, whereas the nominal factor of safety is $FS_n = R_n/Q_n$. Consider what happens if the uncertainty in resistance is increased, and thus the PDF broadened, as suggested by the dashed curve. The mean resistance for this other predictive method remains unchanged, but the variation (i.e., uncertainty) is increased.

In calculating the prescribed probability of failure (P_f), a derived probability density function is calculated for the margin of safety (R,Q), and reliability is expressed using the "reliability index", β, which is the number of standard deviations of the derived PDF of R − Q separating the mean safety margin from the nominal failure value of zero (Figure 2). Further discussion of the relationship of p_f to β are given in section 2.7.1. For computational reasons, the margin of safety is taken as R − Q when the resistances and load effects have normally distributed uncertainty, but as ln(R) − ln(Q) when the uncertainties are logNormally distributed.

1.3.2 Background Information

The concept of using the probability of failure as a criterion for structural design is generally credited to the Russians N. F. Khotsialov and N. S. Streletskii who presented it in the late 1920s, and it was introduced in the United States by Freudenthal (1947). The recent development of LRFD in civil engineering was initiated in structural engineering (see, e.g., Ellingwood et al., 1980). Reliability-Based Design codes using LRFD have been published by the American Institute of Steel Construction (AISC, 1994; Galambos and Ravindra 1978) and the American Concrete Institute (American Concrete Institute, 1995). An effort was made by the National Standards Institute (ANSI) to develop probability-based load criteria for buildings (Ellingwood et al., 1982a, b) and ASCE 7-93 (ASCE, 1993). The American Petroleum Institute (API) extrapolated LRFD technology for use in fixed offshore platforms (API, 1989; Moses 1985, 1986). Comprehensive summaries of the implementation of probabilistic design theory in design codes include those by "Practical Approach to Code Calibration" (Siu et al., 1975) for the National Building Code of Canada (National Research Council of Canada, 1977), *Development of a Probability-Based Load Criterion for American National A58* (Ellingwood et al., 1980) for the National Bureau of Standards, and the *Rationalization of Safety and Serviceability Factors in Structural Codes: CIRIA Report 63* (Construction Industry Research and Information Association, 1977). The AASHTO *LRFD Bridge Design Specifications* (AASHTO, 1994), resulting from work in NCHRP Project 12-33 (Nowak, 1999), provide design guidance for girders.

1.3.3 LRFD Performance and Advantages

Experience has shown that adopting a probability-based design code can result in cost savings and efficient use of materials. Reliability improvements are still under evaluation even though the new LRFD codes are designed to yield reliabilities equal to or higher than those of earlier codes. Experiences are not yet well documented; but anecdotal evidence from naval architecture suggests that, relative to conventional WSD, the new AISC-LRFD requirements may save 5% to 30% of steel weight in ships (Ayyub, 1999). This may or may not be the case for civil engineering applications. Specific benefits for pile design include at least the following:

1. Cost savings and improved reliability because of more efficiently balanced design.
2. More rational and rigorous treatment of uncertainties in the design.
3. Improved perspective on the overall design and construction processes (sub- and superstructures); and the development of probability-based design procedures can stimulate advances in pile analysis and design.

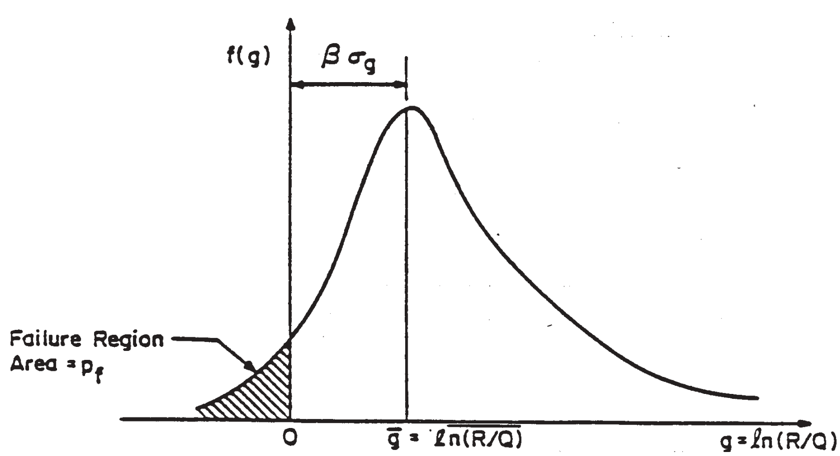


Figure 2. An illustration of a combined probability density function (g(R,Q)) representing the margin of safety and the reliability index, β*.* (σ_g *= Standard deviation of g(R,Q)).*

4. Transformation of the codes into living documents that can be easily revised to include new information reflecting statistical data on design factors.
5. The partial safety factor format used herein also provides a framework for extrapolating existing design practice to new foundation concepts and materials where experience is limited.

1.3.4 LRFD in Geotechnical Engineering

Early use of LSD for geotechnical applications was examined by the Danish geotechnical institute (Hansen 1953, 1956) and later formulated into code (Hansen, 1966). Independent load and resistance factors were used, with the resistance factors applied directly to the soil properties rather than to the nominal resistance.

Considerable effort has been directed over the past decade on the application of LRFD in geotechnical engineering. LRFD approaches have been developed in offshore engineering (e.g., Tang, 1993; Hamilton and Murff, 1992), general foundation design (e.g., Kulhawy et al., 1996), and pile design for transportation structures (Barker et al., 1991; O'Neill, 1995).

In geotechnical practice, uncertainties concerning resistance principally manifest themselves in design methodology, site characterization, soil behavior, and construction quality. The uncertainties have to do with the formulation of the physical problem, interpreting site conditions, understanding soil behavior (e.g., its representation in property values), and accounting for construction effects. Uncertainties in external loads are small compared with uncertainties in soil and water loads and the strength-deformation behaviors of soils. The applied loads, however, are traditionally based on superstructure analysis, whereas actual load transfer to substructures is poorly researched. The approach for selecting load and resistance factors developed in structural practice, though a useful starting point for geotechnical applications, is not sufficient. Work is needed to incorporate factors that are unique to geotechnical design into the LRFD formulation.

Philosophically, the selection of load and resistance factors does not have to be made probabilistically, although in current structural practice a calibration based on reliability theory is commonly used. This approach focuses more on load uncertainties than resistance uncertainties and does not include many subjective factors unique to geotechnical practice. An expanded approach is needed if the full benefits of LRFD are to be achieved for foundation design. The National Research Council reports that the "subjective approach reflects the general lack of robust data sources from which a more objective set of factors can be derived" (National Research Council, 1995). The report continues, "realistically, because of the tremendous range of property values and site conditions that one may encounter, it is unlikely that completely objective factors can be developed in the foreseeable future."

Today, the situation has changed somewhat, but not entirely. The present research team gathered robust data on pile capacity from which a more objective calibration of resistance factors could be made. Nonetheless, there remain uncertainties associated with (1) site conditions, (2) soil behavior and the interpretation of soil parameters, and (3) construction methods and quality. These factors are difficult to understand from the pile databases alone. Such knowledge-based factors should be combined with the reliability-theory-based calibration of the database records to achieve a meaningful LRFD approach, requiring a major research effort. These difficulties are addressed in the present research through the calibration of specific combinations of design and parameter interpretation methods.

1.3.5 LRFD for Deep Foundations

Several efforts have been made to develop LRFD-based codes for deep foundation design.

1.3.5.1 2001 AASHTO LRFD Bridge Design Specifications for Driven Piles

LRFD Bridge Design Specifications (AASHTO, 2001) states that the ultimate resistance (R_n) multiplied by a resistance factor (ϕ), which thus becomes the factored resistance (R_r), must be greater than or equal to the summation of loads (Q_i) multiplied by corresponding load factors (γ_i), and a modifier (η_i). For strength limit states:

$$R_r = \phi R_n \geq \sum \eta_i \gamma_i Q_i \tag{4}$$

where:

$$\eta_i = \eta_D \eta_R \eta_I > 0.95 \tag{5}$$

where η_i = factors to account for; η_D = effects of ductility; η_R = redundancy; and η_I = operational importance.

The *Specifications* provide the following equations for determining the factored bearing resistance of piles, Q_R,

$$Q_R = \phi Q_n = \phi_q Q_{ult} = \phi_{qp} Q_p + \phi_{qs} Q_s \tag{6}$$

for which:

$$Q_p = q_p A_p \tag{7}$$

$$Q_s = q_s A_s \tag{8}$$

where ϕ_q = resistance factor for the bearing resistance of a single pile specified for methods that do not distinguish between total resistance and the individual contributions of tip resistance and shaft resistance; Q_{ult} = bearing resistance of a single pile; Q_p = pile tip resistance; Q_s = pile shaft resistance (F); q_p = unit tip resistance of pile; q_s = unit shaft resistance of pile; A_s = surface area of pile shaft; A_p = area of pile tip; and ϕ_{qp}, ϕ_{qs} = resistance factor for tip and shaft resistance, respectively, for those methods that separate the resistance of a pile into contributions from tip resistance and shaft resistance.

The resistance factors for use in the above equations are presented in Table 10.5.5-2 of the *Specifications* for different design methods based on soil type and area of resistance (tip and side). The resistance factors for compression vary between 0.45 and 0.70. The table also incorporates a factor, λ_v, for different methods and level of field capacity verification. As an example, if, in analysis, an α method is used to determine the pile's friction resistance in clay, a resistance factor of 0.70 is recommended. If, in verification of the pile capacity, a pile driving formula, e.g., an ENR (Engineering News-Record) equation, is used without stress wave measurements during driving, a λ_v factor of 0.80 is recommended. The actual resistance factor to be used in the above analysis verification sequence is, therefore, 0.56 (i.e., 0.70×0.80).

1.3.5.2 2001 AASHTO LRFD Bridge Design Specifications for Drilled Shafts

LRFD Bridge Design Specifications (AASHTO, 2001) provides detailed resistance factors for a large number of design methods for drilled shafts. Differentiation is made between base and side resistance, as for driven piles, with resistance factors varying between 0.45 and 0.65. Static testing is included with the same resistance factor as for driven piles (0.8). Resistance factors are not provided for drilled shafts in sand. The λ_v factor, used for field verification for driven piles, is not used for drilled shafts, and no distinction is made on the basis of construction method.

1.3.5.3 Worldwide LRFD Codes for Deep Foundations and Drilled Shafts

A review of foundation design standards in the world was conducted by the Japanese Geotechnical Society (1998). A review of the development of LRFD applications for Geotechnical Engineering is presented by Goble (1999). A review of LRFD parameters for dynamic analyses of piles is presented by Paikowsky and Stenerson in Appendix B. The present section provides a short review of non-US LRFD codes for deep foundations.

The Australian Standard for Piling-Design and Installation (1995) provides ranges of resistance factors for static load tests (0.7 to 0.9) and static pile analyses (0.40–0.65) related to the source of soil parameters and soil type (e.g., SPT in cohesionless soils). Detailed recommendations are provided for resistance factors to be used with the dynamic methods ranging between 0.45 to 0.65 for methods without dynamic measurements (including WEAP), and between 0.50 to 0.85 when utilizing dynamic measurements with signal matching analysis. Selection of the appropriate resistance factor depends on driving conditions, geotechnical factors (e.g., extent of site investigation), and extent of testing (e.g., low range for <3% of the pile tested and high range for >15%). In traditional structural design specifications, a nominal value is given and the value used is based primarily on engineering judgment and cannot exceed the nominal value. The Australian Standard is therefore unique by providing a guide for choosing the appropriate resistance factor. Interestingly, no distinction is made regarding either soil type or time of driving (i.e. EOD, BOR) when referring to the signal matching based on dynamic measurements. The method by which the resistance factors were generated is not provided in the code.

The AUSTROADS Bridge Design Code (1992) provides resistance factors for the construction stage alone including static load test (to failure $\phi = 0.9$, proof test $\phi = 0.8$), and four categories of dynamic methods. The range of resistance factors is quite large and there is no explanation as to how the resistance factors were obtained. Goble (1999) postulates that the resistance factors were calibrated via the working stress design method.

The Ontario Bridge Code (1992) recommends relatively low resistance factors with no differentiation between the individual static or dynamic analyses. For example, the resistance factors for static analyses and static load tests in compression and tension are 0.4, 0.3, 0.6 and 0.4 respectively. No information is provided on how the resistance factors were obtained.

The Bridge Code (1992) is brief in its design requirements for deep foundations. Resistance factors are based on pile type, $\phi = 0.4$ for all timber and concrete piles (precast, filled pipe, and cast in place) and 0.5 for steel piles. For dynamic load testing, resistance factors of 0.4 and 0.5 are recommended for routine testing and analyses based on dynamic measurements, respectively.

Eurocode 7 (1997) deals with driven piles and drilled shafts at a single section. Factors for static load testing depend on the number of tested piles (irrelevant to the number of piles at the specific site). Range of values from 0.67 to 0.91 is provided for one to three tests, related to the mean or lowest value of the test results. The code is quite complex with quantitative descriptions and limiting conditions. The code is presented with multiple component factors, and for comparison with the form used by U.S. codes, Goble (1999) inverted and combined the factors resulting in values ranging from 0.63 to 0.77 for base, skin, and total resistance of driven, bored, and CFA piles. DiMaggio et al. (1998) presented a summary report of a geotechnical engineering study tour, stating "The team found Eurocode 7 to be a difficult document to read and understand, which may explain the various interpretations that were expressed in the countries visited." Improvements in that direction were achieved through a text that explains the methodology and provides design examples (Orr and Farrell, 1999; see also Orr, 2002). The final draft of the future Eurocode 7 (October 2001, see also Frank, 2002) is an extensive code that is expected to become an EN publication by August 2004. This detailed document contains 12 sections dealing with all geotechnical design aspects ranging from geotechnical data (section 3), to construction supervision (section 4), to hydraulic failure (section 10). Section 7 is dedicated to pile foundations. While not very detailed regarding a specific determination of the pile capacity, the code is elaborating for all cases (i.e., static load test results, static and dynamic methods) factors to be applied to both the minimum and average of the capacity as a function of the number of applications. For example, static load test capacity will have factors (to be divided by) ranging from 1.4 to 1.0 when applied to the results of 1 to 5 or over load tests. Specifically, if, for example, three static load tests are carried out, the mean value of the three will be divided by 1.2, and the minimum value by 1.05, and the lower of the two will determine the factored resistance to be used.

Substantially fewer details are provided by the codes for LRFD design of drilled shafts. The two extremes being the aforementioned Bridge Code (1992), in which drilled shafts are included under a single category of cast-in-place piles ($\phi = 0.4$ like all other concrete piles), and the AASHTO relatively detailed provisions described in section 1.3.5.2.

1.3.5.4 Difficulties with the Existing LRFD Codes

All existing codes suffer from two major difficulties. One is the application of LRFD to geotechnical problems as described in section 1.3.4 (e.g., site variability, construction effects, past experience, etc.). The other problem is lack of data. None of the reviewed codes and associated resistance factors were consistently developed based on databases enabling the calculation of resistance factors from case histories.

The current AASHTO specifications of driven piles reviewed in section 1.3.5.1 encounter additional difficulty due to the multiplication of the resistance factor by the modifier λ_v. This procedure requires the interaction of two independent pile capacity evaluations (e.g., static analysis and dynamic methods) and results in unnecessary and confusing conservatism. A clear separation of the resistance factors on the basis of design and construction is required and is one aim of the present study. As a result of the aforementioned difficulties, the current AASHTO LRFD specifications for geotechnical applications are of limited use. Two surveys presented in this report (see section 2.1) found that only 14 states (30%) are currently committed to the use of LRFD in foundation design. In contrast 93% of the responding use WSD, suggesting that most of those that use LRFD are utilizing the methodology in parallel to WSD.

1.4 RESEARCH APPROACH

1.4.1 Design and Construction Process of Deep Foundations

Figure 3 presents a flow chart depicting the design and construction process of deep foundations. Commonly, design starts with site investigation and soil parameter evaluation, assessments that vary in quality and quantity according to the importance of the project and complexity of the subsurface. Possible foundation schemes are identified based on the results of the investigation, load requirements, and local practice. All possible schemes are evaluated via static analyses. Schemes for driven piles also require dynamic analysis (drivability) for hammer evaluation, feasibility of installation, and structural adequacy of the pile. In sum, the design stage combines, therefore, structural and geotechnical analyses to determine the best prebidding design. This process leads to estimated quantities to appear in construction bidding documents.

Upon construction initiation, static load testing and/or dynamic testing, or dynamic analysis based on driving resistance (using dynamic formulas or wave-equations) are carried out on selected elements (i.e., indicator piles) of the original design. Pile capacity is evaluated based on the construction phase testing results, which determine the assigned capacity and final design specifications. In large or important projects, the pile testing may also be used as part of the design. Two requirements are evident from this process: (1) pile evaluation is carried out at both the design and the construction stage, and (2) these two evaluations should result in foundation elements of the same reliability but possibly different number and length of elements depending on the information available at each stage.

1.4.2 Overview of the Research Approach

The complete application of LRFD to the process described in Figure 3 requires an integrated framework. For example, the method by which a field test (say SPT) is used to obtain soil parameters must be coordinated with the method used for static capacity of the pile, and both must be coordinated with the assessment of uncertainty. Independently, one needs to evaluate the design verification process during construction, i.e., static load testing and dynamic testing to assess and modify the pile installation, as well as quality assurance (e.g., nondestructive testing of drilled shafts) and related issues.

Previous LRFD developments, using back analysis of ASD and judgment, have addressed some of these issues (e.g., Withiam et al., 1998). The present effort to assemble a case history database adds other difficulties, for example determining a "predicted" capacity that can be compared with measured load-test values.

The present effort was focused on calibrating the direct design and construction evaluation process. For the design, specific methods and correlations were chosen. Their results

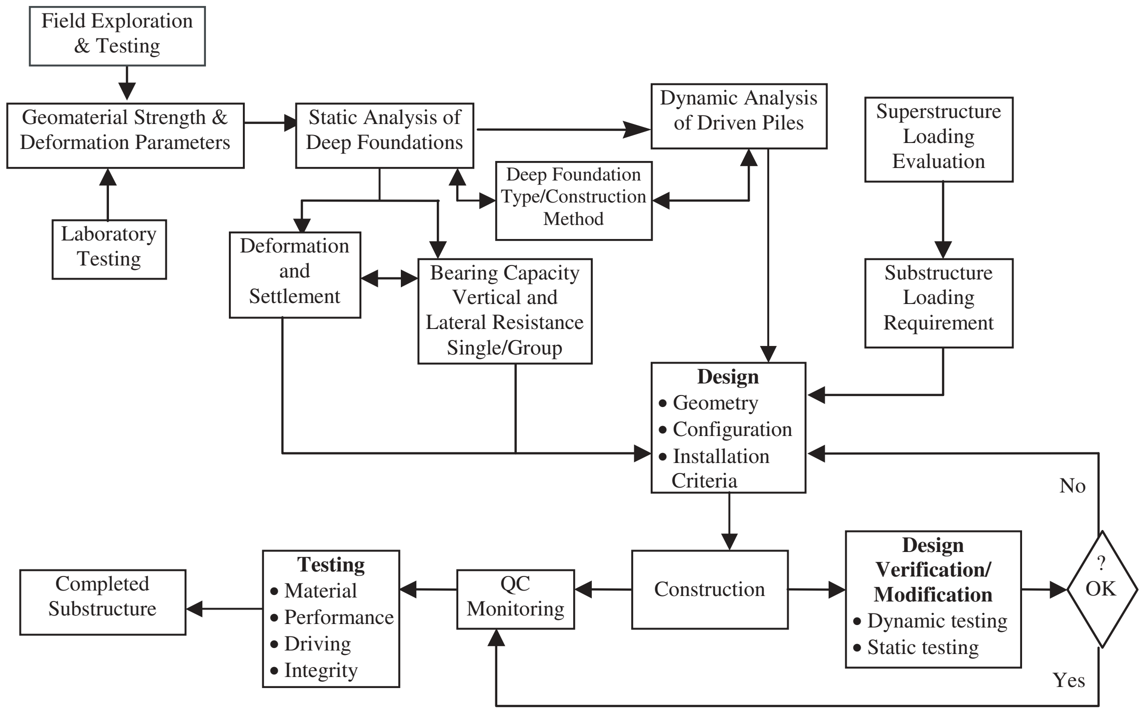


Figure 3. Design and construction process for deep foundations.

(i.e., static capacity evaluations) were compared to measured pile performance under static load. In the dynamic analysis case, the database was used to identify controlling parameters, which were then calibrated. A description of the principles used for the assessments of the three databases is provided in section 1.4.3.3. Figure 4 presents a flowchart of the research approach for this study. The flowchart outlines the framework required for LRFD calibration of design and construction methods of analysis. The stages outlined in Figure 4 are described in the following sections; findings and evaluations related to the various stages of the framework are presented in Chapter 2.

1.4.3 Principles and Framework of the Calibration

1.4.3.1 Determination of Analysis Methods

To establish the state of practice, a questionnaire was developed and distributed to all state highway and federal highway organizations. The material related to the questionnaire and detailed results are presented in Appendix A, on the accompanying CD, and discussed in section 2.1.

1.4.3.2 Databases

Three principal databases and six secondary databases were developed for the evaluation of the analysis methods and interpretation procedures. The major databases—drilled shaft, driven piles, and PD/LT2000—are presented in Appendices B and C, on the accompanying CD, and discussed in section 2.2. The secondary databases are referred to and used as applicable.

1.4.3.3 Conceptual Evaluation of Driven Piles and Drilled Shafts Capacities

Driven Piles—Static Analysis. The vast majority of the database case histories were related to SPT and CPT field testing. Four correlations of soil parameters from SPT and CPT were identified. The case histories were divided on the basis of soil condition (clay, sand, and mixed) and pile types (H pile, concrete piles, pipe piles). In summary, given field conditions were used via various soil parameter identifications and pile capacity evaluation procedures to determine capacities. The capacities were then compared to measured static capacity. Details of the analyses are presented in section 2.3.

Driven Piles—Dynamic Analysis. The dynamic evaluation of driven piles is the most common way to determine capacity during construction. Existing AASHTO specifications, as described in section 1.3.5.1, are complicated by the use of a factor, λ_v, which convolves the design stage and the construction stage. Therefore, a fresh look at the basis for dynamic calibration was required. Details are described in Paikowsky and Stenersen (2000) and in section 2.4.

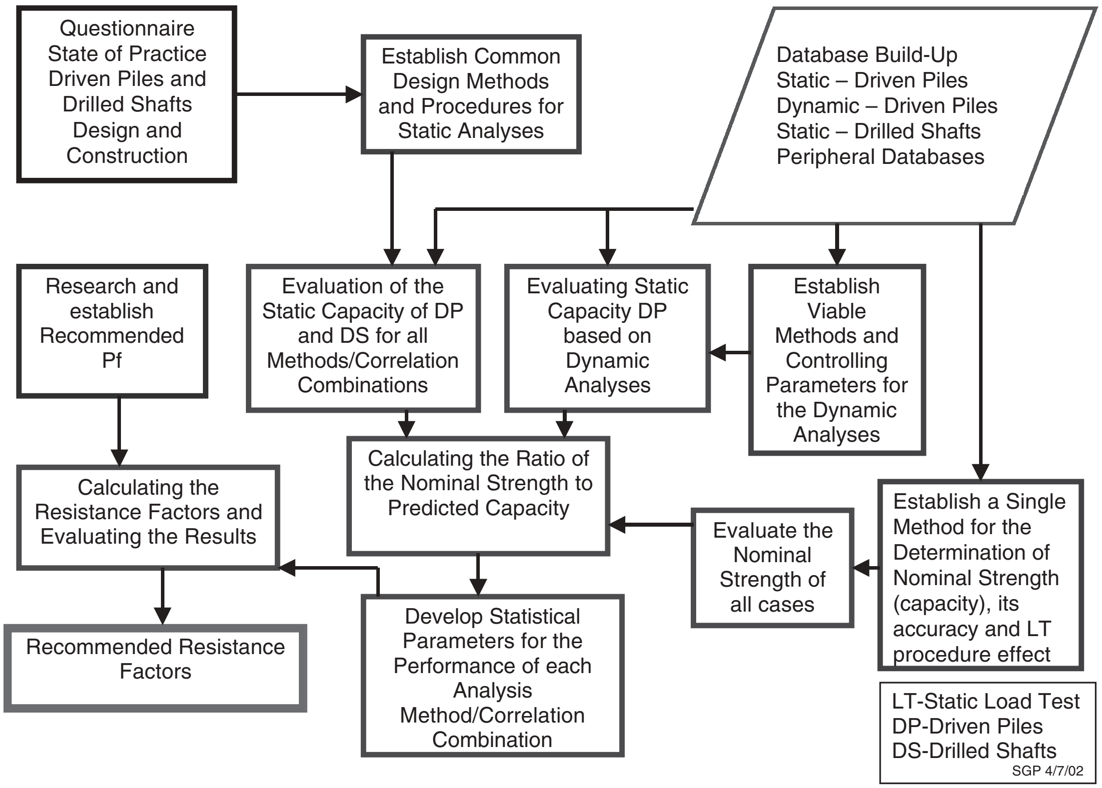


Figure 4. Stages of the research approach outlining the framework for LRFD calibration of the current study.

Drilled Shafts—Static Analysis. Evaluation of the design of drilled shafts is difficult as limited data are available for the separation of capacity components (i.e., shaft and tip), and as both components of capacity are affected by the method of construction. The following procedure was used for the evaluation of the measured skin capacities. The shape of the load-displacement curves was evaluated, and shafts for which more than 80% of the total capacity was mobilized in a displacement of less than 2% of the shaft diameter were considered as having resistance based on friction. Results of these procedures were compared to static analyses as described in section 2.6.

1.4.3.4 LRFD Calibration

Existing AASHTO Specifications. Existing AASHTO specifications are based on First-Order, Second-Moment (FOSM) analysis, using $\eta = 1$ in equation 4, and assuming lognormal distributions for resistance. This leads to the relation (Barker et al., 1991),

$$\phi = \frac{\lambda_R\left(\sum \gamma_i Q_i\right)\sqrt{\dfrac{1+COV_Q^2}{1+COV_R^2}}}{\overline{Q}\exp\left\{\beta_T\sqrt{\ln\left[\left(1+COV_R^2\right)\left(1+COV_Q^2\right)\right]}\right\}} \tag{9}$$

where:

λ_R = resistance bias factor
COV_Q = coefficient of variation (the ratio of the standard deviation to the mean) of the load
COV_R = coefficient of variation of the resistance
β_T = target reliability index

When just dead and live loads are considered, equation 9 can be rewritten as:

$$\phi = \frac{\lambda_R\left(\dfrac{\gamma_D Q_D}{Q_L}+\gamma_L\right)\sqrt{\left[\dfrac{\left(1+COV_{Q_D}^2+COV_{Q_L}^2\right)}{\left(1+COV_R^2\right)}\right]}}{\left(\dfrac{\lambda_{Q_D} Q_D}{Q_L}+\lambda_{Q_L}\right)\exp\left\{\beta_T\sqrt{\ln\left[\left(1+COV_R^2\right)\left(1+COV_{Q_D}^2+COV_{Q_L}^2\right)\right]}\right\}} \tag{10}$$

where:

γ_D, γ_L = dead and live load factors
Q_D/Q_L = dead to live load ratio
λ_{QD}, λ_{QL} = dead and live load bias factors

Present Project Calibration. LRFD for structural design has evolved beyond FOSM to the more invariant First-Order

Reliability Method (FORM) approach (e.g., Ellingwood et al., 1980, Galambos and Ravindra, 1978), while geotechnical applications have lagged behind (Meyerhof, 1994). In order to be consistent with the current structural code and the load factors to which it leads, it is necessary for calibration of resistance factors for deep foundations to use FORM (Nowak, 1999).

Following Ayyub and Assakkaf (1999), the present project calibrates LRFD partial safety factors using FORM, as developed by Hasofer and Lind (1974). FORM can be used to assess the reliability of a pile with respect to specified limit states and provides a means for calculating partial safety factors ϕ and γ_i for resistance and loads, respectively, against a target reliability level, β_O. FORM requires only first and second moment information on resistances and loads (i.e., means and variances) and an assumption of distribution shape (e.g., normal, lognormal, etc.). The calibration process using FORM is presented in Figure 5.

In design practice, there are usually two types of limit state: ultimate limit state and serviceability limit state. Each can be represented by a performance function of the form,

$$g(X) = g(X_1, X_2, \ldots, X_n) \tag{11}$$

in which $X = (X_1, X_2, \ldots, X_n)$ is a vector of basic random variables of strengths and loads. The performance function g(X), often called the limit state function, relates random variables to either the strength or serviceability limit-state. The limit is defined as g(X) = 0, implying failure when g(X) < 0 (Figures 2 and 5). The reliability index, β, is the distance from the origin of the space of basic random variables to the failure surface at the most probable point on that surface, that is, at the point on $g(X) = 0$ at which the joint PDF of X is greatest. This is sometimes called the *design point* and is found by an iterative solution procedure (Thoft-Christensen and Baker, 1982). The relationship of the limit states can also be used to back calculate representative values of the reliability index, β, from current design practice.

The computational steps for determining β using FORM are the following:

1. In the regular coordinates, assume a design point, x_i^*, and, in a reduced coordinate system, obtain its corresponding point, $x_i'^*$, using the transformation:

$$x_i'^* = \frac{x_i^* - \mu_{X_i}}{\sigma_{X_i}} \tag{12}$$

where

μ_{X_i} = mean value of the basic random variable X_i,
σ_{X_i} = standard deviation of the basic random variable.

The mean value of the vector of basic random variables is often used as an initial guess for the design point. The notation x^* and x'^* is used to denote the design point in the regular coordinates and in the reduced coordinate system, respectively.

2. If the distribution of basic random variables is nonnormal, approximate this distribution with an equivalent normal distribution at the design point, having the same tail area and ordinate of the density function, that is with equivalent mean,

$$\mu_X^N = x^* - \Phi^{-1}\left(F_X(x^*)\right)\sigma_X^N \tag{13}$$

and equivalent standard deviation

$$\sigma_X^N = \frac{\phi\left(\Phi^{-1}\left(F_X(x^*)\right)\right)}{f_X(x^*)} \tag{14}$$

where

μ_X^N = mean of the equivalent normal distribution,
σ_X^N = standard deviation of the equivalent normal distribution,
$F_X(x^*)$ = original cumulative distribution function (CDF) of X_i evaluated at the design point,
$f_X(x^*)$ = original PDF of X_i evaluated at the design point,
$\Phi(\cdot)$ = CDF of the standard normal distribution, and $\phi(\cdot)$ = PDF of the standard normal distribution.

3. Set $x_i'^* = \alpha_i^*\beta$, in which the α_i^* are direction cosines. Compute the directional cosines (α_i^*, $i = 1, 2, \ldots, n$) using,

$$\alpha_i^* = \frac{\left(\dfrac{\partial g}{\partial x_i'}\right)_*}{\sqrt{\displaystyle\sum_{i=1}^{n}\left(\dfrac{\partial g}{\partial x_i'}\right)_*^2}} \quad \text{for } i = 1, 2, \ldots, n \tag{15}$$

where

$$\left(\frac{\partial g}{\partial x_i'}\right)_* = \left(\frac{\partial g}{\partial x_i}\right)_* \sigma_{X_i}^N \tag{16}$$

4. With α_i^*, $\mu_{X_i}^N$, $\sigma_{X_i}^N$ now known, the following equation is solved for β:

$$g\left[\left(\mu_{X_1}^N - \alpha_{X_1}^*\sigma_{X_1}^N\beta\right), \ldots, \left(\mu_{X_n}^* - \alpha_{X_n}^*\sigma_{X_n}^N\beta\right)\right] = 0 \tag{17}$$

5. Using the β obtained from step 4, a new design point is obtained from,

$$x_i^* = \mu_{X_i}^N - \alpha_i^*\sigma_{X_i}^N\beta \tag{18}$$

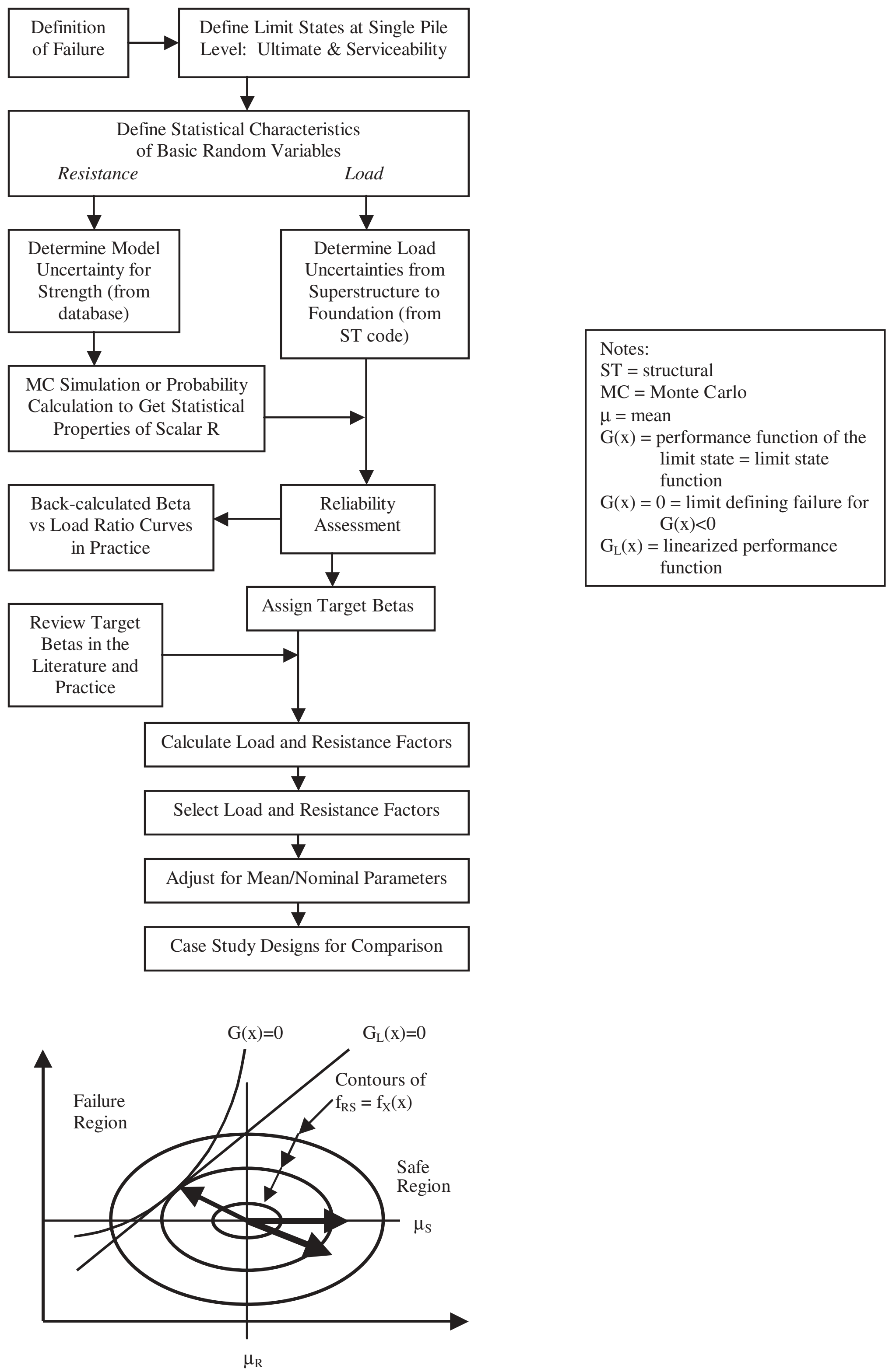


Figure 5. Resistance factor analysis flow chart (after Ayyub and Assakkaf, 1999 and Ayyub et al., 2000), using FORM developed by Hasofer and Lind (1974).

6. Repeat steps 1 to 5 until convergence of β is achieved. This reliability index is the shortest distance to the failure surface from the origin in the reduced coordinate space.

FORM can be used to estimate partial safety factors such as those found in the design format. At the failure point $(R^*, L_1^* - \ldots - L_n^*)$, the limit state is given by,

$$g = R^* - L_1^* - \ldots - L_n^* = 0 \tag{19}$$

or, in a more general form by,

$$g(X) = g\left(x_1^*, x_2^*, \ldots, x_n^*\right) = 0 \tag{20}$$

The mean value of the resistance and the design point can be used to compute the mean partial safety factors for design as,

$$\phi = \frac{R^*}{\mu_R} \tag{21}$$

$$\gamma_i = \frac{L_i^*}{\mu_{L_i}} \tag{22}$$

In developing code provisions, it is necessary to follow current design practice to ensure consistent levels of reliability over different pile types. Calibrations of existing design codes are needed to make the new design formats as simple as possible and to put them in a form that is familiar to designers. For a given reliability index β and probability distributions for resistance and load effects, the partial safety factors determined by the FORM approach may differ with failure mode. For this reason, calibration of the calculated partial safety factors (PSFs) is important in order to maintain the same values for all loads at different failure modes. In the case of geotechnical codes, the calibration of resistance factors is performed for a set of load factors already specific in the structural code (see following section). Thus, the load factors are fixed. In this case, the following algorithm is used to determine resistance factors:

1. For a given value of the reliability index, β, probability distributions and moments of the load variables, and the coefficient of variation for the resistance, compute mean resistance R using FORM.
2. With the mean value for R computed in step 1, the partial safety factor, ϕ, is revised as:

$$\phi = \frac{\sum_{i=1}^{n} \gamma_i \mu_{Li}}{\mu_R} \tag{23}$$

where μ_{Li} and μ_R are the mean values of the load and strength variables, respectively, and γ_i, i = 1, 2,…, n, are the given set of load factors.

Load Conditions and Load Factors. The actual load transferred from the superstructure to the foundations is, by and large, unknown, with very little long-term research having been focused on the subject. The load uncertainties are taken, therefore, as those used for the superstructure analysis. *LRFD Bridge Design Specifications* (AASHTO, 2000) provide five load combinations for the standard strength limit state (using dead, live, vehicular, and wind loads) and two for the extreme limit states (using earthquake and collision loads). The use of a load combination that includes lateral loading may at times be the restrictive loading condition for deep foundations design. Pile lateral capacity is usually controlled by service limit state, and as such, was excluded from the scope of the present study, which focuses on the axial capacity of single piles/drilled shafts. The load combination for strength I was therefore applied in its primary form as shown in the following limit state:

$$Z = R - D - LL \tag{24}$$

Where R = strength or resistance of pile, D = dead load and LL = vehicular live loads. The probabilistic characteristics of the random variables D and LL are assumed to be those used by AASHTO (Nowak, 1999) with the following load factors and lognormal distributions (bias and COV) for live and dead loads, respectively:

$$\gamma_L = 1.75 \qquad \lambda_{QL} = 1.15 \qquad COV_{QL} = 0.2 \tag{25}$$

$$\gamma_D = 1.25 \qquad \lambda_{QD} = 1.05 \qquad COV_{QD} = 0.1 \tag{26}$$

For the strength or resistance (R), the probabilistic characteristics are defined in Chapter 3, based on the databases for the various methods and conditions that are described in Chapter 2.

CHAPTER 2

FINDINGS

2.1 STATE OF PRACTICE

2.1.1 Questionnaire and Survey

Code development requires examining the state of practice in design and construction in order to address the needs, research the performance, and examine alternatives. The identification of current design and construction methodologies was carried out via a questionnaire along with a survey, which was independently developed and analyzed by Mr. A. Munoz of the FHWA. The questionnaire was distributed to 298 state highway officials, TRB representatives, and state and FHWA geotechnical engineers. A total of 45 surveys were returned and analyzed (43 states and 2 FHWA personnel). The survey elicited information concerning design methodology in geotechnical and structural design, foundation alternatives, and design and constitution considerations for both driven piles and drilled shafts. The questionnaire, the survey, and their analyzed results are presented in Appendix A. A summary analysis of the survey results is presented below.

2.1.2 Major Findings

2.1.2.1 Design Methodology

Averaging the responses for driven piles and drilled shafts, about 90% of the respondents used ASD, 35% used AASHTO Load Factor Design (LFD), and 28% used AASHTO Load and Resistance Factors Design (LRFD), suggesting that most of the respondents that use LRFD or LFD use it in parallel with WSD.

Among the respondents using ASD to evaluate capacity, 95% used a global safety factor ranging from 2.0 to 3.0, depending on construction control and 5% used partial safety factors of 1.5 to 2.0 for side friction (3.0 for drilled shafts) and 3.0 for end bearing (2.0 to 3.0 for drilled shafts).

2.1.2.2 Foundation Alternatives

The majority of the respondents use primarily driven pile foundations (75%), 14% use shallow foundations, and 11% use drilled shafts. Of those responding, 64% prefer the use of driven piles and 5% prefer drilled shafts or other foundation type. When using driven piles, 21% primarily use prestressed concrete piles; 52%, steel H piles; 2%, open-ended steel pipe piles; and 25%, closed-end steel pipe piles.

2.1.2.3 Driven Piles—Design Considerations

1. The most common methods used for evaluating the static axial capacity of driven piles were as follows:
   - 59%: α-method (Tomlinson, 1987),
   - 25%: β-method (Esrig & Kirby, 1979),
   - 5%: λ–method (Vijayvergiya and Focht, 1972),
   - 75%: Nordlund's method (Nordlund, 1963),
   - 5%: Nottingham and Schmertmann's method: CPT (1975),
   - 9%: Schmertmann's method: SPT (Sharp, 1987),
   - 14%: Meyerhof's method (1976) modified by Zeitlen and Paikowsky (1982), and
   - 25%: in-house methods and other less common methods.

   Of the computer programs used in design,
   - 39% were developed in-house,
   - 75% were FHWA developed, and
   - 20% were from commercial vendors.
2. Of the primary tests used to assess strength parameters in design, 86% used SPT-N values, 11% used CPT data, 2% used Dilatometer data, and none used Pressuremeter data.
3. The majority of the states used Tomlinson's method to assess the side friction coefficient in cohesive soil (C_A – adhesion) and Nordland's method in cohesionless soil (δ – interfacial friction angle).
4. Pile settlement in the design was considered by 48%, with settlement ranging from 0.25 to 1.0 inches being tolerable.
5. Simplified methods (e.g., Broms, 1964) were used by 34% of the respondents in the lateral pile design methods and/or computer programs, and 88% used methods based on p-y curves. Of the computer programs used in design, 14% were in-house, 82% were from the FHWA, and 55% came from commercial vendors.

6. Responses for the estimated risk or failure probability of the group foundation design were as follows:
   - 27% less than 0.1%,
   - 4% between 0.1 and 1%,
   - 1% of the responses were between 1% and 10%, and
   - 67% were unknown.

The assessment for the acceptable maximum failure probability ranged from about 0 to 1%. Pile failure had been experienced by 14% of the respondents.

2.1.2.4 Driven Piles—Construction Considerations

1. Of the respondents, 77% performed static pile load test during construction, and the primary test method was the Quick Method.
2. The most common dynamic methods used for capacity evaluation of driven piles included the following:
   - Wave Equation Analysis using the program GRL-WEAP (GRL Engineers, Inc. Wave Equation Analysis Program) was used by 80% of the respondents.
   - 45% used the ENR formula,
   - 16% used Gate's equation with safety factors ranging from 2.0 to 3.5, and
   - 1 state used its own dynamic formula.
3. Dynamic pile load tests were performed during construction by 84 % of respondents, testing 1% to 10% of the piles per bridge.
4. When setting production pile length and driving criteria, 82% used EOD conditions, 52% used BOR conditions, and 36% did not consider pile freeze or relaxation effects in determining driving criteria.

2.1.2.5 Drilled Shafts—Design Considerations

1. The most common methods used for evaluating the static axial capacity of drilled shafts were as follows:
   - 36%: the α-method (total stress approach) (Reese and O'Neill, 1998; Kulhawy, 1989),
   - 41%: the β-method (effective stress approach) (Reese and O'Neill, 1988),
   - 9%: the Reese and Wright (1977) approach for side friction in cohesionless soils,
   - 39%: the FHWA (O'Neill et al., 1996) approach for intermediate geomaterials (soft rock),
   - 11%: Carter and Kulhawy (1988) approach for intermediate geomaterials (soft rock), and
   - 27%: other methods.

   Of the computer programs used, 18% were developed in-house, 50% came from the FHWA, 29% from commercial vendors, and 20% from others.
2. Of the primary parameters used, 70% were based on SPT values, 7% were obtained from the CPT test, 2% were based on Pressuremeter data, and 2% were based on Dilatometer data.
3. Of the 16% considering the roughness of the borehole wall in rock socket design, all did so by assumption.
4. Shaft settlement was considered by 61% of the respondents, with tolerable settlements ranging from 0.25 to 2.0 in.
5. Simplified (e.g., Brooms, 1964) lateral drilled shaft design methods and/or computer programs were used by 27%, and 82% used methods based on p-y curves.
6. For drilled shafts subjected to lateral load, the tolerable deflection ranged from 0.25 to 2.0 in., and the safety factor of lateral pile capacity ranged from 1.5 to 3.0.
7. About 30% of the respondents did not take into account the construction method in design.
8. Concerning the estimated risk or probability of failure of group foundation designs based on the safety factor used, the following responses were made:
   - 20%: less than 0.1%,
   - 7%: between 0.1 and 1%,
   - 2%: between 1 and 10%, and
   - 71%: unknown.

The assessment for the acceptable maximum failure probability ranged from about 0% to 5%.

2.1.2.6 Drilled Shafts—Constructions Considerations

1. 66% performed static load testing during construction.
2. The type of load test used included conventional static load testing (32%), Osterberg load cell (43%), Statnamic load testing (11%), and Dynamic load testing (7%).
3. The methods used in drilled shaft installations included drilling in dry (64%), wet (52%), and casing methods (86%).
4. For the drilling slurry used during construction, 25% used a mineral slurry of processed Attapulgite, 52% used a mineral slurry of Bentonite clays, and 36% used synthetic polymer slurries.
5. A majority of the States use the AASHTO Specifications for shaft cleanliness, which requires more than 50% of the base to have less than 0.5 in. of sediment and maximum sediment thickness to be less than 1.5 in.
6. 54% performed inspection of the shaft bottom, in which only one State has a specific inspection device. The rest performed inspection by using manual probes or an underwater camera and camcorder.
7. 16% did not perform integrity testing for drilled shaft quality control; 64% used Cross Sonic Logging

(CSL), 7% used Surface Reflection (Pulse Echo Method), and 7% used Gamma Ray or NX coring.

2.2 DATABASES

2.2.1 General

Three major databases were developed for the primary statistical evaluation of resistance factors for the design and construction of driven piles and drilled shafts. Six additional peripheral databases were assembled and/or used for the investigation of specific issues as needed. The major features of the databases are described below. The detailed cases from which the databases were developed are presented in Appendix B (dynamic) and Appendix C (static).

2.2.2 Drilled Shaft Database—Static Analysis

The soil type and method of construction of the 256 case histories in the drilled shaft database are detailed in Table 2. The database was developed at the University of Florida, mostly through the integration of databases gathered by the Florida DOT, the Federal Highway Administration (FHWA), and O'Neill et al. (1996).

2.2.3 Driven Pile Database—Static Analysis

The soil and pile type of the 338 case histories in the driven pile database are detailed in Table 3. The database was developed at the University of Florida, mostly through the integration of databases gathered by the University of Florida, the FHWA (see, e.g., DiMillio, 1999), the University of Massachusetts Lowell (see, e.g., Paikowsky et al., 1994), and the Louisiana Transportation Research Center.

2.2.4 Driven Pile Database—Dynamic Analysis

The PD/LT2000 database contains information related to 210 driven piles that have been statically load tested to failure and dynamically monitored during driving and/or restrike (403 analyzed measurements). PD/LT2000 comprises information from the PD/LT database (Paikowsky et al., 1994), the PD/LT2 database (Paikowsky and LaBelle, 1994), and 57 additional pile case histories described by Paikowsky and Stenersen (2000). The data in PD/LT2000 were carefully examined and analyzed following procedures described by Paikowsky et al. (1994), resulting in detailed static and dynamic pile capacity evaluations. Table 4 presents a summary of the data contained in PD/LT2000, broken down according to pile type and capacity range, site location, soil type, factors affecting soil inertia, and time of driving (EOD or BOR).

TABLE 2 Summary and breakdown—drilled shafts database

Soil/Rock Type	Method of Construction					
	Casing		Slurry		Dry	
	Total	Skin	Total	Skin	Total	Skin
Sand	13	6	15	4	6	1
Clay	14	3	0	0	40	10
Mixed Soils	23	4	12	5	13	7
Rock	0	0	0	0	8	0
Sand & Rock	4	4	7	5	20	0
Clay & Rock	2	0	2	0	19	7
Mixed Soils & Rock	2	1	0	0	2	0
Total (256)	58	32	36	14	91	25

Note: Total = skin + tip; Skin = side alone

TABLE 3 Driven piles database: soil type and number of cases by type of pile

Soil Type		Number of Cases		
Tip	Side	H-PILES	PPC	PIPE
Rock	Clay	3	0	0
	Sand	12	0	0
	Mix	6	15	3
	Total	21	15	3
Sand	Clay	0	0	0
	Sand	17	37	20
	Mix	13	50	19
	Total	30	87	39
Clay	Clay	8	19	20
	Sand	1	1	0
	Mix	36	34	15
	Total	44	54	35
Insufficient data		0	7	1
All cases (338)		97	163	78

2.3 DEEP FOUNDATIONS NOMINAL STRENGTH

2.3.1 Overview

Probabilistic calibration of resistance factors for any predictive method utilizing a database is possible when the nominal geotechnical pile strength (i.e., static pile capacity) is defined and compared to the outcome of the calibrated prediction method. The definition of ultimate static capacity given static load test results (load-displacement relations) is not unique, and the use of the term "reference static capacity for calibration" (may include judgment) is more appropriate than "nominal strength." The static load test results depend on the load testing procedures and the applied interpretation method, often being subjective. The following sections examine each of these factors and its influence on the reference static capacity, concluding with a recommended unique procedure to be followed in the calibration.

2.3.2 Failure Criterion for Statically Loaded Driven Piles

Past work related to driven piles (Paikowsky et al., 1994) has resorted to a representative static pile capacity based on

TABLE 4 The PD/LT2000 database: pile type, geographical location, soil type, soil inertia, type of data, and pile capacities

Pile Types			Geographical Location		Soil Types			Soil Inertia			Type of Data		Pile Capacities	
Pile Type		No.	Location	No.	Soil Type	Side	Tip	Criteria	Blow Ct.	AR	Time	No.	Range (kN)	No.
H –Pile		37	Northeast USA	44	Clay /Till	67	61	≥ 16 blows /10cm	272	----	EOD & BOR	92	0-445	2
OEP		10	Southeast USA	69									445-890	6
CEP		61	North USA	24							EOD & BORs	30	890-1334	17
Voided Concrete		35	South USA	10				< 16 blows /10cm	112	----			1334-1779	44
Sq. Conc	254	9	Northwest USA	3	Rock	0	11				EOD	135	1779-2224	27
	305	5	Southwest USA	14									2224-2669	25
	356	8	Australia	2				≥ 350	-----	134	BOR	239	2669-3114	15
	406	1	New Brunswick	3									3114-3559	10
	457	8	Holland	4	Sand /Silt	140	137				EOR	11	3559-4003	13
	508	8	Hong Kong	4				< 350	-----	255			4003-4448	13
	610	16	Israel	4							DD	2	4448-4893	11
	762	5	Ontario	22									4893-5338	6
Octagonal Concrete		3	Sweden	1	NA	3	1	NA	5	----	DR	1	5338-5783	5
			NA	6									5783-6228	4
Timber		2									ALT	1	6228-6672	6
Monotube		2											>6672	6
Total		210		210		210	210		389	389		389		210

Notes: Pile types: OEP = Open Ended Pipe Pile; CEP=Closed Ended Pipe Pile.
Geographic Location: Northeast USA = Federal Highway Regions 1, 2 & 3; Southeast USA = Federal Highway Region 4;
North USA = Federal Highway Regions 5, 7 & 8; South USA = Federal Highway Region 6;
Northwest USA = Federal Highway Region 10; Southwest USA = Federal Highway Region 9.
Type of Data: EOD = End of Driving; BOR = Beginning of Restrike; EOD & BOR = Cases containing both EOD & BOR;
EOD & BOR's = Cases containing both EOD & multiple BOR measurements; EOR = End of Restrike;
DD = During Driving; DR = During Restrike; ALT = Alternate measurement.
NA = Non Applicable / unknown

the assessment by five interpretation methods; (1) Davisson's Criterion (Davisson, 1972), (2) Shape of Curve (similar to the procedure proposed by Butler and Hoy, 1977), (3) Limiting Total Settlement to 25.4 mm, (4) Limiting Total Settlement to 0.1B (Terzaghi, 1942), and (5) the DeBeer log-log method (DeBeer, 1970).

A single representative capacity value was then calculated for the analyzed case as the average of the methods considered relevant (i.e., provided reasonable value). The development of a calibration in a framework suitable for future modifications requires that the evaluated resistance factors be based on an objective, reproducible procedure. In order to do so, the static capacity of each pile in database PD/LT2000 was evaluated according to all five aforementioned criteria and a representative capacity was assigned for each pile. The mean and standard deviations of the ratio of the representative pile capacity to the capacity given by the method being evaluated was then determined. Details of the analyses and their results are presented by Paikowsky and Stenerson in Appendix B. Figure 6 shows the histogram and calculated distributions (normal and lognormal) for Davisson's failure criterion in which K_{SD} is the ratio of the designated static capacity to that defined by Davisson's failure criterion. Davisson's criterion was found to perform the best overall and was therefore chosen as the single method to be used when analyzing load-displacement curves. Davisson's method provides an objective failure criterion and was also found to perform well for piles exceeding a diameter of 610 mm (examined through 30 pile cases). The data presented in Figure 6 demonstrates, however, that (1) a small bias exists in the static capacity being used as a reference for the evaluation of the methods predicting the capacity of driven piles, and (2) this bias (and other considerations) needs to be accounted for when evaluating the resistance factor to be used for field static load tests.

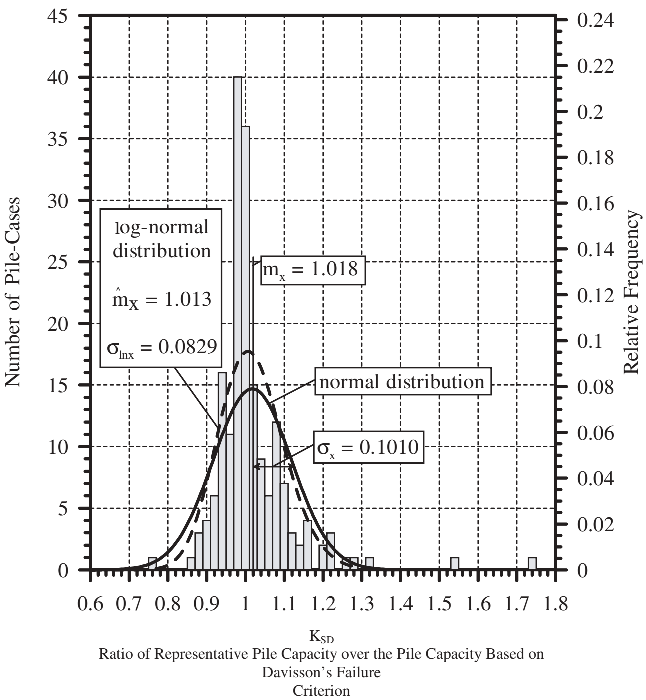


Figure 6. Histogram and frequency distributions of K_{SD} for 186 PD/LT2000 pile-cases in all types of soils. (Paikowsky and Stenersen, 2000).

2.3.3 Load Test Procedure for Statically Loaded Driven Piles

The influence of the static load testing procedure (loading rate) on the designated pile capacity was examined in two ways.

Two detailed case histories from a research site in Newburyport, Massachusetts, were evaluated. A pipe pile and prestressed concrete heavily instrumented friction pile were tested over a lengthy period at a bridge reconstruction site. Both piles were tested using three types of static load testing procedures: slow maintained (testing duration of about 45 hrs), short duration (testing duration of about 6 to 8 hrs), and static cyclic (testing duration of about 15 min). Details about the piles and the testing are presented by Paikowsky and Hajduk (1999, 2000) and Paikowsky et al. (1999). The interpretation of the load-displacement relationships in both cases suggested that the test type had an insignificant influence on the pile capacity (referring to a failure criterion irrespective of the displacement).

The effect of the test type was further investigated utilizing a database containing information related to 75 piles tested under slow maintained and static-cyclic load testing procedures. In the static-cyclic procedure, the piles were loaded to failure using a high loading rate and then unloaded. The process was repeated for four cycles. The testing procedure and its interpretation method are presented by Paikowsky et al. (1999). A comparison between the pile capacity based on Davisson's failure criterion for the slow maintained tests and the static-cyclic capacity is presented in Figure 7. The obtained relations and the associated statistical information suggest that there is no significant influence on the static pile capacity based on the applied static load rate.

The static-cyclic load test results were also compared to the representative static pile capacity (based on the aforementioned five methods), resulting in a mean K_{SC} of 1.023 and a standard deviation of 0.057.

These evaluations led to the conclusion that Davisson's pile failure criterion can be used to determine the reference pile capacity for driven piles, irrespective of the pile's diameter and the static load-testing procedure.

2.3.4 Failure Criterion for Statically Loaded Drilled Shaft

Static load tests of small- to medium-capacity drilled shafts (say up to 5 MN) are similar to that of driven piles. It is common, however, for example in the Northeast region of the United States, to design and build high-capacity drilled shafts (10 MN and more), often as an alternative to a large group of small-capacity driven piles. The testing for capacity of such shafts is a challenge that often requires alternatives to the common external reaction testing, for example, the Osterberg load-cell (Osterberg, 1992), statnamic tests (Bermingham and White, 1995, Middendorp and Bielefeld, 1995), and drop

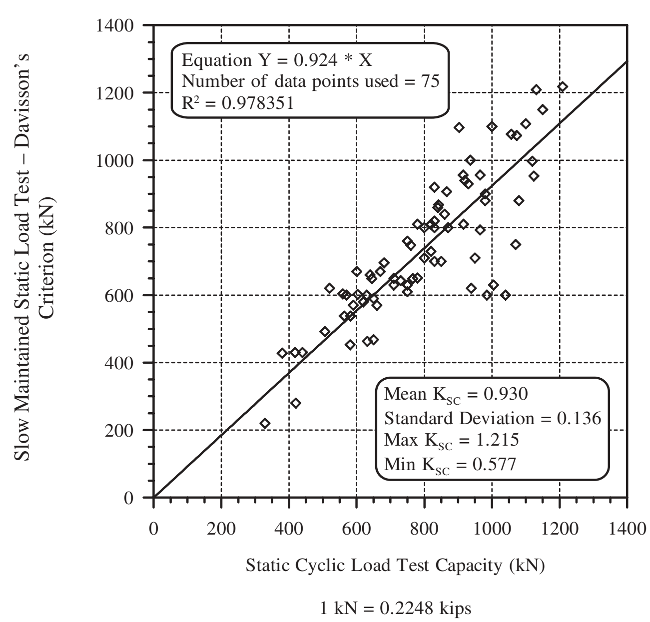


Figure 7. Comparison between pile capacity based on Davisson's criterion for slow maintained load tests and static-cyclic load test capacity for 75 piles. (Paikowsky et al., 1999).

weight dynamic testing. Another National Cooperative Highway Research Program project, NCHRP-21-08 "Innovative Load Testing Systems," headed by the principal author of this report, examines such alternative methods. As part of this ongoing project, the static load-test results of statically loaded drilled shafts were examined utilizing the failure criteria previously described for driven piles, and the FHWA criterion for drilled shafts (O'Neill and Reese, 1999). The FHWA criterion establishes the failure load as that associated with a displacement of 5% of the diameter at the shaft, if plunging of the shaft cannot be achieved. The results of this preliminary study, presented in Table 5, suggest that the FHWA criterion provides a reliable and simple failure interpretation. For the presented LRFD calibration study, the FHWA failure criterion for drilled shaft (i.e., load at a displacement of 0.05 B) was, therefore, adopted.

2.4 DRIVEN PILES—STATIC ANALYSIS METHODS

Table 6 presents a summary of the methods used for static capacity evaluation of driven piles detailing the equations for side and tip resistances, required parameters, and constraints on their use. The associated correlations used to evaluate the soil properties from SPT and CPT tests are presented in Tables 7a and 8, respectively. While two internal friction angle interpretations are listed in Table 7 and were used initially, only the method proposed by Peck Hansen and Thornburn was found to provide more realistic results, and hence utilized in the calibrated analyses. The methods and the correlations listed in Tables 7a and 8 are based on the state of practice established via the questionnaire (see section 2.1 and Appendix A.) Table 7b elucidates the combinations and the manner in which the correlations were applied. The notations used in Table 7b are further noted when the analysis results are reported. The tables were, by and large, prepared as part of the study of static pile capacity at the University of Florida, which is presented in Appendix C.

2.5 DRIVEN PILES—DYNAMIC ANALYSIS METHODS

2.5.1 Overview

Prior to detailed analyses leading to the determination of resistance factors, two components must be established: (1) the

TABLE 5 Evaluation of failure criteria for statically loaded drilled shafts

Statistics for the Ratio between Drilled Shaft Capacity of Different Interpretation Methods and the Representative Capacity											
Davisson			DeBeer			Shape of Curve			FHWA		
#	m_x	σ_x	#	m_x	σ_x	#	m_x	σ_x	#	m_x	σ_x
47	0.862	0.17	39	0.908	0.11	36	0.956	0.09	40	0.999	0.13

Notes: # = no. of cases; m_x = mean; σ_x=standard deviation; loads 0.85 to 20 MN; diameter 0.3 to 1.5m; length 5.3 to 58.5m

TABLE 6 Summary of static capacity methods for driven piles

Method	Side resistance	Tip resistance	Parameters required	Constraints
α-Tomlinson (Tomlinson, 1980/1995)	$q_s = \alpha S_u$	$q_p = 9\ S_u$	S_u; D_b (bearing embedment)	+Bearing layer must be stiff cohesive + Number of soil layers ≤ 2
α-API (Reese et al., 1998)			S_u	
β in cohesive (AASHTO, 1996/2000)	$q_s = \beta\sigma'$		OCR	
λ (US Army Corps of Engineers, 1992)	$q_s = \lambda(\sigma'+2S_u)$		S_u	Only for cohesive soils
β in cohesionless (Bowles, 1996)	$\beta\sigma'$		D_r	
Nordlund and Thurman (Hannigan et al., 1995)	$q_s = K_\delta C_F \sigma' \frac{\sin(\delta+\varpi)}{\cos\varpi}$	$q_p = \alpha_t N'_q\ \sigma'$	ϕ	
Meyerhof SPT (Meyerhof, 1976/1981)	$q_s = k\ N$	$q_p = 0.4D/BN'$	N	+ For cohesionless soils + SPT data
Schmertmann SPT (Lai and Graham, 1995)	q_s = function(N)	q_p = fn(N)	N	SPTdata
Schmertmann CPT (McVay and Townsend, 1989)	q_s = function(f_s)	q_p = fn(q_c)	q_c, f_s	CPT data

TABLE 7a Correlations of soil properties from SPT

Properties	From SPT	Reference (Kulhawy & Mayne, 1990)
ϕ	Peck, Hanson and Thornburn: $\approx 54 - 27.6034 \exp(-0.014N')$	Figure 4.12
	Schmertmann $\varphi \approx \tan^{-1}[N / (12.2 + 20.3\, \sigma'/p_a')]^{0.34}$	Figure 4.13 and Equation 4.11
S_u (bar)	Terzaghi and Peck (1967): 0.06 N	Equation 4.59
	Hara 1974: $0.29\, N^{0.72}$	Equation 4.60
OCR for clay	Mayne and Kemper $\approx 0.5\, N / \sigma'_o$ (σ'_o in bar)	Figures 3.9 and 3.18
Dr	Gibbs and Holtz's Figures	Figures 2.13 and 2.14

type of the dynamic methods to be evaluated and (2) the conditions under which these methods need to be examined. Sections 2.5.2 and 2.5.3 address these issues, respectively, based on a detailed study by Paikowsky and Stenerson provided in Appendix B.

2.5.2 Methods of Analysis

2.5.2.1 General

Table 9 presents a summary of the major available dynamic methods for evaluating pile capacity. The methods are subdivided according to the project stage (i.e., design vs. construction) and the need for data obtained through dynamic measurements. The incorporation of dynamic equations and WEAP reflects the need to address the state of practice as described in section 2.1.

The methods that require dynamic measurements can be broadly categorized as those that utilize a simplified analysis of an instantaneous pile capacity evaluation for each hammer blow and those that require elaborate calculations (i.e., signal matching) traditionally carried out in the office.

2.5.2.2 WEAP

Based on Smith (1960), the use of the WEAP (Goble and Rausche, 1976) during design is of great importance for achieving compatibility between the driving system, the pile, and the soil conditions. Drivability studies and pile stress analyses often determine the pile type and geometry and the adequacy of the proposed equipment. Typically, two analyses are carried out: one by the designer during the design stage (prebid), in which a range of equipment to be specified in the bidding documents is examined, and the other by the contractor, demonstrating the adequacy of the proposed construction equipment. The evaluation of WEAP effectiveness for capacity predictions is difficult, as a large range of input parameters is possible and the results are greatly affected by the actual field conditions. Examination of the method through analyses making use of default values is probably the best avenue. Other evaluations, including WEAP analysis adjustments following dynamic measurements (e.g., matching energy), seem to be impractical in light of the other methods available and lead to questionable results regarding their quality and meaning (Rausche et al., 1997; Rausche, 2000). The WEAP analysis is evaluated in this study as a dynamic method for pile capacity prediction, using WEAP default input values and the pile's driving resistance at EOD compared to the static load test results. The evaluation of WEAP as a pile design method examining the analyzed stresses at the design stage to the measured stresses during construction leads to a strength factor (related to the allowed structural stresses in the pile) that is beyond the scope of the presented research.

2.5.2.3 Dynamic Equations

The chosen dynamic equations address the state of practice and reflect a range in equation type and performance. While the Engineering News-Record Equation (Wellington, 1892)

TABLE 7b Notations for combinations of correlations between soil parameters and standard penetration test results and their manner of application

Notations	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
limit ϕ below tip	40°				36°			
contributed zone for tip resistance	2B	11.5B	2B	11.5B	2B	11.5B	2B	11.5B
ϕ, if from SPT, is correlated by	Peck, Hanson and Thornburn		Schmertmann		Peck, Hanson and Thornburn		Schmertmann	
Su, if from SPT, is correlated by	Terzaghi and Peck							

Notations	(1h)	(2h)	(3h)	(4h)	(5h)	(6h)	(7h)	(8h)
limit ϕ below tip	40°				36°			
contributed zone for tip resistance	2B	11.5B	2B	11.5B	2B	11.5B	2B	11.5B
ϕ, if from SPT, is correlated by	Peck, Hanson and Thornburn		Schmertmann		Peck, Hanson and Thornburn		Schmertmann	
Su, if from SPT, is correlated by	Hara							

TABLE 8 Correlations of soil properties from CPT

Properties	From CPT	Reference (Kulhawy & Mayne, 1990)
ϕ	Robertson and Campanella: atan(0.1+0.38*log(q_c/σ'))	Figure 4.14 and Eq. 4.12
S_u (bar)	Theoretical: (q_c - σ_o) / Nk q_c and σ_o in bars.	Eq. 4.61
OCR for clay	Mayne: 0.29 q_c / σ'_o q_c and σ_o in bars.	Figure 3.10
Dr	Jamiolkowski: 68 log(q_{cn}) – 68 $q_{cn} = \frac{q'_c}{\sqrt{P_a \sigma'_0}}$ (dimensionless) $q'_c = q_c / K_q$ $K_q = 0.9 + Dr/300$ q_c and σ'_o in bars.	Figure 2.24 and Eq. 2.20

has proven to be unreliable through the years—as shown, for example, by Olsen and Flaate (1967)—it was founded on a solid theoretical basis and is still used in construction in about half of the states in the country. The equation's traditional formulation—as used, for example, in the Massachusetts State Building Code (Massachusetts, 1997)—includes an FS of 6, which needs to be recognized. The Gates equation (Gates, 1957), while empirical, was found to provide reasonable results (e.g., Olsen and Flaate, 1967; Long et al., 1998). The equation was further enhanced by Richard Cheney of the FHWA (FHWA, 1988) (see also Fragaszy et al. 1985), based on statistical correlations with static load tests and has the following format:

$$R_u = 1.75 \times \sqrt{E} \times \log 10N - 100 \tag{27}$$

where:

R_u = Ultimate capacity (tons)
E = Gross energy of pile hammer, ft-lb
Note: The equation includes an 80% efficiency factor on the rated energy, which is a value between 75% and 85% recommended by Gates (1957) for drop hammers and all other hammers, respectively.
N = Number of blows per inch

2.5.2.4 Dynamic Measurement: The Case Method

The Case method (Goble et al., 1970 and Rausche et al., 1975) is often used in field evaluations, as it is built into Pile Dynamics Inc.'s Pile Driving Analyzer (PDA), the most commonly used system for obtaining dynamic measurements during pile driving in the United States. The method is based on simplified pile and soil behavior assumptions (free end and plastic soil), resulting in a closed form solution related to the impact and its reflection from the tip. With the years, at least five different variations of the method have evolved (GRL, 1999). The Case method utilizes a damping coefficient (J_c) that is assumed to be associated with soil type. The influence of this factor on predicted static capacity depends on the stress wave reflected from the pile's tip, hence on the driving resistance. The Case-damping coefficient was inves-

TABLE 9 Dynamic methods for evaluating pile capacity: advantages, disadvantages, and comments

Category	Method	Advantages	Disadvantages	Comment
Design Stage	WEAP (Smith, 1960, Goble et al., 1976)	- Equipment Match - Drivability Study - Structural Stresses	- Non unique Analysis - Performance sensitive to field conditions	- Required for Construction - Required Evaluation for capacity predictions
Dynamic Equations	ENR (Wellington, 1892)	- Sound Principles - Common use	- Unreliable	- Needs to be examined without a built in FS.
	Gates (Gates, 1957)	- Empirical - Common use	- Depends on original database	- Found to be more reliable than other equations
	FHWA version of Gates Eqn. (FHWA, 1988)	- Correction based on additional data	- Depends on database	- Was found to be reliable
Dynamic Measurements	Signal Matching (e.g. CAPWAP) (Goble et al., 1970)	- Solid principle of matching calculations to measurements by imposing msd. B.C.	- Stationary soil forces - Expensive - Requires time	- Office Method - Found reliable at BOR
	Case Method (Goble et al., 1970, Rausche et al., 1975)	- Simplified Analysis - Field Method	- Requires local calibration - Presumed dependency of soil conditions found baseless	- Was found reliable with local calibration - How to obtain national or international calibration?
	Energy Approach (Paikowsky, 1982, Paikowsky et al., 1994)	- Simplified Analysis - Field Method	- Shows long-term capacity which may not be present at EOD	- Ideal for construction

NOTES: ENR = Engineering News Record; FS = Factor of Safety; BOR = Beginning of Restrike; EOD = End of Driving.

tigated through a back calculation (to match the measured static capacity). The results (see section 2.5.3.2.) suggest that there is no correlation between the soil type and the Case-damping coefficient. The recommended practice is to use the Case method based on a specific site/area calibration (GRL, 1999). This approach, in conjunction with the application of the method for maximum resistance (RMX), has proven worthwhile. Accumulated experience on extensive jobs in the Boston area (e.g., Geosciences Testing and Research, Inc. 1997, 1998) has demonstrated the effectiveness of the Case method, when calibrated. A statistical examination of local calibration was performed in Florida by McVay et al. (2000). The results of this analysis suggest that for 48 case histories, the ratio of the static pile capacity to the Case method prediction at EOD was 1.344 ± 0.443 (mean ± 1 SD).

As no generic conditions exist for the use of the Case method, international or national calibrations are unrealistic. Because the projection of local calibration (based on good experience and practice) beyond the geographical location may be unwise or unsafe, the Case method was excluded from the dynamic analyses examined for this project.

2.5.2.5 *Dynamic Measurement: The Energy Approach*

The Energy Approach uses basic energy relations in conjunction with dynamic measurements to determine pile capacity. The concept was first presented by Paikowsky (1982) and was examined on a limited scale by Paikowsky and Chernauskas (1992). Extensive studies of the Energy Approach method were carried out by Paikowsky et al. (1994) and Paikowsky and LaBelle (1994). The underlying assumption of this approach is the balance of energy between the total energy delivered to the pile and the work done by the pile/soil system. The basic Energy Approach equation is

$$R_u = \frac{E_{max}}{Set + \frac{(D_{max} - Set)}{2}} \tag{28}$$

where R_u = maximum pile resistance, E_{max} = measured maximum energy delivered to the pile, D_{max} = measured maximum pile top displacement, and Set = permanent displacement of the pile at the end of the analyzed blow, or 1/measured blow count. For further details regarding the Energy Approach method see Paikowsky et al. (1994) and Paikowsky (1995).

2.5.2.6 *Dynamic Measurement: The Signal Matching Techniques*

The signal matching technique is often referred to as post-driving analysis or the office method. With the availability of faster, portable computers, it became reasonably simple to conduct the analysis in the field, although the field method analyses cannot be carried out for each blow during driving. The response of the modeled pile-soil system (e.g., force at the pile top) under a given boundary condition (e.g., measured velocity at the pile top) is compared to the measured response (force measured). The modeled pile-soil system or, more accurately, the modeled soil that brings about the best match (visual graphical match) between the calculated and measured responses, is assumed to represent the actual soil resistance. The static component of that resistance is assumed to be the pile's capacity and reflects that time of driving. The signal matching procedure was first suggested by Goble et al. (1970), utilizing the computer program CAPWAP. Others developed similar analyses, (e.g., Paikowsky, 1982; Paikowsky and Whitman, 1990) utilizing the computer code TEPWAP. The TNO program was developed by Middendrop and van Weel (1986), which led to improvements and to the CAPWAPC program, which is used to date.

2.5.3 The Controlling Parameters

2.5.3.1 *Overview*

Preliminary examination of the parameters controlling the performance of the dynamic analyses was carried out prior to a final detailed evaluation of these methods, leading to the calculation of appropriate resistance factors. Such examination influenced the subcategorization of the dynamic methods (according to the important controlling parameters), hence directing the user to utilize the appropriate resistance factor according to the relevant conditions of the employed method. For example, if soil type is a controlling factor and the accuracy of the signal matching method is largely affected by soil type, evaluation of the method for different soil types will result in the development of different resistance factors depending on the soil type. Conversely, if soil type does not control the accuracy of the specific dynamic method, categorization based on soil type is neither desired nor pursued.

The following sections outline the logic used for the preliminary examination of the controlling parameters, the analyses, and the results. The rationale presented in this section follows previous studies by Paikowsky et al. (1994), Paikowsky (1995), Paikowsky et al. (1995), and Paikowsky and Chernauskas (1996). Paikowsky and Stenersen (2000, 2001) present more detailed results related to the dynamic analyses of this study and are provided in Appendix B.

The evaluation of static capacity through data derived from pile driving is based on the concept that the driving operation induces failure in the pile-soil system, (i.e., a very fast load test is carried out under each blow). Dynamic analyses encounter three fundamental difficulties: (1) correct formulation of the penetration process (e.g., soil motion, soil plugging etc.), (2) separation of the static resistance out of the total resistance overcome during penetration, and (3) time dependent pile capacity (Paikowsky, 1995). The parameters controlling the accuracy of the dynamic predictions reflect, therefore, the ability of each method to address the above difficulties.

Based on the concept of a pile loading to failure under each blow, it has traditionally been assumed that during high driving resistance (i.e., refusal) there is not sufficient pile penetration to mobilize the full pile capacity (Chellis, 1961). Therefore the dynamic methods are deficient under high driving resistance, categorized as equal or above 12BPI (Blows Per Inch) or approximately 5BPcm (Blows Per cm) (Massachusetts Highway Department, 1988).

Soil type is also believed to have a major effect on the dynamic analyses because soil damping parameters are commonly employed to represent viscous resistance in the modeling of the soil's dynamic behavior. This viscosity is assumed to be soil type dependent and associated with intrinsic soil properties. High viscosity values are expected for cohesive soils and low viscosity values are expected, therefore, for cohesionless soils. Naturally, under a given velocity, high viscous values are associated with higher dynamic resistance and logically should prove more difficult to accurately define the static resistance.

The effect of time is well recognized but poorly quantified. With time, piles undergo a decrease or increase of capacity, known as relaxation and set-up, respectively. While the resistance during driving and its static component represent the conditions encountered during penetration, the major interest remains the long-term ability of the pile to carry load during its service life. The examination of the dynamic-method predictions with static load tests (often carried out long after the driving) therefore remains valid. The predictions can be assessed in relation to the time at which the data have been obtained (i.e., EOD or BOR).

The following sections provide a short summary of the process in which the importance of each of the above assumed controlling parameters was examined. The results are used to evaluate additional possible controlling factors, laying down the framework for the detailed evaluation of the dynamic methods and the resulting resistance factors. More details are provided by Paikowsky and Stenersen in Appendix B.

2.5.3.2 *The Effect of Soil Type*

The effect of soil type was examined in two ways: (1) the correlation between the parameters assumed to be soil type dependent and soil type, i.e., damping parameters; and (2) the accuracy of the predictive methods relative to the soil type.

Figures 8 and 9 present the relationship between soil type and Smith-damping parameters (Smith, 1960) used in approximately 370 CAPWAP analyses from PD/LT2000 for the tip and side pile resistances, respectively. Figure 10 presents the back-calculated Case-damping coefficient required to obtain a match between the predicted capacity and the measured static capacity for 290 case histories from the PD/LT database (Paikowsky et al., 1994). All three figures clearly indicate that no unique relationship exists between soil type and damping parameters, suggesting that mechanisms other than the soil type control the value that should be used as a damping factor.

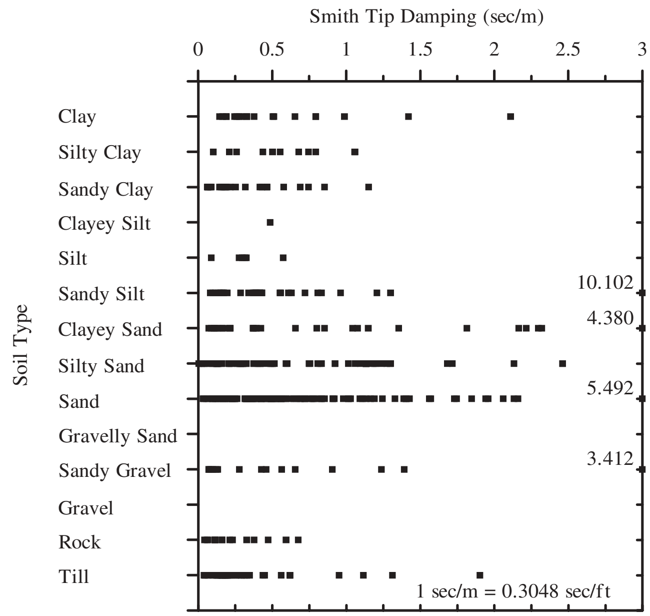


Figure 8. Soil type at the pile's tip versus Smith tip damping coefficients used in CAPWAP for 372 PD/LT2000 pile-cases.

A summary of the statistics obtained when examining the accuracy of the signal matching technique (specifically CAPWAP) based on soil type is presented in Table 10. The statistics shown are the mean and standard deviation of a normal

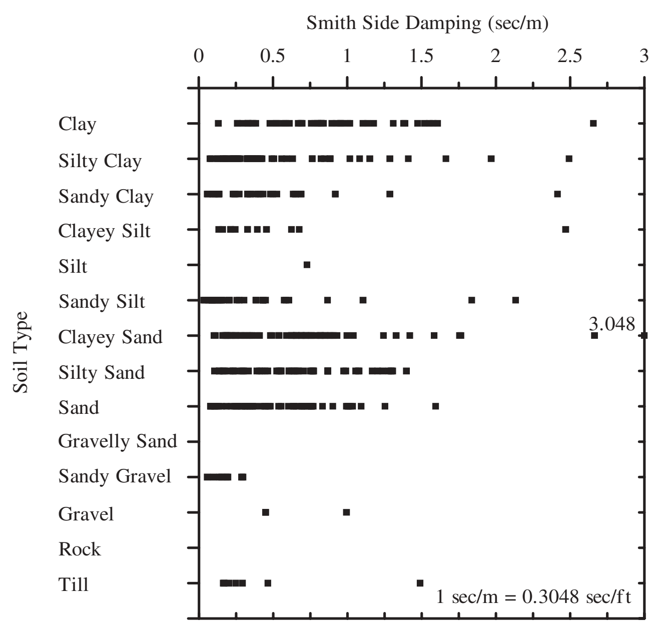


Figure 9. Soil type at the pile's side versus Smith side damping coefficients used in CAPWAP for 371 PD/LT2000 pile-cases.

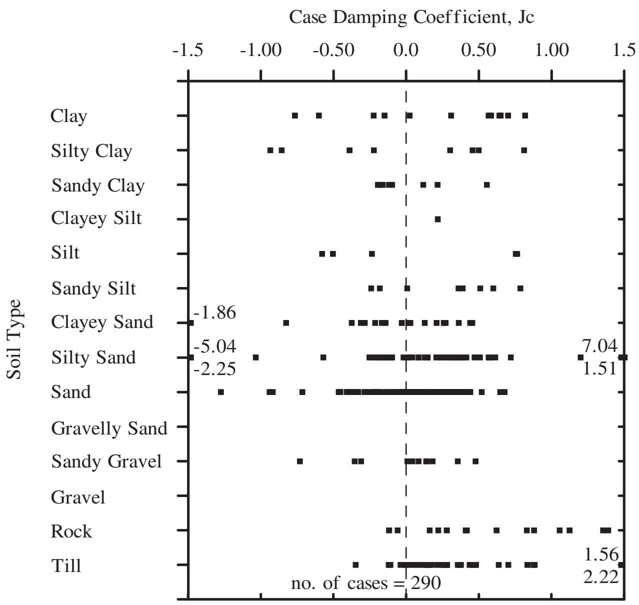


Figure 10. Soil type at the pile's tip versus back calculated Case-damping coefficient (Jc) based on static load test results for 290 PD/LT pile-cases (Paikowsky et al., 1994).

distribution function for the ratio of the pile's static capacity (based on Davisson's failure criterion) to the pile capacity obtained in the CAPWAP analysis. There are no significant differences between clay and till versus sand and silt that justify analysis categorization based on soil type. Although the case histories for piles found on rock provide different values, the numbers are based on a small subset of 15 pile case histories, compared to 100 and 265 pile case histories for the other soil type categories.

Table 10 provides further examination of time of driving and driving resistances as subsets of the soil type categorization. Two sets are examined based on the time of driving: EOD and last BOR, (i.e., in the case of multiple restrikes, only the last restrike is considered for the analysis). The results suggest that the time of driving significantly affects the performance of the CAPWAP prediction, regardless of soil type. The mean values for the BOR sets are closer to one, while the mean values for the EOD are closer to two. The COVs show values of 0.33 and 0.39 for BOR, while the EOD ratios are 0.55 and 0.85, indicating the existence of a substantial scatter. Again, the cases examined for piles in rock are not indicative and are excluded from being meaningful in respect to soil type effect.

Further evaluation of the records was carried out on the basis of driving resistance. The division between cases for which the driving resistance is smaller or greater than 5BPcm (5 blows per centimeter), examines the aforementioned notion of refusal and the expected accuracy of the dynamic methods. The results, shown in Table 10, suggest that analyses were less accurate and had larger scatter in cases for which the driving resistance was smaller than 5BPcm than when driving resistance was above 5BPcm. Though driving resistance seems to be an important factor, clear understanding of its influence on the accuracy of the dynamic methods calls for additional investigation, which is briefly presented in section 2.5.3.4.

In summary, while the performance of the signal matching analysis (CAPWAP) is not well correlated to soil type, other factors associated with soil type may be important (e.g., low driving resistance in soft cohesive soils or gain of capacity with time); but soil type itself does not appear to be important. The data presented in Table 10 suggests that time of driving must be considered and driving resistance needs to be further examined.

2.5.3.3 The Effect of Time on Tested Capacity

Penetration of piles into fine-grained soils causes compression and disturbance, resulting in soil strength during driving

TABLE 10 Statistical parameters of the ratio between static capacity (Davisson's Criterion) and signal matching analysis (CAPWAP) categorized according to soil type, time of driving and driving resistance

	Clay & Till				Sand & Silt				Rock			
Mean	1.352				1.517				0.930			
Standard Deviation	0.723				1.085				0.172			
Number of Cases	100				265				15			
Time of Driving	EOD		BOR(last)		EOD		BOR(last)		EOD		BOR(last)	
Mean	1.634		1.133		2.068		1.193		0.968		0.925	
Standard Deviation	0.899		0.444		1.765		0.391		0.132		0.203	
Number of Cases	45		40		77		116		7		7	
Blow Count (BPcm)	< 5	≥ 5	< 5	≥ 5	< 5	≥ 5	< 5	≥ 5	< 5	≥ 5	< 5	≥ 5
Mean	1.127	1.725	0.750	1.315	2.191	1.458	1.126	1.283	1.070	0.952	0.671	0.879
Standard Deviation	0.637	0.807	0.241	1.160	1.901	0.512	0.386	0.355	-----	0136	0.163	0.230
Number of Cases	35	35	11	10	64	13	74	40	1	6	3	3

NOTES: EOD = End of Driving; BOR(last) = Beginning of the last restrike; BPcm = Blows per centimeter that differs from its long-term strength, thus affecting pile capacity. Although factors such as thixotropy and aging contribute to this phenomenon, the migration of pore water is the most significant cause of capacity gain with time. Measurements carried out on a model (Paikowsky and Hart, 2000) and full-scale piles (Paikowsky and Hajduk, 1999, 2000) show that pore pressure at magnitudes similar to the total soil pressure creates in clays around the pile's shaft zones of about zero effective stress, resulting in almost a complete loss of frictional resistance. Paikowsky et al. (1995, 1996) examined the static and dynamic gain of capacity with time based on radial consolidation; a normalization process was followed, allowing for comparison between different pile sizes.

Table 11 presents a summary of parameters describing the pile capacity gain with time based on static and dynamic testing. The slope of the relation between the static capacity and the maximum static capacity (scale of 0 to 1) to the elapsed time after driving (logarithm scale) for a 152.4 mm radius (1 ft diameter) pile is denoted as C_{gt}. Similar relations for the ratio of dynamic capacity (with time) to the maximum static capacity result in a slope denoted by the parameter C_{gtd}. The time required for the standard pile to gain 75% of its maximum capacity is denoted as t_{75}. The time extrapolation for any desired pile size is achieved through the relationship of

$$t_{75\,(\text{pile})} = 4r^2\, t_{75\,(\text{table})} \tag{29}$$

For which r = the desired pile radius (ft.) or its equivalent for a pile of different shape.

The data in Table 11 show that while the rate of capacity gain is similar according to both analyses ($C_{gt} = 0.389$, $C_{gtd} = 0.348$), the associated time for achieving 75% of the maximum capacity (normalized for all piles to 304.8 mm diameter) is about 20 times greater when analyzed by static methods than when analyzed by dynamic methods. In other words, dynamic testing and analyses (namely CAPWAP), while following the physical behavior of capacity gain, exhibit this gain much faster than the actual gain monitored by the static load test results. The ramifications of these conclusions are that (1) actual gain of capacity is much slower than that exhibited by the dynamic methods, (2) scheduling of construction or testing based on capacity gain should consider the reason for time evaluation (i.e., actual loading in construction or dynamic testing as part of quality control), and (3) at present, the dynamic methods evaluation should concentrate on the long-term pile capacity.

2.5.3.4 The Effect of Soil Motion

Overview. Paikowsky and Chernauskas (1996) show that the stationary soil assumption, under which the soil/pile interaction models were developed, does not reflect the physical phenomenon that occurs during pile driving. Pseudo-viscous damping serves as a mechanism to absorb energy; but, as it does not reflect the actual phenomenon, it cannot be correlated to physical properties (e.g., soil type) or time of driving. If the motion of the displaced soil is a major factor contributing to energy loss during driving, a substantial portion of the dynamic resistance should be a function of two parameters: (1) acceleration of the displaced soil (especially at the tip) that can be conveniently examined as a function of the driving resistance, and (2) mass/volume of the displaced soil that is a function of the pile geometry, namely, small vs. large displacement piles. A brief summary of the findings described by Paikowsky and Stenersen regarding the above two factors follows. Further details of their research are provided in Appendix B.

Soil Acceleration/Driving Resistance. The energy loss through the work performed by the displaced soil mass at the tip is directly related to the acceleration of this mass. The detailed evaluation of the soil's motion at the tip is beyond the scope of the present research and is described by Hölscher (1995), Hölscher and Barends (1996), and Hajduk et al. (2000). The indirect evaluation of these accelerations can be performed through analysis of the driving resistance, which is the measure of the pile's final displacement under each hammer blow. With low driving resistance (easy driving), high acceleration and velocity (i.e., free-end analogy) are developed at the tip. In the case of high driving resistance (hard driving), there is small acceleration at the tip, resulting in little, if any, mobilization of the soil mass beyond a radiating elastic wave. The corresponding energy loss due to soil motion is, therefore, small.

TABLE 11 Summary of static-and-dynamic-based capacity gain with time parameters based on data sets (Paikowsky et al. 1996)

	Static Data Sets LTT and PUT/LTT		Dynamic Data Set PD/LTT		ALL DATA	
	C_{gt}	t_{75}*	C_{gtd}	t_{75}**	C_{gt}	t_{75}**
No. of Cases	15	5	7	6	22	11
Average for all piles in set.	0.389	385.0	0.348	21.3	0.376	186.6
Standard Deviation	0.119	226.3	0.068	7.9	0.106	237.9

Notes: *closed-ended pipe piles only; **t_{75} = time for a standard pile (0.3048m radius) to gain 75% of its maximum capacity; C_{gt} = rate of pile capacity gain with the logarithm of time

To evaluate the blow count that identifies the transition between easy and hard driving (high and low soil acceleration) the ratio between the static capacity and the CAPWAP prediction (K_{SW}) by blow count for all pile case histories in PD/LT2000 was determined, as presented in Figure 11a. Figure 11b presents the data separated into intervals of 8 BP10cm (2BPI), with the mean and standard deviation of each group graphed as a point and an error bar against the mid point blow count of the interval. For example, for driving resistance between 0 and 8BP10cm there were 42 case histories with a mean of 2.506 and a standard deviation of 2.217 plotted at the center of the interval, i.e., at 4BP10cm. The data presented in Figure 11b show that for the first two intervals (up to 16BP10cm) the predicted capacity was substantially lower than for all other intervals with a significantly higher scatter. After approximately 16 blows per 10cm, the mean and standard deviation of the individual intervals fall within the range of all case histories. The boundary of the dynamic method evaluation based on driving resistance was defined, therefore, as 16BP10cm (4BPI).

Displaced Soil/Pile Area Ratio. The volume of the displaced soil is identical to the volume of the penetrating pile, except when pile plugging takes place (Paikowsky and Whitman, 1990). The piles, therefore, can be classified as small (e.g., H and unplugged open pipe) and large (e.g., closed pipe and square concrete) displacement piles. Additional classification of open-pipe piles can be made according to a tip-area ratio similar to that used for soil samplers (Paikowsky et al., 1989).

As most soil displacement takes place at the tip area, the classification of piles can be better served by looking at the ratio between the pile's embedded surface area and the area of the pile tip (Paikowsky et al., 1994):

$$A_R = \frac{A_{skin}}{A_{tip}} = \frac{\text{Surface area in contact with soil}}{\text{Area of pile tip}} \qquad (30)$$

Using this ratio, a pile traditionally referred to as a "large displacement" pile can behave like a "small displacement pile" if it is driven deeply enough. A quantitative boundary of A_R = 350 between "small" and "large" displacement piles was proposed by Paikowsky et al. (1994).

Figure 12a presents the relationship between A_R and the ratio of the static capacity over CAPWAP prediction (K_{SW}) for all pile case histories in PD/LT2000. The data are separated into A_R intervals of 175, with the mean and standard deviation of each group graphed as a point and error bar at

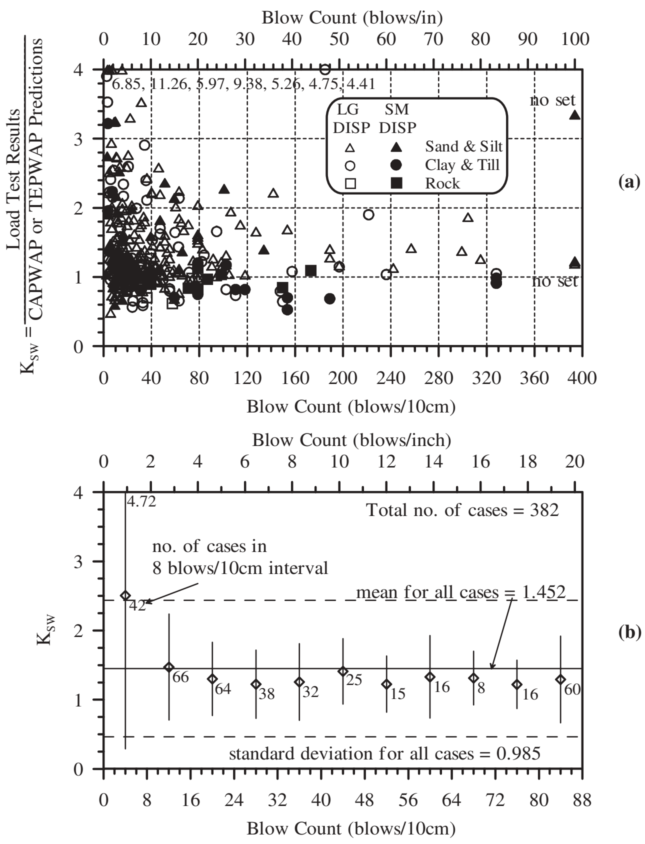


Figure 11. The ratio of static capacity to dynamic signal matching prediction, K_{SW} versus blow count for all pile-cases in PD/LT2000 (a) all data points, and (b) data grouped in intervals of 8 blows/10cm (2BPI).

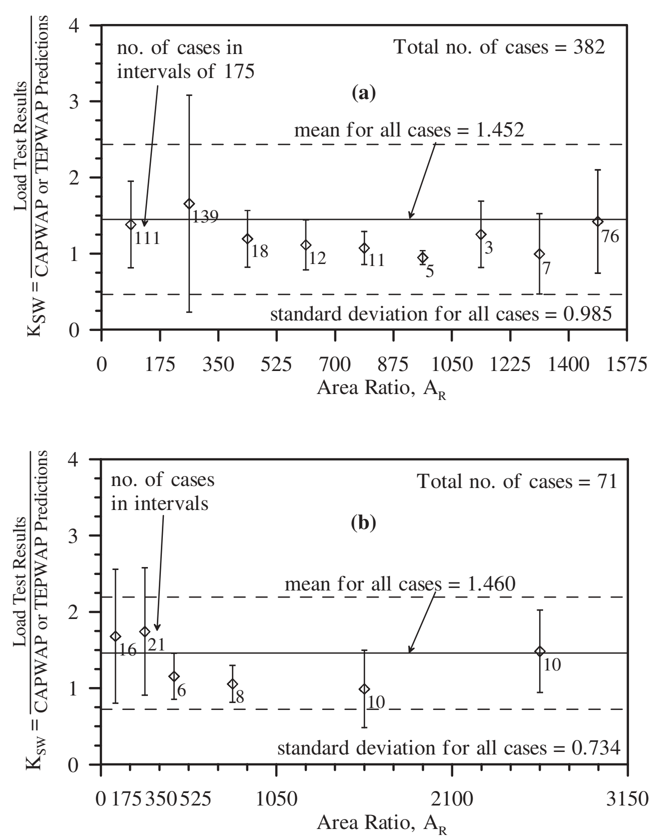


Figure 12. K_{SW} versus area ratio, (a) for all pile-cases in PD/LT2000 and (b) for 71 pile-cases with driving resistance exceeding 16 BP10cm (4BPI) at the EOD.

the midpoint A_R of the interval. For example, for the 139 piles with A_R between 175 and 350, the mean K_{SW}, 1.656, and the standard deviation, 1.425, are plotted at the center of the interval (i.e., A_R 262.5). Figure 12a suggests that piles with an A_R smaller than 350 present less accurate predictions and larger scatters compared to the mean and the scatter of all cases. Above an A_R of 350, the mean and standard deviation of the individual intervals fall within the range of all cases.

Because driving resistance may affect the data, in Figure 12a the influence of the area ratio was further examined for piles with a driving resistance greater than 16 BP10cm at EOD. Figure 12b presents the relationship between A_R and K_{SW} for 71 case histories answering to this criterion. These data suggest even when excluding the easy driving resistance effects, the accuracy of the dynamic predictions are still lower and have a larger scatter for piles with A_R smaller than 350. The boundary of $A_R = 350$ between small and large displacement piles was therefore confirmed, based on database PD/LT2000.

2.6 DRILLED SHAFTS— STATIC ANALYSIS METHODS

Based on the established state of practice in design (reviewed in section 2.1 and presented in Appendix A), the following analysis methods and correlations have been used for the static capacity evaluation of the drilled shaft database:

1. FHWA Method (Reese and O'Neill, 1988)—β method and α method were used for sand and clay respectively. For the undrained shear strength, Su, the SPT correlation given by Terzaghi and Peck (1967) was used.
2. R&W Method (Reese and Wright, 1977)—for sands while for sand and clay mix layers the α method was used for the clay.
3. C&K Method (Carter and Kulhawy, 1988)—for rock.
4. IGM Method (Intermediate Geomaterials) (O'Neill et al., 1996; O'Neill and Reese, 1999). The design assumed a smooth rock socket for skin friction and closed joints for end bearing.

Details of the analysis methods, the analyzed case histories, and the obtained results are summarized in Appendix C.

2.7 LEVEL OF TARGET RELIABILITY

2.7.1 Target Reliability and Probability of Failure

The utilization of LRFD requires the selection of a set of target reliability levels, which determine the probability of failure and, hence, the magnitude of the load and resistance factors (see section 1.3.1 and Figure 2). The probability of failure represents the probability for the condition at which the resistance multiplied by the resistance factors will be less than the load multiplied by the load factors. When fitting LRFD to ASD, the issue is less significant because, in practice, the factors are established to conform (often conservatively) to existing factors of safety. When calibrating for a database, however, the establishment of an acceptable probability of failure is cardinal, including the question of a new design versus the existing state of practice. An approximate relationship between probability of failure and target reliability for a lognormal distribution was presented by Rosenbleuth and Esteva (1972) and is commonly in use (e.g., Withiam et al., 1998):

$$p_f = 460\, e^{-4.3\beta} \tag{31}$$

Baecher (2001) shows, however, that this approximation is not very accurate below β of about 2.5; and Table 12 provides a comparison between the approximation and the "exact" numbers for different values of β that suggests significant errors, especially in the zone of interest for foundation design, (β = 2 to 3).

2.7.2 Concepts for Establishing Target Reliability

2.7.2.1 General Methods of Approach

Three accepted methods exist to determine probabilities of an event occurring: (1) historical data providing the results of frequent observations, (2) mathematical modeling derived from probability theory, and (3) quantification of expert systems (Benjamin and Cornell, 1970). Combination of the three, when possible, can lead to a practical tool in design (e.g., Zhang et al., 2002, for dam slope failure). Such knowledge does not exist for foundations, and the selection of target reliability levels is a difficult task as these values are not readily available

TABLE 12 Comparison between Rosenbleuth and Esteva approximation and series expansion labeled "Exact" of the probability of failure (pf) for different values of reliability index (β) (Baecher, 2001)

β	Rosenbleuth and Estevas' p_f	Exact p_f	Percent Error
2.0	8.4689E- 2	2.2750E-2	272.3%
2.5	9.8649E- 3	6.2097E-3	58.9%
3.0	1.1491E- 3	1.3500E-3	-14.9%
3.5	1.3385E- 4	2.3267E-4	-42.5%
4.0	1.5592E- 5	3.1686E-5	-50.8%
4.5	1.8162E- 6	3.4008E-6	-46.6%
5.0	2.1156E- 7	2.8711E-7	-26.3%
5.5	2.4643E- 8	1.9036E-8	29.5%
6.0	2.8705E- 9	9.9012E-10	189.9%

and need to be generated or selected (Payer et al., 1994). Target reliability levels vary from one application to another due to various factors, including implied reliability levels in current design practice, failure consequences, public and media sensitivity, types of users and owners, design life of a structure, and other political, economic, and societal factors. For a general view, see Whitman (1984) and Becker (1996). Two approaches to generating target reliability levels are used in general: (1) calibrated reliability levels that are implied in currently used codes, and (2) cost-benefit analysis.

The first approach is commonly used to develop reliability-based codified design, such as LRFD. The target reliability levels developed according to this approach are based on calibrated values of implied levels of uncertainty in a currently used design practice. The argument for using this approach is that a code documents an accepted practice, and, as such, can be used as a launching point for code revision and calibration. Any adjustments in the implied levels should be for the purpose of creating consistency in reliability among the resulting designs when using the reliability-based code. Using the same argument, it can be concluded that target reliability levels used in one industry might not be fully applicable to another industry.

Cost-benefit analysis, the second approach to generating target reliability levels, is used effectively in dealing with designs for which failures result only in economic losses and consequences. Since structural failures might result in human injury or loss of life, the use of this method might be very difficult because of its need for assigning a monetary value to human life. One way to avoid the need to measure the monetary value of human life is to assign probabilities of failure as a function of both, monitoring cost and loss of lives (see, e.g., Zhang et al., 2002).

2.7.2.2 Calibration

A number of efforts for the purpose of calibrating a new generation of structural design codes have resulted in the development of target reliability levels (i.e., safety indices, or β values). The general methodology for code calibration based on specific reliability theories, using second-moment reliability concepts, is outlined by Melchers (1987) and others. Melchers notes that frequently the information is insufficient for this determination and one must make a "semi-intuitive" judgment in selecting target reliability, β_t, values. While the specific reliabilities will be a function of the strength criteria needed for specific materials and load combinations within designated structures, it is useful to have an indication of the range of possible target reliability levels.

2.7.3 Target Reliability for Structures

Ellingwood et al. (1980) present ranges for reliability levels for metal structures, reinforced and prestressed concrete, heavy timber, and masonry structures, as well as discussions of issues that should be considered when making the calibrations. Table 13 provides typical values for β_T based on values provided by Ellingwood et al. (1980). The target reliability levels shown in Table 14 are used by Ellingwood and Galambos (1982) to demonstrate the development of partial safety factors.

Moses and Verma (1987) suggested target reliability levels in calibrating bridge codes (i.e., AASHTO Specifications). Assuming that bridge spans of less than 100 ft are most common, a β_T of 2.5 to 2.7 is suggested for redundant bridges, and a β_T of 3.5 for nonredundant bridges.

Wirsching (1984) estimated the safety index, or β values, implied by the API specifications (American Petroleum Institute, 1989) for fixed offshore structures in fatigue of tubular welded joints to be 2.5. He reported that this value is on the low end, because of the reference wave values.

Madsen et al. (1986) discuss target reliability levels that were used by the National Building Code of Canada (National Research Council of Canada, 1977) for hot-rolled steel structures. The values selected were $\beta_T = 4.00$ for yielding in tension and flexure, $\beta_T = 4.75$ for compression and buckling

TABLE 13 Target reliability levels by structural type [based on Ellingwood et al. (1980)]

Structural Type	Target Reliability Level (β_t)
Metal structures for buildings (dead, live, and snow loads)	3
Metal structures for buildings (dead, live, and wind loads)	2.5
Metal structures for buildings (dead, live, snow, and earthquake loads)	1.75
Metal connections for buildings (dead, live, and snow loads)	4 to 4.5
Reinforced concrete for buildings (dead, live, and snow loads)	
- ductile failure	3
- brittle failure	3.5

Note: The β_t values are for structural members designed for 50 years of service.

TABLE 14 Target reliability, levels for members, used by Ellingwood and Galambos (1982)

Member, Limit State	Target Reliability Level (β_t)
Structural Steel	
Tension member, yield	3.0
Beams in flexure	2.5
Beams in shear	3.0
Column, intermediate slenderness	3.5
Reinforced Concrete	
Beam in flexure	3.0
Beam in shear	3.0
Tied column, compressive failure	3.5
Masonry, unreinforced	
Wall in compression, inspected	5.0
Wall in compression, uninspected	7.5

Note: The β_t values are for structural members designed for 50 years of service.

failure, and $\beta_T = 4.25$ for shear failures. These values are higher than those in Tables 13 and 14 because they reflect different environmental loading conditions and, possibly, different design life. The Canadian Standard Association presented the following target failure probabilities for developing design criteria for offshore installation in Canadian waters (Mansour et al., 1994): 10^{-5} per year for failures that would result in great loss of life or have a high potential for environmental damage; and 10^{-3} per year for failures that result in small risk to life or a low potential for environmental damage. (It is important to note that no direct relationship exists between general probability of failure and annual probability of failure.)

Madsen et al. (1986) also discuss target reliability levels that were used by the Nordic Committee on Building Regulations (1978). Target reliability values were selected depending on the failure consequences of a building: $\beta_T = 3.1$ for less serious failure consequences, $\beta_T = 5.2$ for very serious failure consequences, and $\beta_T = 4.3$ for common cases.

2.7.4 Geotechnical Perspective

The review provided in section 2.7.3 suggests that typical target reliability for members and structures relevant to bridge construction varies between 1.75 and 3.0, with a target reliability of 2.5 to 2.7 for relevant bridges.

Barker et al. (1991, p. A-51) state the following regarding target reliability index for driven piles:

> Meyerhof (1970) showed that the probability of failure of foundations should be between 10^{-3} and 10^{-4}, which corresponds to values of β between 3 and 3.6. The reliability index of offshore piles reported by Wu, et al. (1989) is between 2 and 3. They calculated that the reliability index for pile systems is somewhat higher and is approximately 4.0, corresponding to a lifetime probability of failure of 0.00005. Tang et al. (1990) reported that offshore piles have a reliability index ranging from 1.4 to 3.0.
>
> Reliability indices for driven piles are summarized in Table 5.4 [Table 15 of this report]. Values of β between 1.5 and 2.8 are generally obtained for the lognormal procedure. Thus a target value of β between 2.5 to 3 may be appropriate. However, piles are usually used in groups. Failure of one pile does not necessarily imply that the pile group will fail. Because of this redundancy in pile groups, it is felt that the target reliability index for driven piles can be reduced from 2.5 to 3.0 to a value between 2.0 and 2.5.

TABLE 15 Reliability indices for driven piles (Barker et al., 1991)

Dead to Live Load Ratio	Reliability Index, β	
	Lognormal	Advanced
1.00	1.6 – 2.8	1.6 – 3.0
3.69	1.7 – 3.1	1.8 – 3.3

Zhang et al. (2001) used a first order reliability method to evaluate the reliability of axially loaded pile groups designed using the traditional concept of group efficiency. Group effects and system effects were identified as the major causes of the significantly greater observed reliability of pile foundations compared to the calculated reliability of single piles. Group effect relates to the combined action of any number of piles vs. a single pile. A system effect is the contribution of the superstructure stiffness to the load distribution and resistance.

The calculated probability of failure of pile groups was found to be 1 to 4 orders of magnitude smaller than that of single piles, depending on the significance of system effects (changing the system bias factor λ_s from 1 to 2). Based on their study, Zhang et al. (2001) state that the target reliability index, β_T, for achieving a specified reliability level should differ for an isolated single pile (β), an isolated pile group (β_{TG}), and a pile system (β_{TS}). They give the following recommendations based on their research:

1. A β_{TG} value of 3.0 requires a β of 2.0 to 2.8 if no system effects are considered.
2. A β_{TG} value of 3.0 requires a β of 1.7 to 2.5 if a system effect factor of 1.5 is considered.

Additional aspect to the increased reliability of deep foundations can be obtained from the limited data available regarding the loads, which actually arrive at the piles during their service. Tang et al. (1994) followed the response of drilled shafts during construction loading and found that, while 44% to 67% of the design load was measured at the pile's top, only 6% to 13% of the design load arrived at the tip in the rock socket. In the design of drilled shafts the friction or the end bearing are often being neglected, especially in rock sockets. This practice and the observed values suggest that piles are often underutilized (over conservative), a fact contributing to the reliability of pile foundations, which rarely fail. These facts, while recognized, cannot be considered when assigning a target reliability value until more data are available and relevant load factors can be directly developed for foundations.

2.7.5 Recommended Target Reliability

2.7.5.1 General Range for Single Piles and Pile Groups

Based on the above review and the data presented, it seems reasonable to establish the target reliability between 2.0 and 2.5 for pile groups and as high as 3.0 for single piles.

It is clear from the review that, while the redundancy of pile groups serves as the major reason for the decrease in target reliability, no logical distinction was made (when choosing target reliability) between the target reliability of single piles and pile groups. One can evaluate the performance of the piles on the basis of their "redundancy." A nonredundant member is one for which failure will directly affect the element carried by it (i.e., the column) with limited or no ability of other foundations supporting the same element to mitigate the effect of the failure of the member. Referring to Figure 13, one can intuitively see that, as three points define a plane, a failure of any deep foundation element in such a configuration cannot be mitigated by the others. Though details of the foundation scheme are important—see, e.g., *Foundation Design Standards in the World* (Japanese Geotechnical Society, 1998)—one can distinguish between a 5-member scheme (clearly redundant) and a 3- or fewer member scheme (nonredundant) for the purpose of establishing a target reliability.

The evaluation of the resistance factors in the present study was originally carried out by using reliability indices of 2.0, 2.5, and 3.0 associated with p_f = 2.28%, 0.62%, and 0.14%, respectively. This approach provided a reasonable range of values to investigate before the final target reliability values were set.

2.7.5.2 Recommended Concept and Targets

Based on the review of the state of the art, the survey of common practice, and the evaluation of the above values, the following reliability indices and probability of failure were developed and are recommended in conjunction with methods for capacity evaluation of single piles (see Figure 13):

1. For redundant piles, defined as 5 or more piles per pile cap, the recommended probability of failure is $p_f = 1\%$, corresponding to a target reliability index of $\beta = 2.33$.
2. For nonredundant piles, defined as 4 or fewer piles per pile cap, the recommended probability of failure is $p_f = 0.1\%$, corresponding to a reliability index of $\beta = 3.00$.

2.8 INVESTIGATION OF THE RESISTANCE FACTORS

2.8.1 Initial Resistance Factors Calculations

The factors were evaluated using FORM (First Order Reliability Method) with dead load (DL) to live load (LL) ratios ranging from 1 to 4. The results for a bias of one and a coefficient of variation of 0.4 and target reliability values of 2.0, 2.5, and 3.0 presented in Figure 14, suggest very little sensitivity of the resistance factors to the DL to LL ratio. A similar trend was observed using DL to LL ratio of 10. The large dead-to-live-load ratios represent conditions of bridge construction, typically associated with very long bridge spans. The relatively small influence of the dead-to-live-load ratio on the calculated resistance factors suggests that (1) the use of a DL to LL ratio of 2 or 2.5 as a typical value is reasonable, and (2) the obtained factors are, by and large, applicable for long span bridges.

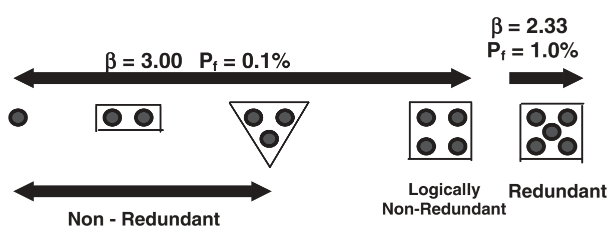


Figure 13. Redundant vs. non-redundant pile support and the current research recommendations of target reliability.

2.8.2 Parameter Study—The Limited Meaning of the Resistance Factor Value

The use of FORM requires an iterative process and hence a parametric study more easily obtained by using the FOSM relationships, assuming the results of both are within a close range (to be demonstrated in section 3.2.2). Figure 15 presents such relations using Equation 10, the chosen load distribution parameters (Equations 25 and 26), DL to LL ratio of 2.5 and a target reliability $\beta = 2.33$ (see section 2.7.5.2). The obtained relationship shows that a perfect prediction ($\lambda = 1$, COV = 0) would result with a resistance factor of ($\phi = 0.80$. With a prediction method for which the bias is one but the distribution is greater (COV > 0), the resistance factor would sharply decrease so that for COV = 0.4 the resistance factor would reduce to $\phi = 0.44$. The influence of the bias of the method (λ, or mean ratio of measured over predicted) on the resistance factor is equally important. As seen in the figure, an under predictive method ($\lambda > 1$) has a "built in" safety and hence a higher resistance factor is used in order to achieve the same target reliability as would be obtained by using a method which predicts, on average, more accurately ($\lambda \approx 1$). For example, for methods having the same distribution (COV = 0.4), an underpredictive method with a bias of λ = 1.5 would result in a resistance factor $\phi = 0.67$, whereas a method with a bias $\lambda = 1.0$ would result in $\phi = 0.44$. Although

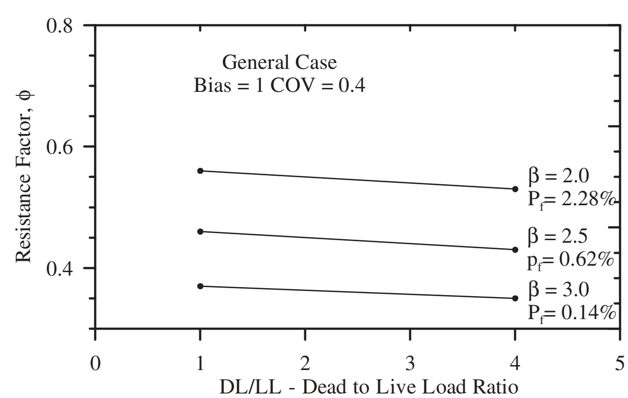


Figure 14. Calculated resistance factors for a general case showing the influence of the dead-to-live-load ratio.

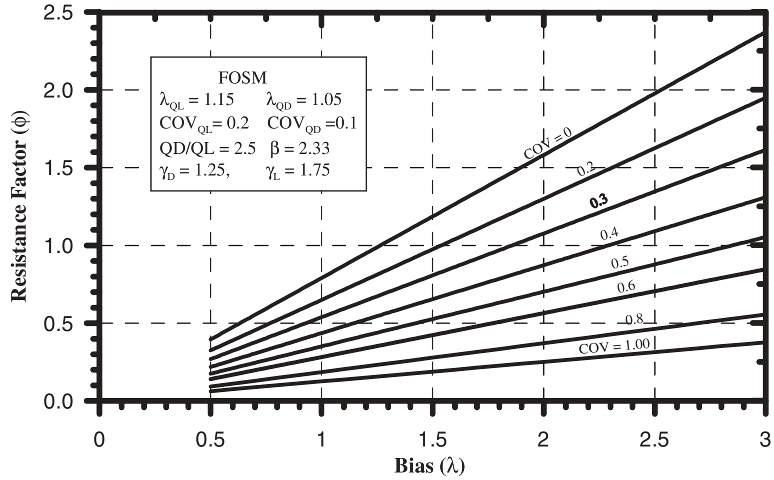


Figure 15. Calculated resistance factors as a function of the bias and COV for the chosen load distributions and DD/LL ratio of 2.5.

both methods predict the same way (i.e., have the same distribution), the method, which predicts more accurately (lower bias) will result in having a resistance factor lower than the underpredictive method. The judgment of the methods' economic value ("efficiency") on the basis of the resistance value is therefore misleading. The same argument can be made regarding the misleading absolute values of the factor of safety disregarding the bias. The FS values in Table 1 seem to be high (and not attractive economically) for the static analyses compared to the dynamic prediction methods. Again these values are of limited meaning if the bias of the method is not considered. For example, if the bias of the static methods (to be discussed further in Chapter 3, section 3.5.2) is lower than 1 (overprediction), while the bias of the dynamic methods is greater than one (underprediction), the methods may have practically a similar "actual" FS (and hence economical viability).

2.8.3 The Design Methods' Efficiency

The values of the resistance factors alone (or the factors of safety) do not provide a measure for evaluating the efficiency of the design methods, as previously discussed. Such efficiency can be evaluated through the bias factor, and its COV, or the ratio of the resistance factor to the bias factor, i.e., ϕ/λ, as proposed by McVay et al. (2000). Figure 16 illustrates the meaning of the efficiency factor showing that the ratio of ϕ/λ is systematically higher for methods which predict more accurately regardless of the bias. The value of the

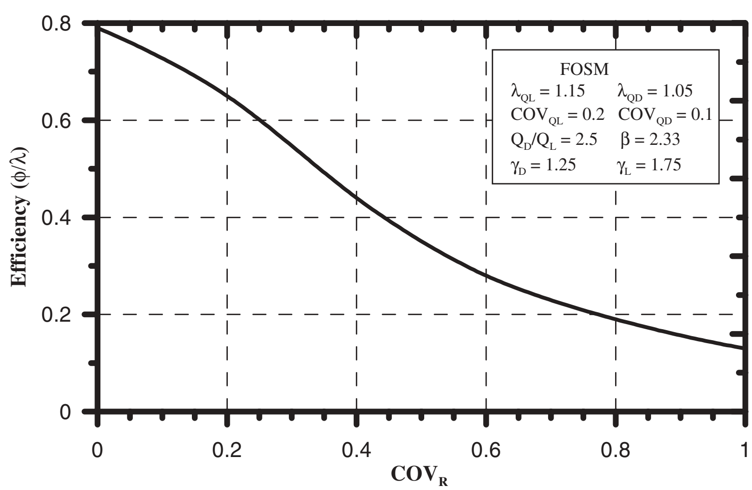


Figure 16. Illustration of the efficiency factor as a measure of the effectiveness of a design method when using resistance factors.

efficiency factor remains constant for all bias combinations for a given COV, leading to higher values for methods with a lower COV. Using the example given in section 2.8.2, a method with COV = 0.4, $\lambda = 1.0$, and $\phi = 0.44$ will result in $\phi/\lambda = 0.44$; a second method with COV = 0.4, $\lambda = 1.5$, and $\phi = 0.67$ will result in the same $\phi/\lambda = 0.44$. Thus, although one method presents a resistance factor of 0.67 and the other of 0.44, both methods have identical efficiency and should result in identical design; hence they have the same economic value. The efficiency of a given capacity prediction method can, therefore, be improved only through a reduction in its variability (COV); alternatively, design methods need to be chosen based on their COV.

This measure of efficiency needs to accompany prescribed resistance factors in order to avoid a misconception of the existence of a correlation between the economy of a design method and high resistance factors when compared to others. Similarly, such misconceptions exist between the economic value of a method and the lower level of a factor of safety, where a mean factor of safety (defined as FS x bias) represents the economic value of the method (the lower the better), as proposed by Paikowsky et al. (1994).

CHAPTER 3

INTERPRETATION, APPRAISAL, AND APPLICATIONS

3.1 ANALYSIS RESULTS AND RESISTANCE FACTORS

3.1.1 Driven Piles—Static Analysis

Table 16 presents a summary of the results obtained from the analyses used for static capacity evaluation of driven piles, compared with the nominal resistance based on Davisson's failure criterion. The information is grouped by soil and pile type and design method. The table includes statistical parameters and resistance factors for a range of reliability index values, and a ratio of DL to LL of 2.0.

The data leading to Table 16 were statistically analyzed to remove outliers (i.e., extreme cases; Section 3.5 provides a discussion of this process); and the table includes only those cases within ±2 standard deviations of the mean. As can be seen, subcategorization based on pile or soil type may result in subsets too small for reasonable statistical analysis. On the other hand, many of the subsets have similar statistics and resistance factors and hence can be combined. It is important to note that many common design methods for all piles in all soils overpredict the actual (i.e., measured) pile-capacity. This explains the traditional need for high factors of safety for static design (e.g., see Table 1).

A more complete picture of the performance of a method is obtained by plotting the histogram of observed to predicted capacities and overlaying the best-fitting normal and lognormal PDFs. Figures 17 through 26 present such plots for the selected cases of static analyses of driven piles. The figures are arranged in order from the most inclusive logical case, as the data permit, to subsets of the same category. For example, Figure 17 presents the performance of the α-API method for all pile types (52 cases of H, concrete and pipe piles) in clay. Figures 18 and 19 present the performance of the method for subsets of concrete (36 cases) and H piles (16 cases), respectively. Additional graphical presentations of the data are included in Appendix C.

3.1.2 Driven Piles—Dynamic Analysis

3.1.2.1 The Analyzed Cases

Time of driving, driving resistance, and area ratio proved to be controlling parameters for the dynamic methods (section 2.5.3). The PD/LT 2000 database was first separated into design and construction categories. The dynamic methods used in construction were subdivided between methods that use dynamic measurements and those that do not. These, in turn, were subcategorized according to the controlling parameters. Figure 27 presents the analyzed subsets, the number of case histories in each set, and the mean and standard deviation for each.

WEAP is utilized in the design stage. The analysis (not included in this research) needs to be carried out for driving stress evaluation, leading to a load factor. The use of the method for the evaluation of pile capacity was examined through the comparison of WEAP results for default input values and the blow count at the EOD with static load test results. The data presented were provided by GRL Inc. (Hannigan et al., 1996).

For the construction category, the dynamic analyses methods without dynamic measurements are the ENR, Gates, and FHWA version of Gates. The methods with dynamic measurements are CAPWAP and the Energy Approach. The dynamic methods are broken down into subsets based on time of driving, driving resistance, and area ratios. Judgment and statistical guidelines were used for the inclusion or exclusion of cases. For example, extreme CAPWAP underpredictions (beyond 2 standard deviations) were observed at EOD at one site. All the case histories on that site included easy driving and large area ratios; if included in the general population of the data, the EOD statistics would have become 1.861 ± 1.483 (mean ± 1 S.D.). This site is included only in the subcategory of blow count < 16 BP10cm and A_R < 350.

3.1.2.2 The Critical Categories

The outcomes of the statistical analyses presented in Figure 27 allow the identification of critical categories that require calibration and development into resistance factors. For example, the critical CAPWAP cases include (1) all data, (2) EOD, (3) BOR, and (4) the worst combination of soil motion effect (Blow count < 16 BP10cm and A_R < 350). Table 17 presents a summary of the major categories of the dynamic methods identified from Figure 27 as those that require calibration for a resistance factor.

TABLE 16 The performance of the driven piles' static analysis methods—statistical summary and resistance factors for data using mean ± 2 SD

Soil Type	Pile Type	N	Design Method[(1)]	Details of Method[(2)] Application	Mean	Stand. Dev.	COV	Resistance Factors for a Given Reliability Index β		
								2.00	2.50	3.00
Clay	H-Piles	4	β-Method	11.5 B; T&P(2)	0.61	0.37	0.61	0.23	0.18	0.13
		16	λ-Method	11.5B; T&P(2) 2B; T&P(5)	0.74	0.29	0.39	0.43	0.35	0.29
		17	α-Tomlinson	2B; T&P(2)	0.82	0.33	0.40	0.46	0.38	0.31
		16	α-API	2B; T&P(5)	0.90	0.37	0.41	0.50	0.41	0.33
		8	SPT-97 mob		1.04	0.43	0.41	0.57	0.47	0.38
	Concrete Piles	18	λ-Method	2B; Hara (5h)	0.76	0.22	0.29	0.53	0.45	0.38
		17	α-API	2B; Hara (5h)	0.81	0.21	0.26	0.60	0.52	0.44
		8	β-Method	2B; Hara (5h)	0.81	0.41	0.51	0.37	0.30	0.23
		18	α-Tomlinson	2B; Hara (5h)	0.87	0.42	0.48	0.42	0.34	0.26
	Pipe Piles	18	α-Tomlinson	2B; T&P (1)	0.64	0.32	0.50	0.30	0.24	0.19
		19	α-API	2B; T&P (1)	0.79	0.43	0.54	0.34	0.27	0.20
		12	β-Method	2B; T&P (1)	0.45	0.27	0.60	0.17	0.13	0.10
		19	λ-Method	2B; T&P (1)	0.67	0.37	0.55	0.28	0.22	0.17
		12	SPT-97 mob	2B; T&P (1)	0.39	0.24	0.62	0.15	0.11	0.08
Sand	H-Piles	19	Nordlund	36; 11.5B,P(6)	0.94	0.38	0.40	0.53	0.43	0.35
		18	Meyerhof		0.81	0.31	0.38	0.47	0.39	0.32
		19	β-Method	36; 2B; P(5)	0.78	0.40	0.51	0.36	0.28	0.22
		18	SPT-97 mob		1.35	0.58	0.43	0.72	0.59	0.47
	Concrete Piles	36	Nordlund	36: 11.5B; P(6)	1.02	0.49	0.48	0.50	0.40	0.31
		35	β-Method	36; 2B; P(5)	1.10	0.48	0.44	0.58	0.47	0.38
		36	Meyerhof		0.61	0.37	0.61	0.23	0.18	0.13
		36	SPT-97 mob		1.21	0.57	0.47	0.60	0.48	0.38
	Pipe Piles	19	Nordlund	36; 2B P(5)	1.48	0.77	0.52	0.67	0.52	0.41
		20	β-Method	36; 2B P(5)	1.18	0.73	0.62	0.44	0.33	0.25
		20	Meyerhof		0.94	0.55	0.59	0.37	0.29	0.22
		19	SPT-97 mob		1.58	0.82	0.52	0.71	0.56	0.44
Mixed Soils	H-Piles	20	α-Tomlinson/Nordlund/Thurman	36; 2B; P(5)	0.59	0.23	0.39	0.34	0.28	0.23
		34	α-API/Nordlund/Thurman	36; 2B; P(5)	0.79	0.35	0.44	0.41	0.33	0.27
		32	β-Method/Thurman	36; 2B; P(5)	0.48	0.23	0.48	0.23	0.19	0.15
		40	SPT-97		1.23	0.55	0.45	0.64	0.51	0.41
	Concrete Piles	33	α-Tomlinson/Nordlund/Thurman	36; 2B; P; Hara(5h)	0.96	0.47	0.49	0.46	0.36	0.29
		80	α-API/Nordland/Thurman	36; 11.5B; Sch; T&P(8)	0.87	0.42	0.48	0.42	0.34	0.26
		80	β-Method/Thurman	36; 11.5B; Sch; T&P(8)	0.81	0.31	0.38	0.47	0.39	0.32
		71	SPT-97 mob		1.81	0.91	0.50	0.84	0.67	0.52
		30	FHWA CPT		0.84	0.26	0.31	0.57	0.48	0.40
	Pipe Piles	13	α-Tomlinson/Nordlund/Thurman	36; 2B; P(5)	0.74	0.44	0.59	0.29	0.22	0.17
		32	α-API/Nordland/Thurman	36; 2B; P(5)	0.80	0.36	0.45	0.41	0.33	0.26
		29	β-Method/Thurman	36; 2B; P(5)	0.54	0.26	0.48	0.26	0.21	0.16
		33	SPT-97 mob		0.76	0.29	0.38	0.45	0.37	0.30

[(1)]See Table 6 for details;
[(2)] Numbers in parentheses refer to notations used for detailing soil parameters combinations (see Table 7b and Appendix C for more details), See Tables 7a and 8 for soil properties' correlations to SPT and CPT respectively, 36 = limiting friction angle, B = pile diameter 2B, 11.5B contributing zone to tip resistance.

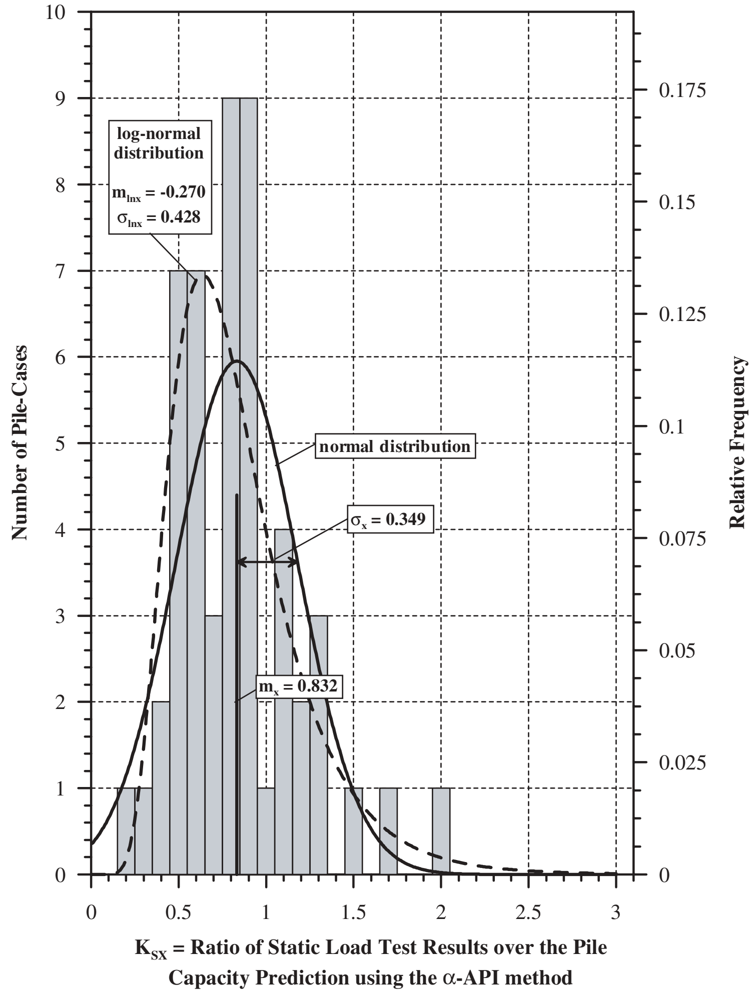


Figure 17. Histogram and frequency distributions of K_{sx} for 52 cases of all pile types (concrete, pipe, H) in clay.

Histogram and frequency distributions were prepared for the identified critical categories in order to examine the match between the actual data and PDFs. Figures 28 through 35 present histograms of some of the datasets, along with the best-fitting normal and lognormal distributions. By and large, the lognormal distributions seem to match the data well and hence are the preferable choice to the normal distributions. Moreover, the lognormal distribution matches the low end tail of the cases (lower left corner of the data), where the extreme overpredicting cases exist. Appendix B presents detailed graphical presentations of the data and various correlations.

3.1.2.3 Intermediate Conclusions

The data presented in Table 17 and Figures 27 through 35 lead to the following intermediate conclusions: (1) Signal matching generally underpredicts pile capacity, while the method performs well for BOR (last restrike) cases and, (2) the simple Energy Approach provides a good prediction for pile capacity during driving (EOD). These conclusions suggest that construction delays due to restrike and costly signal matching analyses need to be examined in light of capacity-time dependency and economic factors.

3.1.3 Drilled Shafts—Static Analysis

Table 18 presents a summary of the analysis results used for static capacity evaluation of drilled shafts, compared with the nominal resistance based on the FHWA failure criterion. The data in Table 18 are limited to cases within two standard deviations of the mean of the initial analysis results of all the drilled shafts. The resistance factors for the different target reliability values were calculated for a ratio of dead load to live load of 2.0. Reviewing the information

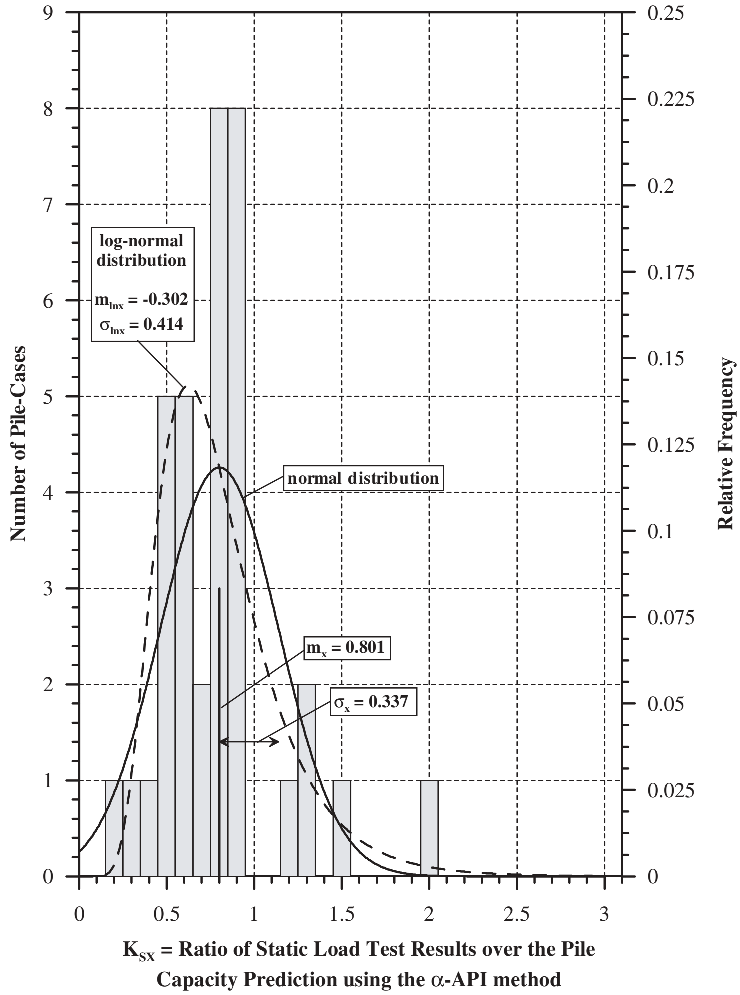


Figure 18. Histogram and frequency distributions of K_{sx} for 36 cases of concrete and pipe pile types in clay.

presented in Table 18, one can conclude that the resistance factors are within the range of current practice and the subcategorization provides details regarding both the method of design and construction. The design methods in general provide more accurate predictions than those for driven piles, as indicated by the bias being closer to one where the COVs are of similar magnitudes to those in Table 16. The "mixed" construction method represents the combination of other available construction methods. The actual numbers for the mixed case do not necessarily add up to the sum of the individual categories as each set (individual or combined) is treated independently.

Figures 36 through 40 present selected subsets from Table 18 as histograms, along with the best-fitting normal and lognormal distributions. Four of the figures relate to the mixed construction case histories that include other construction methods. The smaller subsets' databases and wider distributions are the result of the variation between the methods and the wide sources of the data. Additional graphical presentation of the data is included in Appendix C.

3.2 INITIAL EXAMINATION OF RESULTS

3.2.1 Overview

An initial examination of the results is required in order to assess the magnitude of values and to allow the process of transforming the large number of methods and correlations to meaningful and inclusive categories. This is done by checking the number of case histories needed to be eliminated when limiting the set being investigated to those within the two standard deviation band, recalculating the resistance factors for the recommended target reliabilities, evaluating equivalent factors of safety, and examining the efficiency of the

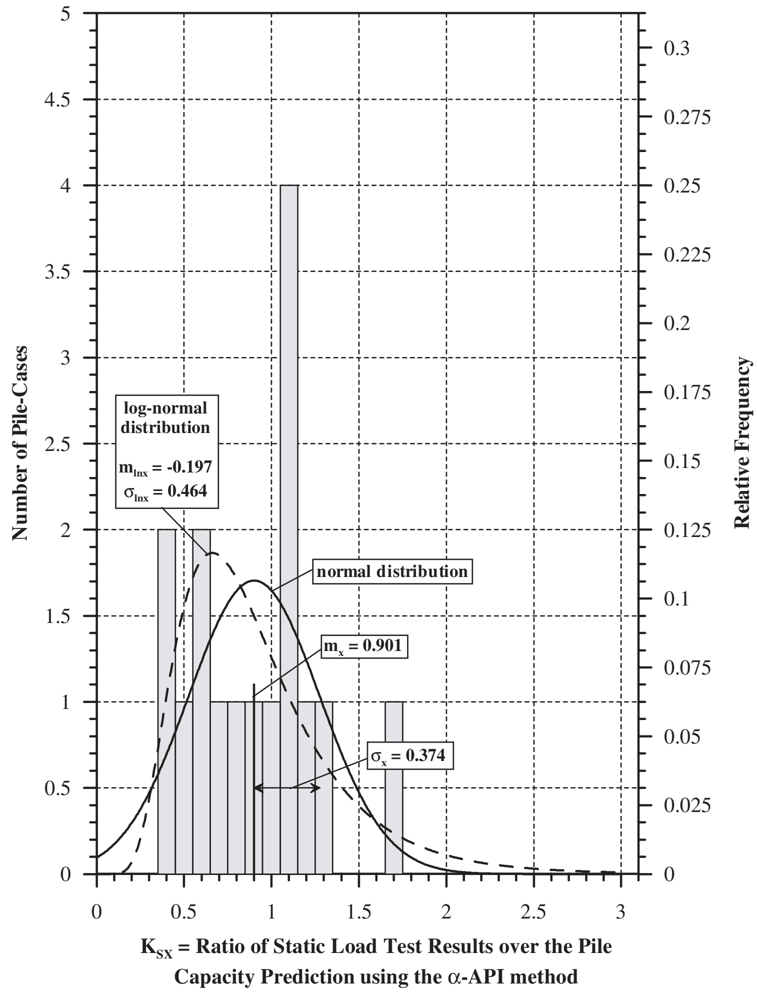


Figure 19. Histogram and frequency distributions of K_{sx} for 16 cases of H piles in clay.

methods by comparisons of factors of safety and the efficiency factors. Additional evaluations are described in section 3.5.

3.2.2 FOSM Versus FORM

As the existing AASHTO specifications are based on FOSM (see section 1.4.3.4), the relationship between the factors obtained by FOSM and those obtained by the current methodology, FORM, needed to be checked. Figure 41 presents these relationships for the different categories of the analyzed methods for all three databases and for a reliability index of $\beta = 2.33$. The data in Figure 41 suggest that FORM results in resistance factors consistently higher than those obtained by FOSM. The ratio between the two suggests that, as a rule of thumb, FORM provides resistance factors approximately 10% higher than those obtained by FOSM. Two practical conclusions can be drawn from these data: (1) first evaluation of data can be done by the simplified, closed form FOSM approach, with the obtained resistance factors on the low side; and (2) the resistance factors obtained in this study (as presented in Tables 16 through 18) can be directly compared to the current specifications and other LRFD codes based on FOSM.

3.2.3 Equivalent Factors of Safety

The fact that the resistance factors using FORM approximate those obtained by FOSM allows the use of a simplified relationship between resistance factor and FS based on FOSM and provided by Barker et al., 1991:

$$FS = \frac{\gamma_D \dfrac{Q_D}{Q_L} + \gamma_L}{\phi\left(\dfrac{Q_D}{Q_L} + 1\right)} \tag{32}$$

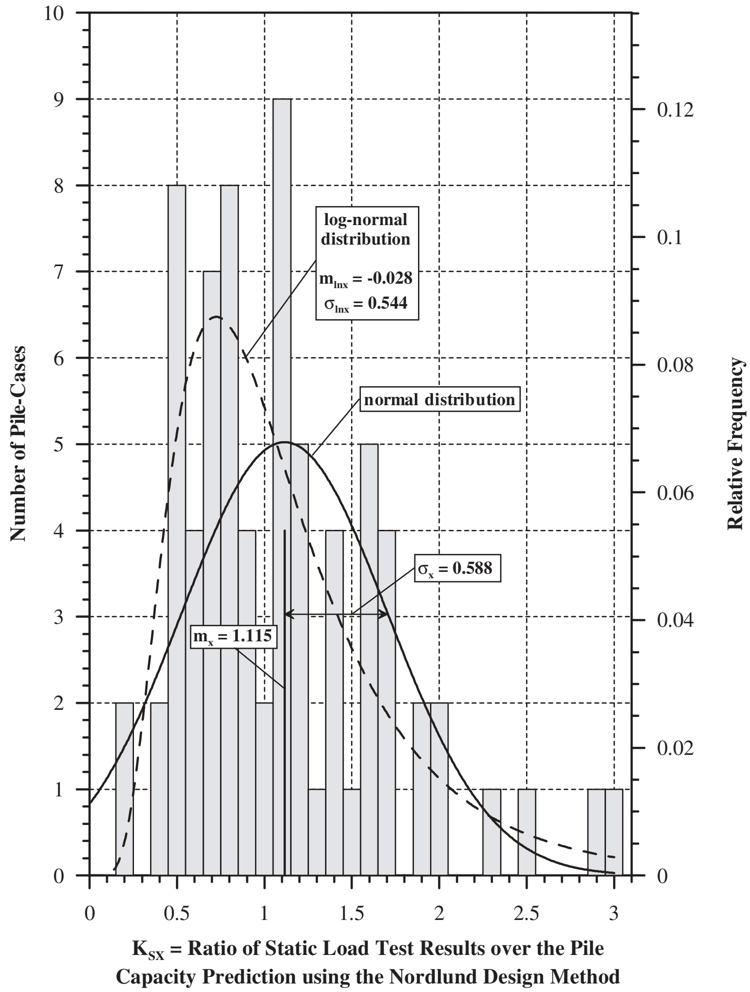


Figure 20. Histogram and frequency distributions of K_{sx} for 74 cases of all pile types (Concrete, Pipe, H) in sand.

using DL/LL = 2, $\gamma_L = 1.75$,and $\gamma_D = 1.25$ (load factors taken from the structural code) for which the resistance factors were calculated, results in:

$$FS \cong 1.4167/\phi \quad (33)$$

3.2.4 Detailed Tables

Tables 19, 20, and 21 present detailed evaluations of the analyzed case histories for static analyses of driven piles, dynamic analyses of driven piles, and static analyses of drilled shafts, respectively. The tables include the number of case histories in the subset as well as the number of case histories used in the analysis of resistance factors. The efficiency factors, ϕ/λ, are calculated and presented with the resistance factors. The approximated factors of safety associated with the calculated resistance factors based on equation 33 are provided as well. The factors of safety are presented along with the mean overprediction ratio (calculated FS × the bias of the method), which in effect represents both the actual FS and a measure of the economic efficiency of the method—the lower the value, the smaller the number of deep foundations required and the lower the cost, therefore the greater economic efficiency of the method.

Table 22 provides a summary of Tables 19 through 21, presenting resistance factors and efficiency measures for select categories of method/pile/soil combinations.

The LRFD principles are clearly seen in the obtained values as the application of consistent target reliability produces values related to the individual method. While a method/condition combination that has large variability (expressed as COV) results in low resistance factors, the resistance factors alone do not provide a measure of the efficiency of the method. For example, SPT 97 for H piles in sand has a resistance factor $\phi_{(\beta=2.33)} = 0.63$ while the Nordlund method for the same category results in a lower resistance factor $\phi = 0.46$. In fact, SPT 97 underestimates the capacity ($\lambda = 1.35$), while Nordlund's method slightly overestimates it ($\lambda = 0.94$); as a result, Nordlund's method has an efficiency similar to that of SPT 97

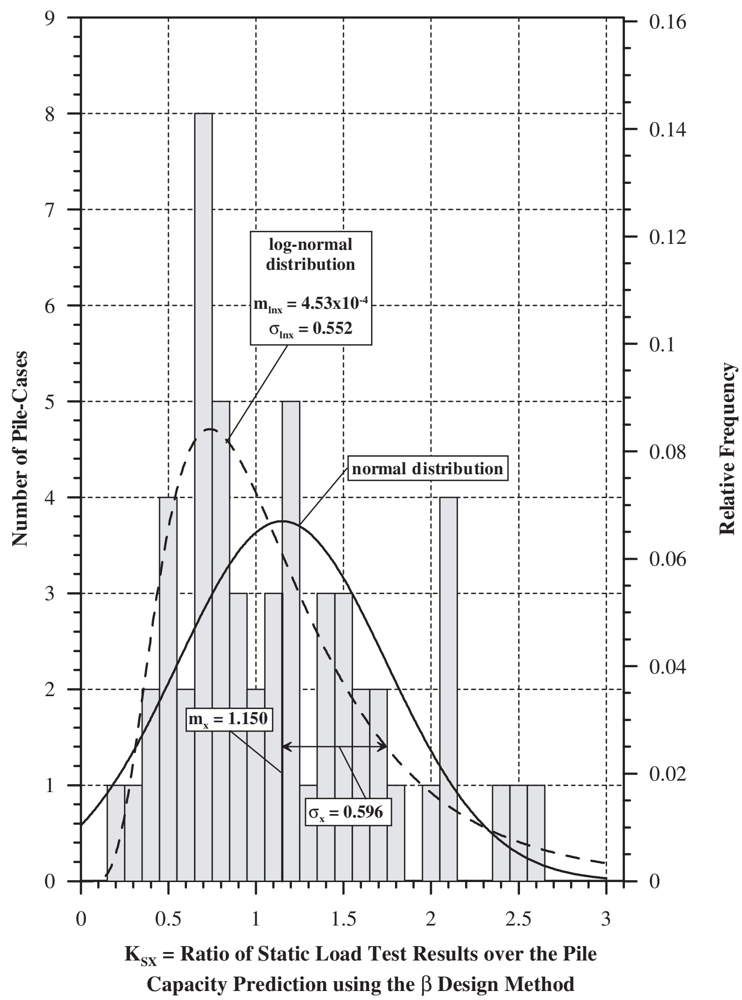


Figure 21. Histogram and frequency distributions of K_{sx} for 56 cases of pipe and concrete pile types in sand.

(COV 0.40 vs. 0.43) in spite of the large difference in the resistance factors. Examining the efficiency factors, one clearly sees that the method that provides the highest ϕ/λ ratio also provides the lowest "actual" factor of safety ($FS \times \lambda$). The factors of safety presented in Table 22 for $\beta = 3.0$ (the lower of the two values in the last column) are in line with what one would expect, ranging from 2.59 to 5.63, with an average of 3.73. The use of lower target reliability for redundant piles ($\beta = 2.33$) provided factors of safety ranging from 2.11 to 4.00 (avg. 2.94), which are judged to be reasonable as well.

The recommended resistance factors based on Tables 18 through 22 are presented in section 3.4.

3.2.5 Resistance Factors for Pullout of Driven Piles

Utilizing the University of Massachusetts Lowell static pile database, a limited number of case histories were identified for which a static pile load test in tension (pullout) was carried out. The available data were analyzed and the resulting statistical parameters and associated resistance factors are presented in Table 23. The results, though based on limited data, seem to be consistent with expected behavior. Comparing the data in Table 23 to that presented in Table 19 for driven piles under compression, the following can be observed: (1) large displacement piles in clay develop similar friction under compression or tension, (2) friction for small displacement piles (H) is smaller in tension than in compression, and (3) friction under pullout of all piles in sand is smaller than that which develops under compression. The recommended resistance factors for pullout tests are presented and discussed in section 3.4.

3.3 PILE TESTING

3.3.1 Overview

Deep foundation testing is carried out as a quality control to check or verify pile capacity and integrity. Quality control

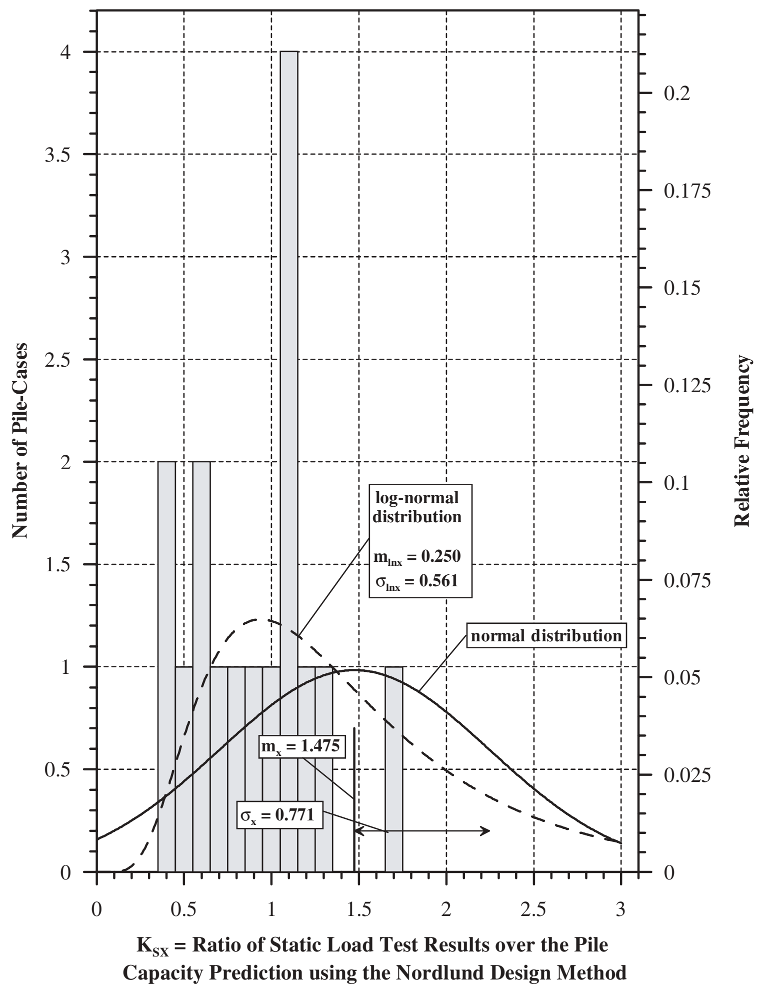


Figure 22. Histogram and frequency distributions of K_{sx} for 19 cases of pipe piles in sand.

of piles and drilled shafts is performed via static load testing, various methods of dynamic (impact) testing, and integrity testing. The first two are carried out to determine pile capacity and integrity while the last is utilized for structural quality assurance only. Two issues need to be addressed: (1) the testing method's performance and associated resistance factors, and (2) the number of tests that need to be carried out.

Section 3.1 addressed the methods of dynamic analysis most commonly used during driving. The case histories in the extensive PD/LT2000 database have widely varied subsurface conditions; hence, the direct calibration of the different analysis method is applicable to all site conditions. The evaluation of the required number of tests needs to assess a single site variability and evaluate how many piles are required to be tested to guarantee a target capacity. A single site variability, therefore, utilizes judgment and assigns categories that cannot be based on firm data. The following sections address issues associated with pile testing.

3.3.2 Resistance Factors for Static Pile Load Tests

Assigning resistance factors to associate with (in situ) pile (or drilled shaft) static load test results requires an estimate of the corresponding mean bias and COV. By definition, the mean bias is 1.0, since load tests directly measure in situ pile capacity either to failure or to a maximum applied load (proof test). The COV reflects spatial variation from one pile to another at the same site, along with whatever variation is introduced by the definition of failure criterion.

Empirical data of sufficient quality to estimate within-site variability is lacking. Therefore, an assumption is made to categorize sites as having low, medium, or high variability and to assign coefficient of variations of 0.15, 0.25, and 0.35 to these three cases respectively (Phoon and Kulhawy, 1996; Trautmann and Kulhawy, 1996).

In addition to the natural variability within a site, the interpretation of failure criterion itself (i.e., Davisson's criterion

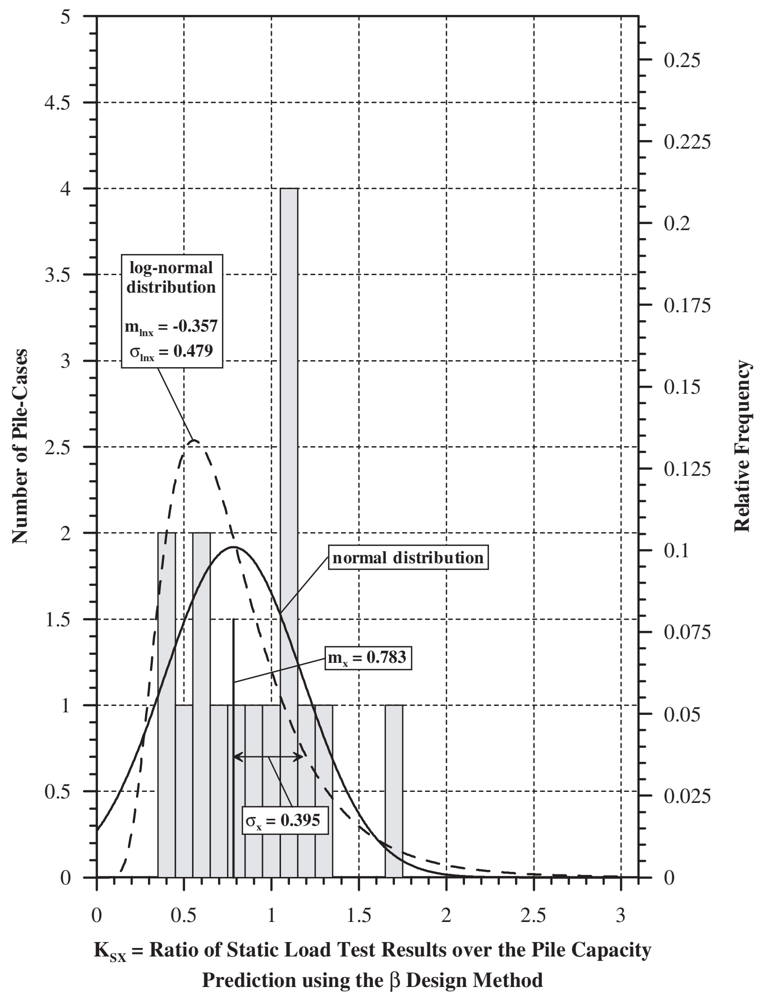


Figure 23. Histogram and frequency distributions of K_{sx} for 19 cases of H piles in sand.

for driven piles and FHWA for drilled shafts) introduces variability. Analyses reported in sections 2.3.2 and 2.3.4 suggest that this COV due to the failure criterion alone is about 0.10. Adding this to the COV for the natural within-site variability yields COVs of 0.18, 0.27, and 0.36 for low, medium, and high variability sites, respectively.

The reduction in COV on the mean pile capacity at the site should decrease in proportion to $1/\sqrt{n}$, where n is the number of pile tests (Benjamin and Cornell, 1970). This leads to the results presented in Table 24, which describes the resistance factors as a function of the site variability, number of piles tested, and target reliability. The recommended factors assigned are presented in section 3.45.

3.3.3 Numbers of Dynamic Tests Performed on Production Piles

Dynamic pile testing is carried out for quality control, target capacity, integrity, and driving system performance. A certain number of production piles are tested to ensure that the piles, as constructed, are satisfactory, in a way that is similar to quality assurance testing in manufacturing.

For specifying a quality assurance testing plan, the number of piles to be dynamically tested and the criterion for accepting a set of piles need to be determined. Two concerns are at issue: the chance that poor quality (i.e., under capacity) piles are incorrectly accepted as being good, and the chance that good quality piles are incorrectly rejected as being poor (see Figure 42). For a given level of sampling effort or cost, reducing the chance of one kind of error invariably increases the chance of the other, and thus the two must be balanced.

The definition of "poor quality" piles was taken to be that the average pile capacity is less than the design capacity, that is, less than the reciprocal of the factor of safety. The probability chosen as a reasonable chance that such a set of poor quality piles be incorrectly accepted as good was equilibrated to a reliability index of three, or a probability of about 0.001. This is sometimes called the "buyer's risk" or the "owner's risk" as demonstrated in Figures 42 and 43. The definition of

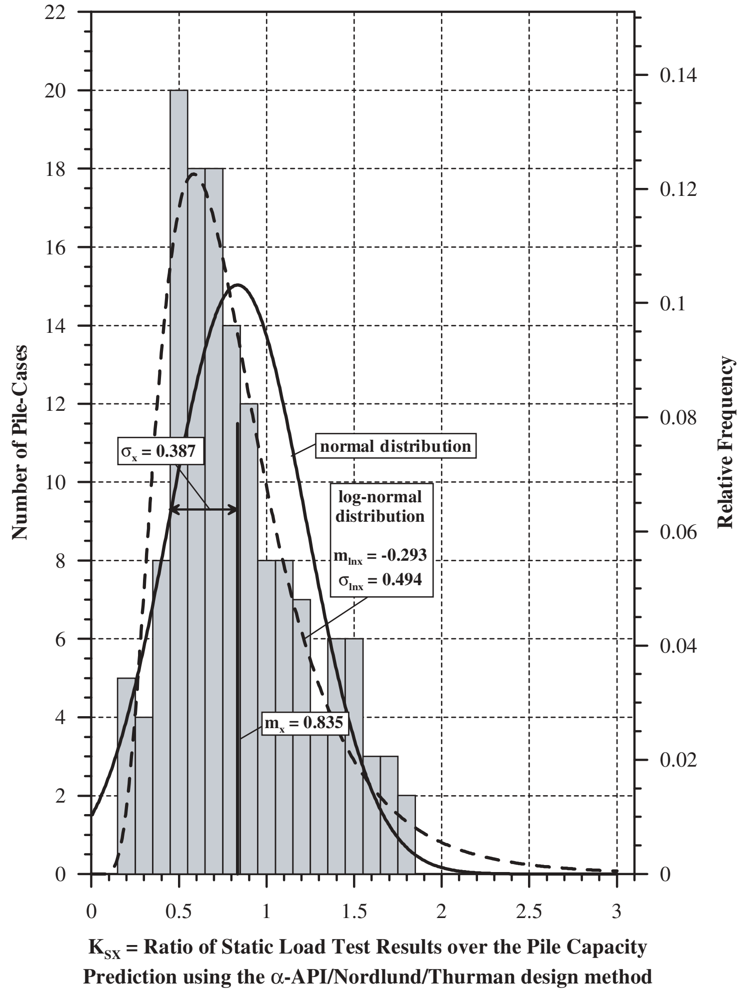


Figure 24. Histogram and frequency distributions of K_{sx} for 146 cases of all pile types (concrete, pipe, H) in mixed soil.

"good quality" piles was taken to be nominal capacity. The probability chosen as a reasonable chance that such a set of good quality piles be incorrectly rejected as poor was taken to be 0.10. This is sometimes called the "seller's risk" or the "contractor's risk."

Given a large total number of piles, N, relative to the number tested, n, the variance of the sample mean, $\bar{x}$, is (Benjamin and Cornell, 1970),

$$Var(\bar{x}) \cong \frac{N-n}{N-1}\frac{\sum_{i=1}^{n} x_i}{n-1} \tag{34}$$

For sampling fractions, $f = n/N$, greater than about 10%, the "finite population correction factor," $(N - n)/(N - 1)$, comes into play. This reduces the sampling variance, because the assumption of sampling without replacement is no longer reasonable. For example, if 100% of the piles are tested, that is, if $f = n/N = 1$, then there is no variance in the sample average since all the piles have been accounted for.

An assumption, consistent with that projected in section 3.3.2, is made to categorize sites as having low, medium, or high variability and to assign COVs of 0.15, 0.25, and 0.35 to these three cases, respectively. In addition, the dynamic test method also introduces variability. For that, two methods are considered based on the results presented in sections 3.1.2 and 3.2.4, Energy Approach (EA) to be used for capacity evaluation at the EOD and signal matching (CAPWAP) to be used during BOR.

Setting the owner's and contractor's risks on the one hand, and the definitions of "good" and "poor" piles on the other, as defined above, and noting that the sampling variance of the average pile capacity of the tested piles decreases in proportion to $1\sqrt{n}$, where n is the number of pile tests, values for the number of piles to be tested can be estimated. The obtained

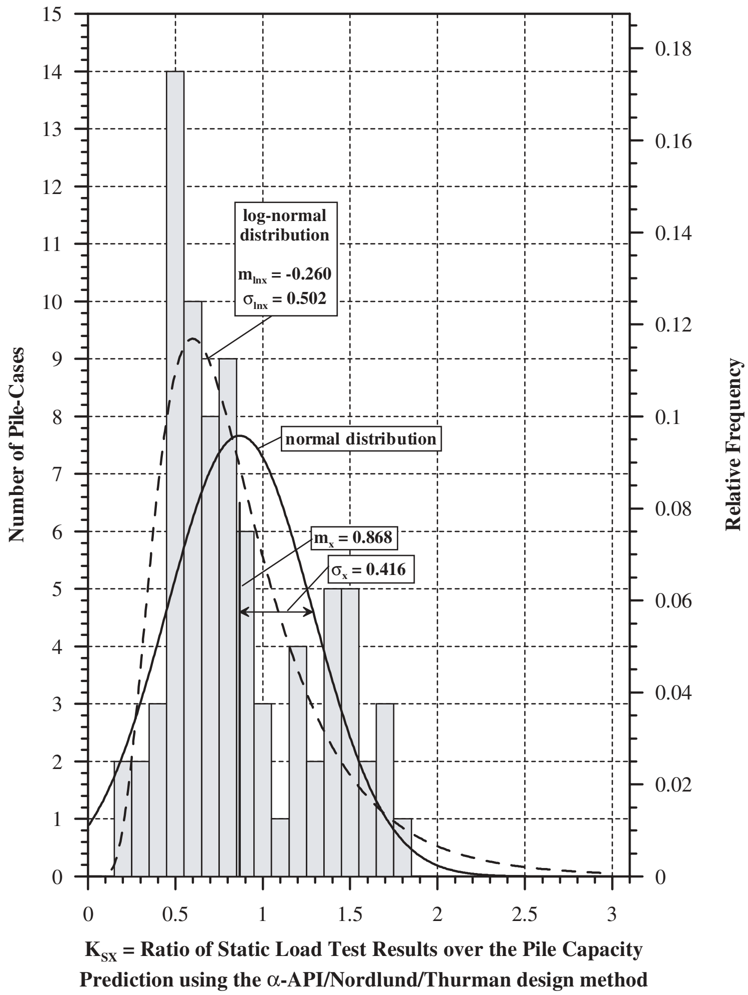


Figure 25. Histogram and frequency distributions of K_{sx} for 80 cases of concrete piles in mixed soil.

recommendations based on these estimates are presented in section 3.4.3.

3.3.4 Testing Drilled Shafts for Major Defects

3.3.4.1 Overview

Drilled shafts require in-field casting and are subject to defects (especially when unlined in cohesionless soils). Acceptance sampling is used to assess whether an adequate majority of a set of shafts is free of major defects.

3.3.4.2 Statistical Background

A sample of n from N shafts is tested to identify major defects. Major defects are defined as any defect that significantly compromises the ability of the shaft to carry the assigned loads. Each tested shaft is categorized as either "good" or "defective." If no more than c of the n tested shafts are "defective," the set of shafts is accepted. The test parameter, c, is usually a small number.

Suppose that the set of N actual shafts includes m shafts with major defects. The fraction defective is denoted, $p = m/N$. Among samples of n tested shafts, the frequency distribution of the number of defective tested shafts, c, is of the hypergeometric form,

$$f_c(c|n,N,m) = \frac{C_{n-c}^{N-m} C_c^m}{C_n^N} \tag{35}$$

in which $f_c(c\,|\,n, N, m)$ is the frequency distribution, c is the number of defective test results within the sample, m is the number of defectives in the entire set of N shafts, and C_k^q is the number of combinations of k out of q things.

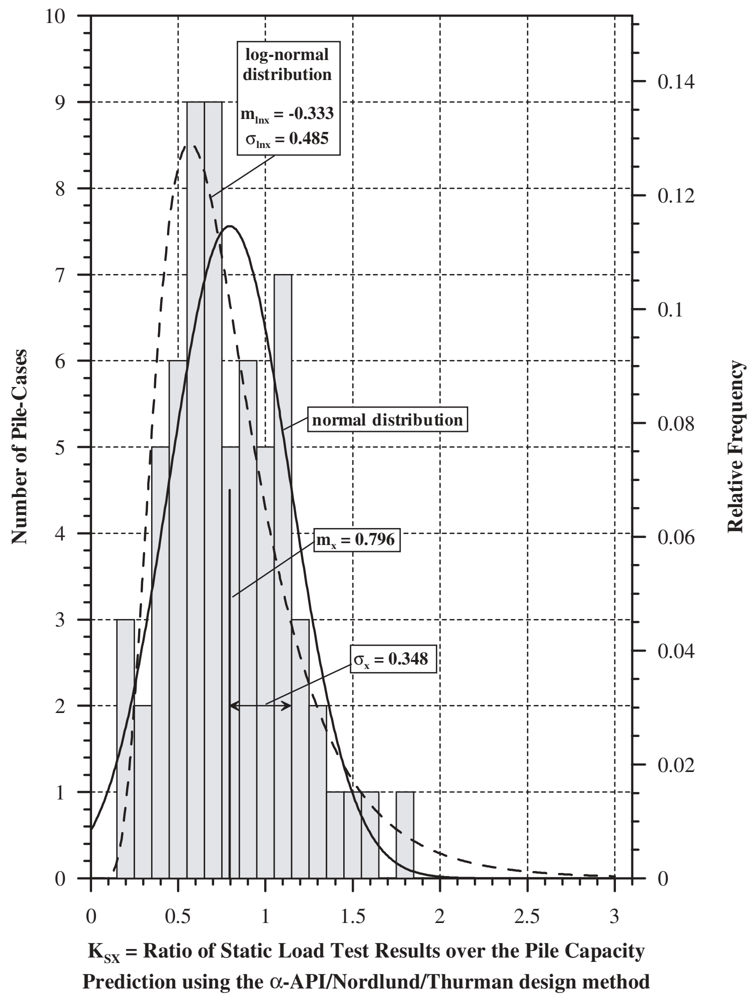


Figure 26. Histogram and frequency distributions of K_{sx} for 66 cases of pipe and H pile types in mixed soil.

For n/N less than about 10%, this frequency distribution can be reasonably approximated by the more easily calculated binomial distribution,

$$f_c(c \mid p, n) = \frac{n!}{c!(n-c)!} p^c (1-p)^{n-c} \tag{36}$$

in which $p = m/N$ is the fraction defective (Figure 44).

3.3.4.3 Sample Calculation

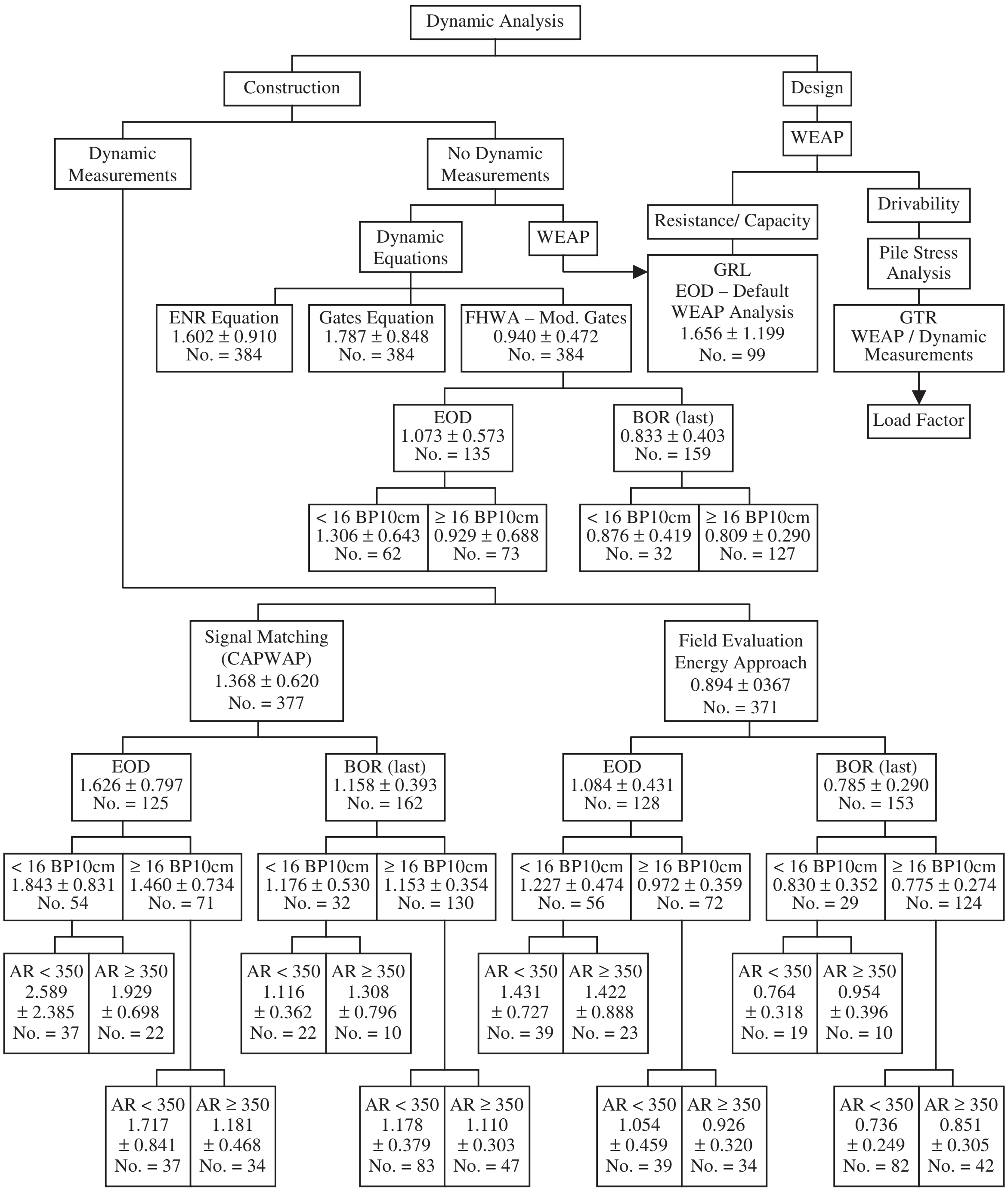
Presume that the maximum fraction of shafts with a major defect that the owner is willing to tolerate in a large set of N shafts is 5% and that the owner's risk of incorrectly accepting a set of shafts with greater than 5% defects is set at $\alpha = 0.10$. Let the contractor's risk of rejecting a set of N shafts with no more than, say, 1% defects be set at $\beta = 0.10$.

From the nomograph in Figure 44 (see insert), the assurance sampling plan is to test n = 110 of the shafts and require that no more than two are defective, (c = 2). This is a very large number of tests, but as can be seen from the nomograph, decreasing the tolerable percent defective from the owner's perspective or reducing either the owner's or contractor's risk, only increases the number of shafts, n, that must be tested.

This calculation assumes that n/N is less than about 10%, but the conclusion that large sample sizes, n, are required also holds for the case of a larger sampling fraction. Performing an iterative solution on the hypergeometric model for the same case as above, but assuming a finite N = 100, yields a sample size of about 80.

3.3.4.4 Conclusion

The conclusion to be drawn from these simple calculations is that, in order to statistically ensure very low rates of major defects within a set of drilled shafts, a very high proportion of the shafts must be tested. Thus, it seems reasonable practically



*All values represent the ratio of the static capacity based on Davisson's failure criterion over the dynamic methods prediction, mean ± 1 S.D.

Figure 27. Statistical parameters of a normal distribution for the various dynamic analyses (applied to PD/LT2000 database) grouped by the controlling parameters.

TABLE 17 The performance of the dynamic methods: statistical summary and resistance factors

	Method	Time of Driving	No. of Cases	Mean	Standard Deviation	COV	Resistance Factors for a given Reliability Index, β		
							2.0	2.5	3.0
Dynamic Measurements	CAPWAP	General	377	1.368	0.620	0.453	0.68	0.54	0.43
		EOD	125	1.626	0.797	0.490	0.75	0.59	0.46
		EOD - AR < 350 & Bl. Ct. < 16 BP10cm	37	2.589	2.385	0.921	0.52	0.35	0.23
		BOR	162	1.158	0.393	0.339	0.73	0.61	0.51
	Energy Approach	General	371	0.894	0.367	0.411	0.48	0.39	0.32
		EOD	128	1.084	0.431	0.398	0.60	0.49	0.40
		EOD - AR < 350 & Bl. Ct. < 16 BP10cm	39	1.431	0.727	0.508	0.63	0.49	0.39
		BOR	153	0.785	0.290	0.369	0.46	0.38	0.32
Dynamic Equations	ENR	General	384	1.602	1.458	0.910	0.33	0.22	0.15
	Gates	General	384	1.787	0.848	0.475	0.85	0.67	0.53
	FHWA modified Gates	General	384	0.940	0.472	0.502	0.42	0.33	0.26
		EOD	135	1.073	0.573	0.534	0.45	0.35	0.27
		EOD Bl. Ct. < 16BP10cm	62	1.306	0.643	0.492	0.60	0.47	0.37
WEAP		EOD	99	1.656	1.199	0.724	0.48	0.34	0.25

Notes: EOD = End of Driving; BOR = Beginning of Restrike; AR = Area Ratio; Bl. Ct. = Blow Count; ENR = Engineering News Record Equation; BP10cm = Blows per 10cm; COV = Coefficient of Variation; Mean = ratio of the static load test results (Davisson's Criterion) to the predicted capacity = $K_{SX} = \lambda$ =bias

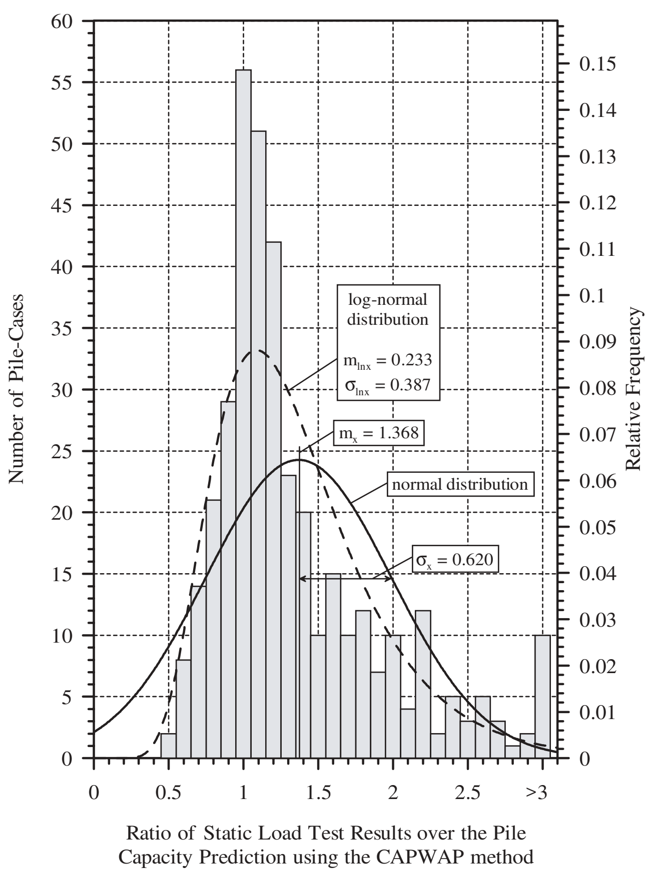


Figure 28. Histogram and frequency distributions for all (377) CAPWAP pile-cases in PD/LT2000.

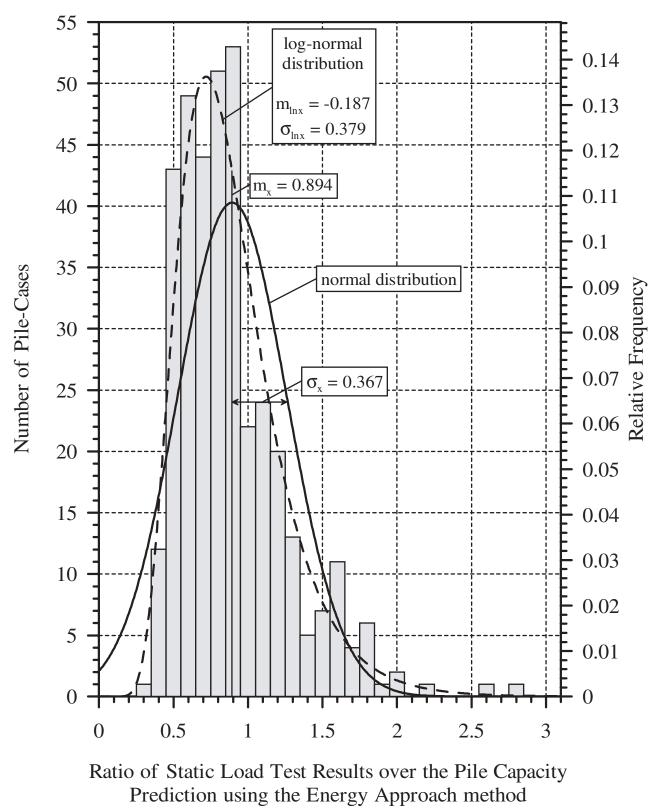


Figure 29. Histogram and frequency distributions for all (371) Energy Approach pile-cases in PD/LT2000.

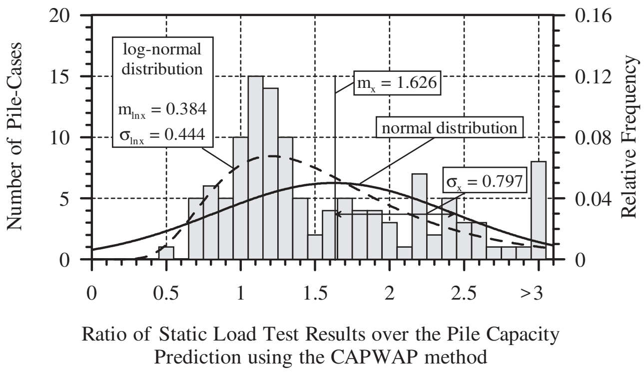


Figure 30. Histogram and frequency distributions for all EOD (125) CAPWAP pile-cases in PD/LT2000.

to require 100% of drilled shafts be postconstruction tested for major defects, as this ensures total quality at little additional cost.

3.4 RECOMMENDED RESISTANCE FACTORS

3.4.1 Overview

This section presents all the relevant resistance factors and recommendations for the AASHTO LRFD deep foundation design specifications. Tables 25 through 30 are based on material provided in the previous sections and are presented in an integrated format according to similarity of calculated factors, extent of data on which the factors were based, and relevant issues that have to be addressed as comments to the recommended values. The factors are divided based on pile redundancy as discussed in section 2.7.5.

The recommended resistance factors represent the most significant attempt to date to develop LRFD code for deep foundations based on empirical data.

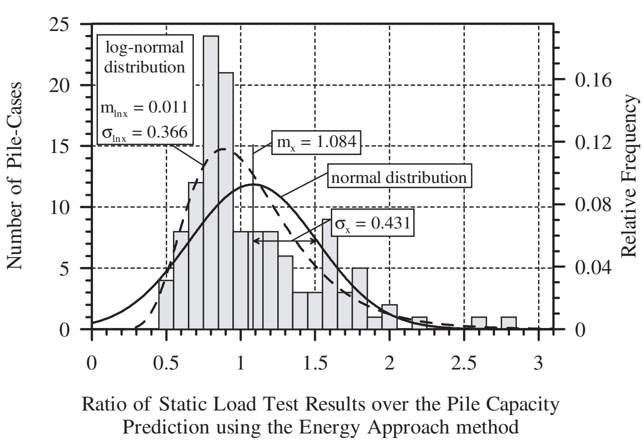


Figure 31. Histogram and frequency distributions for all EOD (128) Energy Approach pile-cases in PD/LT2000.

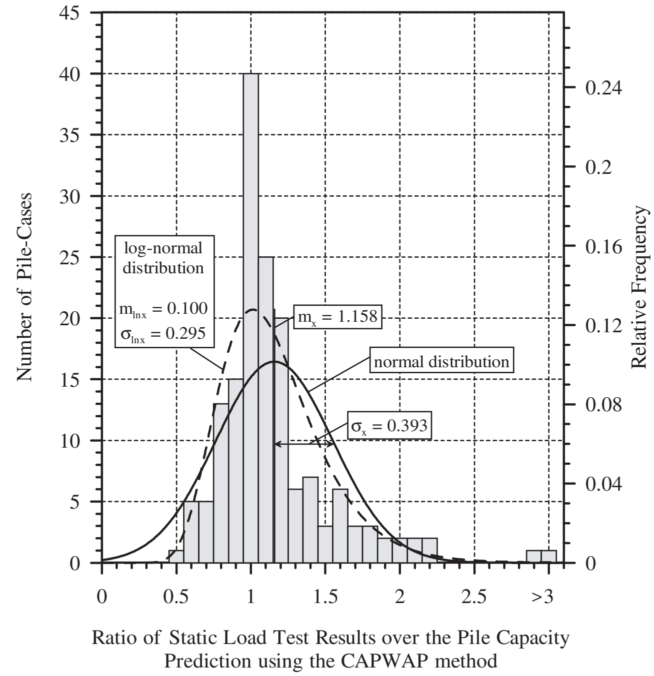


Figure 32. Histogram and frequency distributions for all BOR-last (162) CAPWAP pile-cases in PD/LT2000.

3.4.2 Static Analysis of Driven Piles

Table 25 presents the recommended resistance factors to be used with static analysis of driven piles under compression, as well as the individual efficiency factor of each method, which indicates the method's relative economic merit. The design methods should be applied based on soil parameters obtained via a subsurface exploration program with the detailed application and correlations as outlined in Tables 6, 7, 8, and 19 and further detailed in Appendices B and D. Table 26 presents the recommended factors to be used under tension (pullout) conditions, analyzing the skin friction in the same way as it was for compression loads, excluding tapered piles.

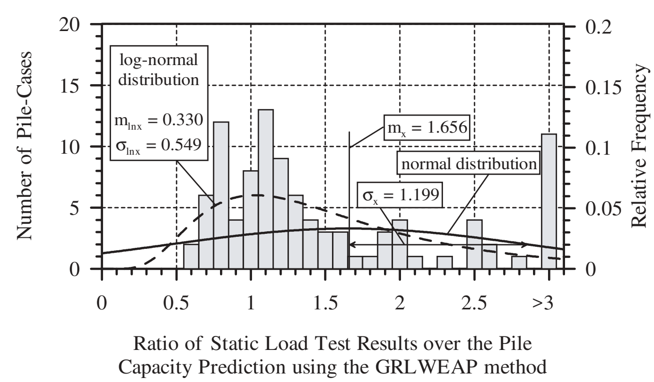


Figure 33. Histogram and frequency distributions for EOD default value GRLWEAP pile-cases, (99), data provided by GRL (see Hannigan et al., 1996).

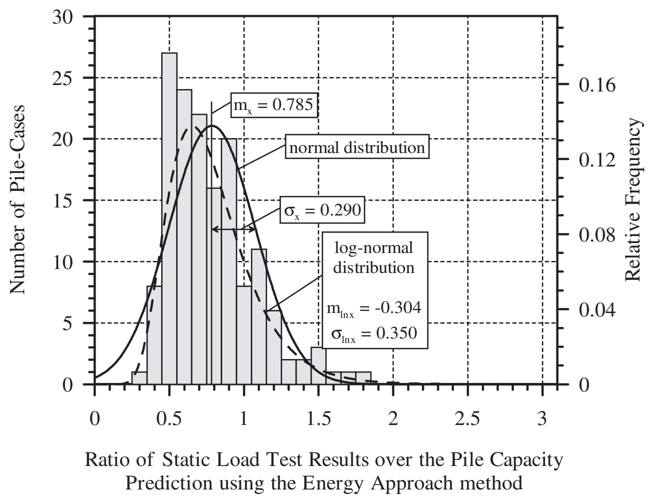


Figure 34. Histogram and frequency distributions for all BOR-last (153) energy approach pile-cases in PD/LT2000.

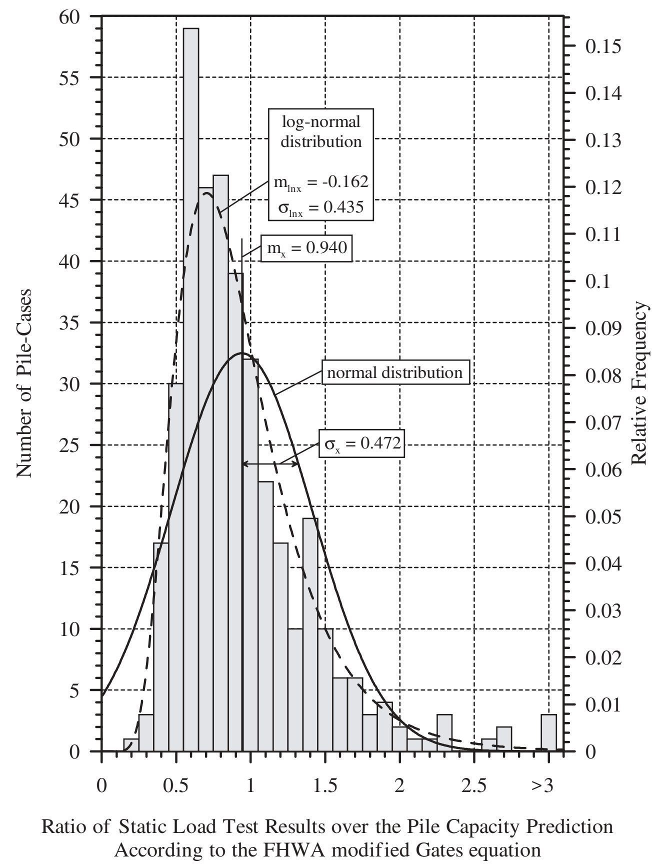


Figure 35. Histogram and frequency distributions for all (384) FHWA modified Gates equation pile-cases in PD/LT2000.

The assigned resistance factors are based on the LRFD principle of a consistent prescribed reliability for either a redundant or a nonredundant pile cap configuration. The recommended values should not be affected by the quality control procedure to be implemented in the construction stage other than through the relationship with the anticipated ultimate capacity as explained in item 3 of section 3.4.7.

3.4.3 Dynamic Analysis of Driven Piles

Table 27 presents the recommended resistance factors to be used for dynamic monitoring of driven piles and the relevant method's efficiency factors. The dynamic methods are categorized according to the controlling parameter and the time of driving. Table 28 presents the recommended number of tests required during production, with values rounded to the next highest integer. Dynamic tests at EOD are carried out for capacity evaluation, monitoring the performance of the driving system, and establishing driving criteria. As such, EOD tests are of great importance beyond the capacity evaluation alone.

The following comments relate to the way site variability is being established:

1. Site variability relates to the variability within similar subsurface conditions of the same site, not between sites. For example, when piers are based on substantially different subsurface conditions (i.e., in the stratum mostly influencing the pile capacity). The criteria should be applied independently to each pier location as a separate site.
2. Site variability can be determined by judgment or using the following approximate criteria related to borings representative of the entire site subsurface conditions:
   a. Relate to each significant bearing layer, average parameters used for strength analysis (e.g., N SPT) at each boring location.
   b. Check the COV between the average values for each identifiable significant layer obtained at each boring location.
   c. Categorize site variability in the following way:
      i) COV < 25%—Low
      ii) 25% ≤ COV < 40%—Medium
      iii) 40% ≤ COV—High

The following recommendations apply to dynamic tests:

1. Restrike should be scheduled according to the guidelines provided in section 3.4.6
2. The recommended values in Table 28 relate to similar pile types driven at the same site.
3. For EOD conditions:
   - If dynamic measurements are available, evaluate pile capacity using the Energy Approach; if dynamic measurements are not available, evaluate pile capacity using the Gates or the FHWA modified Gates.
   - Signal matching is recommended for EOD conditions for end bearing piles only.

TABLE 18 The performance of the drilled shafts' static analysis methods—statistical summary and resistance factors for data using mean ± 2 SD

Capacity Component	Soil Type	Design Method	Construction Method	No. of Cases	Mean	COV	Resistance factors for a given reliability index β		
							2.0	2.50	3.0
Skin Friction + End Bearing	Sand	FHWA	Mixed	32	1.71	0.60	0.66	0.51	0.38
			Casing	12	2.27	0.46	1.15	0.92	0.73
			Slurry	9	1.62	0.74	0.48	0.35	0.25
		R&W	Mixed	32	1.22	0.67	0.41	0.31	0.23
			Casing	12	1.45	0.50	0.68	0.54	0.42
			Slurry	9	1.32	0.62	0.49	0.37	0.28
	Clay	FHWA	Mixed	53	0.90	0.47	0.45	0.36	0.28
			Casing	13	0.84	0.50	0.39	0.31	0.24
			Dry	40	0.88	0.48	0.43	0.34	0.27
	Sand + Clay	FHWA	Mixed	44	1.19	0.30	0.82	0.69	0.58
			Casing	21	1.04	0.29	0.73	0.62	0.52
			Dry	12	1.32	0.28	0.94	0.80	0.68
			Slurry	10	1.29	0.27	0.94	0.80	0.69
		R&W	Mixed	44	1.09	0.35	0.68	0.57	0.47
			Casing	21	1.01	0.42	0.55	0.45	0.36
			Dry	12	1.20	0.32	0.79	0.67	0.56
			Slurry	10	1.16	0.25	0.88	0.76	0.65
	Rock	C&K	Mixed	46	1.23	0.41	0.68	0.56	0.45
			Dry	29	1.29	0.40	0.73	0.60	0.49
		IGM	Mixed	46	1.30	0.34	0.83	0.69	0.57
			Dry	29	1.35	0.31	0.91	0.77	0.65
Skin	Sand	FHWA	Mixed	11	1.09	0.51	0.50	0.40	0.31
		R&W	Mixed	11	0.83	0.54	0.36	0.28	0.22
	Clay	FHWA	Mixed	13	0.87	0.37	0.52	0.43	0.36
	Sand + Clay	FHWA	Mixed	14	1.25	0.29	0.87	0.75	0.63
		R&W	Mixed	14	1.24	0.41	0.69	0.56	0.46
	All Soils	FHWA	Mixed	39	1.08	0.41	0.60	0.49	0.40
		R&W	Mixed	25	1.07	0.48	0.52	0.42	0.33
	Rock	C&K	Mixed	16	1.18	0.46	0.60	0.48	0.38
		IGM	Mixed	16	1.25	0.37	0.75	0.62	0.51

- Restrike data is recommended to be interpreted by dynamic measurements, using signal matching analysis (CAPWAP).

4. Systematically, test the number of piles according to Table 28 at the chosen time of driving (EOD or BOR). Restrike tests following EOD tests on the same pile are important for the identification of changes with time (setup and relaxation); however, one test type should not be substituted for the other.
5. Average the results of the tested piles. All tested piles should be included regardless of their performance. No "good" pile result can substitute for a "bad" one, even if a replacement is required. If the average capacity of the tested pile is greater than or equal to 85% of the nominal ultimate capacity, then accept the set of piles as "good"; otherwise reject the set as "poor." At this stage, alternative solutions are chosen, e.g., reduce the required nominal strength by adding piles, drive piles deeper, etc.
6. The above criteria provide a statistical approximation for the entire pile group. From a practical point of view, a separate criterion should be set as a minimum accepted value for each pile (e.g., 1.25 to 1.5 times the design load).

3.4.4 Static Analysis for Drilled Shafts

Table 29 provides the recommended resistance factors to be used for the static analysis of drilled shafts. The design methods should be applied based on soil parameters obtained via a subsurface exploration program with the detailed method application and correlations as outlined in section 2.5, and Tables 7 and 8, with further details in Appendix C. All drilled shafts should be tested for structural integrity as recommended in section 3.3.4.4.

3.4.5 Static Load Test

Table 30 provides the recommended resistance factors for static load tests under any testing procedure for both driven piles and drilled shafts. Testing of driven piles should be scheduled according to the recommendations provided in section 3.4.6. The site variability can be determined according to the comments listed in section 3.4.3. The nominal strength for driven piles should be determined based on Davisson's failure criterion or the maximum applied load if the pile does not reach failure. The same criterion should be used for piles tested in tension, omitting the offset displacement of the elastic compression line. Drilled shaft capacity

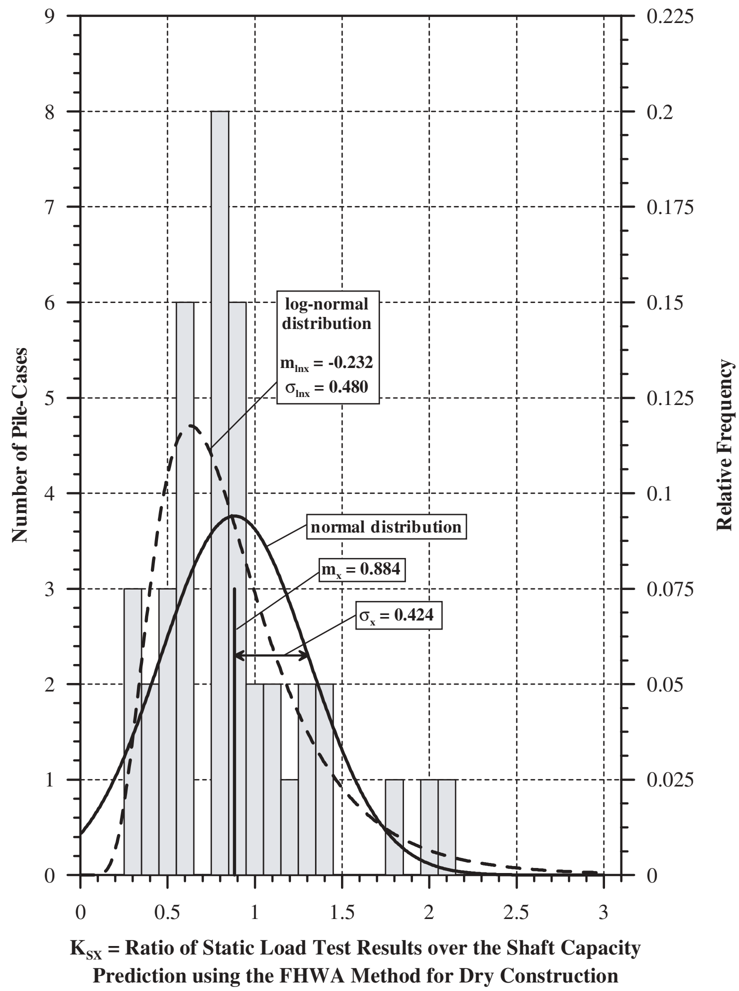


Figure 36. Histogram and frequency distributions for K_{sx} for 40 cases of drilled shafts in clay.

should be determined based on the smaller of the two, the FHWA criterion or the maximum applied load on the pile. The relationship between the number of tests and the resistance factor is based on similar piles (geometry and size) tested at the same site (see section 3.4.3). The recommended resistance factors should be applied to the mean capacity determined for all tests.

3.4.6 Pile Test Scheduling

Static or dynamic tests (restrikes) should be performed no sooner than before the pile has gained 75% of its capacity. This can be established as follows:

For piles embedded completely in clay:

For static testing purpose: $t_{75} = 1540 \times r^2$ (37)

For dynamic testing purpose: $t_{75} = 85 \times r$ (38)

For piles embedded in alternating soil conditions (granular and cohesive):

For dynamic testing purpose: $t_{75} = 39 \times r$ (39)

Where:

t_{75} = time to reach 75% of maximum capacity in hours
r = pile radius (or equivalent) in feet.

3.4.7 Design Considerations

Figure 3 outlines the process of deep foundation design and construction. The following sequence of comments address several of the steps in that process in relation to the previous sections:

1. When analyzing the field and laboratory testing for strength and deformation parameters, two additional

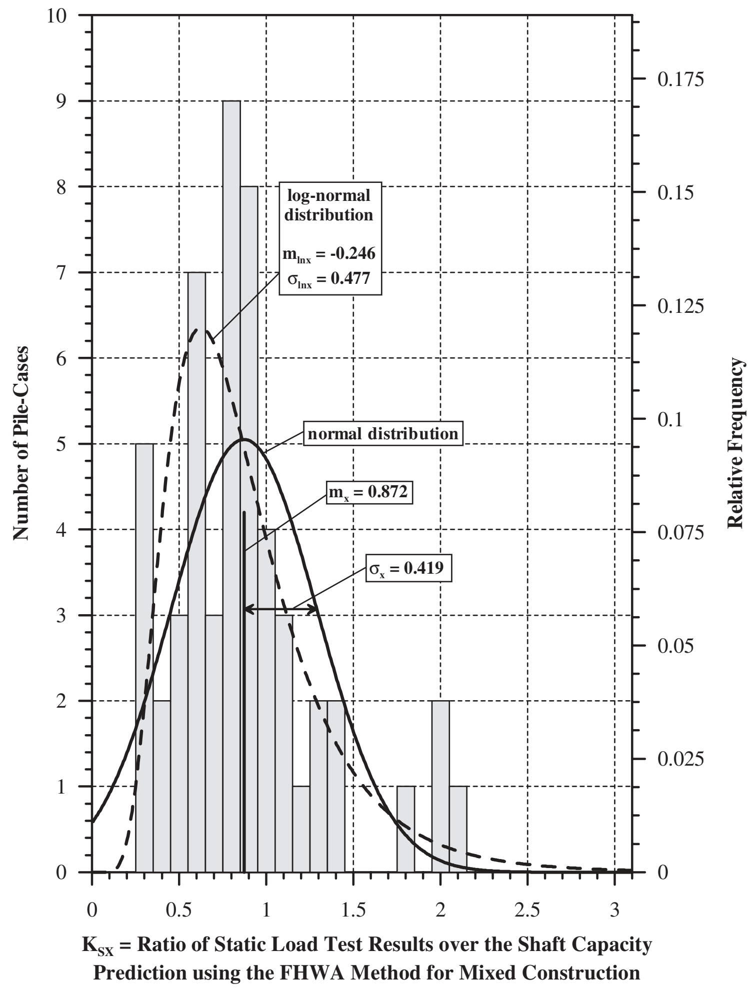


Figure 37. Histogram and frequency distributions for K_{sx} for 53 cases of drilled shafts in clay.

factors need to be established (related to the comments in section 3.4.3) (a) the number of different "sites" recognized in the project, and (b) the level of site variability associated with each site.

2. When performing static analysis for the designed deep foundations, resistance parameters from Tables 25 and 26 should be used for driven piles and from Table 29 for drilled shafts. Resistance parameters from Table 26 should be used for driven piles under tension. Attention should be given to the efficiency factors as a measure of economic scale. The factors should be applied according to the redundancy status of the pile cap arrangement. Without prebid pile field testing, the testing planned during construction (e.g., static and/or dynamic) should not affect the resistance factors used in the design stage other than as described in the following item (3).
3. For driven piles, a drivability study is carried out during the design stage in order to assess the pile installation. For this purpose alone, the required ultimate pile capacity can be established through the required design load and the resistance factors to be used during the construction. For example, if the required design load is F_d, the site is of medium variability, and two static load tests will be performed, Table 30 indicates that $\phi = 0.75$. Using equation 33, FS = 1.4167/0.75 = 1.89 and hence the ultimate capacity for the WEAP drivability analysis can be taken as $F_u = 1.89 \times F_d$. If the design load is established via LRFD analysis (i.e., factored design load) than $F_u = F_d/\phi$. In case of scour and/or downdrag, both components should be added to the design load, i.e., F_d + net scour + downdrag. It should be noted that the results of this analysis should <u>not</u> be used for pile capacity prediction in the field. Table 27 provides resistance factors that should be used at EOD if WEAP analysis is required as a prediction method for pile capacity based on measured blow count. That table also provides resistance factors associated with the anticipated testing method that should be used (in the same manner as described above for static load tests) if dynamic testing is to be performed.

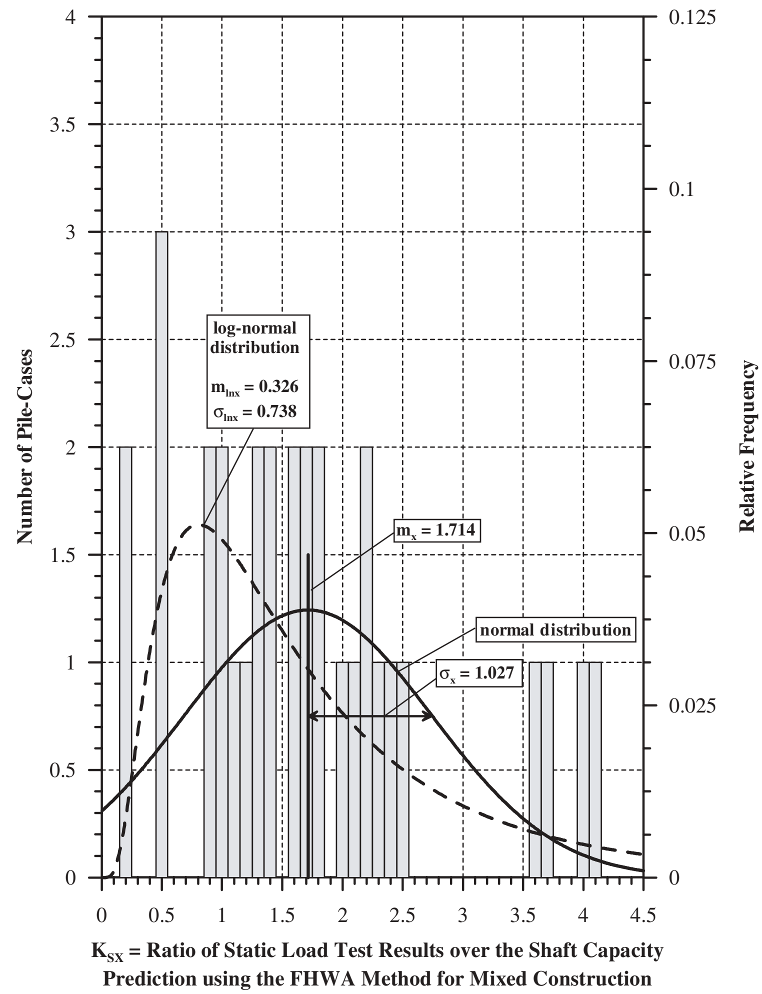


Figure 38. Histogram and frequency distributions for K_{sx} for 32 cases of drilled shafts in sand.

4. The determination of the required number of indicator piles can be combined along with the required number of dynamic pile tests presented in Table 28. For sites at which fewer than 100 piles are driven, the number of indicator piles can be used as the number of tested piles (approximately 8). Specifically, at least one pile should be tested under each substructure, using the test results as outlined in section 3.3.3. When more than 100 piles are driven, particularly at sites of high variability, separation can be made between indicator piles and production piles, with the former used for assigning driving criteria and the latter used for production quality control, as outlined in section 3.3.3. Restrike testing of piles should be scheduled according to equations 37 through 39, as outlined in section 3.4.6.
5. Resistance factors for static load tests of driven piles and drilled shafts should be assigned according to Table 30. The driven pile tests should be scheduled according to equations 37 through 39, as outlined in section 3.4.6.
6. All drilled shafts should be tested using small or high strain integrity testing.

3.5 EVALUATION OF THE RESISTANCE FACTORS

3.5.1 Overview

Evaluation of the recommended resistance factors to be incorporated into a code is a complex and extensive process. The aim of the process is to compare an existing code of practice to the recommended new factors. Very often this evaluation cannot be done directly, as either the principles behind the factors differ (e.g., WSD vs. LRFD), or the applied methodology is not compatible (e.g., the design and construction combined factors of the existing code). As a result, the evaluation can be carried out in two ways:

1. Analyzing design case histories in light of both the new factors and the existing codes. In this way what has been done can be compared with what would have been done; and, if a sufficient number of case histories are analyzed, statistically valid conclusions can be derived regarding the effectiveness and overall performance of the recommendations.

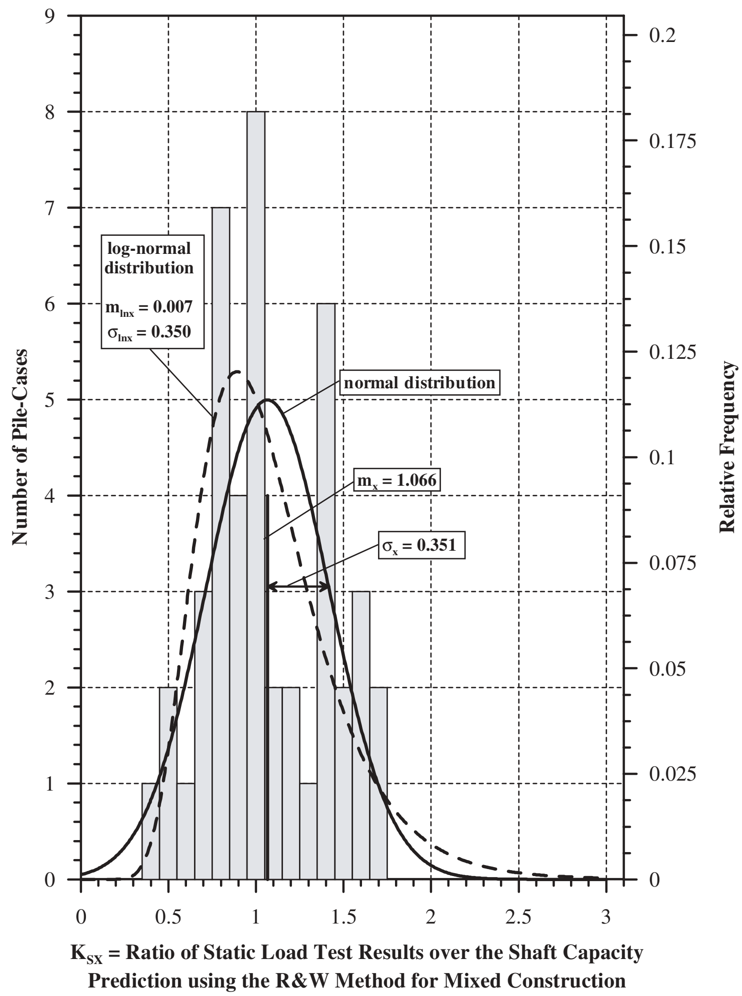


Figure 39. Histogram and frequency distributions for K_{sx} for 44 cases of drilled shafts in sand + clay.

2. Searching for common factors that can be compared, for example establishing a connection between resistance factors and factors of safety (e.g. see section 3.2.3).

The following sections deal with various aspects associated with the recommended factors and means for their evaluation.

3.5.2 Working Stress Design

The traditional factors of safety presented in Table 1 can now be evaluated in light of the available data. For example, the COV for the ENR equation and the WEAP analyses are 0.910 and 0.724, respectively, which practically means that the methods are unsuitable for the purpose of capacity prediction (see Figure 33). The reduction in the factor of safety from 3.50 to 2.75 when adding WEAP analysis to static calculations (as shown in Table 1) is therefore unfounded. Nor does the use of unspecified CAPWAP (general case) justify the reduction of the factor of safety to 2.25, even though the average prediction is conservative and hence the mean case with an FS = 2.25 relates to an overprediction ratio of 3.1 (1.368 × 2.25). In comparison, the use of FS = 2.25 with a specified CAPWAP at the BOR is reasonable and is associated with an acceptable probability of failure for a single pile application (approximately 1.85%; see Figure 32). The use of a large factor of safety for the static analysis appears to be very sensible, as most of the methods overpredict the actual capacity. The WSD existing factor (FS = 3.5) is probably based on historical cumulative experience and matches the presented results without being excessive or wasteful. The data summarized in Figure 45 are used to demonstrate this issue. For example, the average static capacity analysis of a driven pile in clay results in a mean underprediction ratio of about 0.82 and 0.72 for α and λ methods, respectively. The actual factors of safety in theses cases are 2.87 and 2.52. These factors of safety are in good agreement with the actual factor of safety when using the CAPWAP BOR results considering the bias (FS = 2.61; see Figure 45). However, using CAPWAP results at the EOD, considering the bias, results in

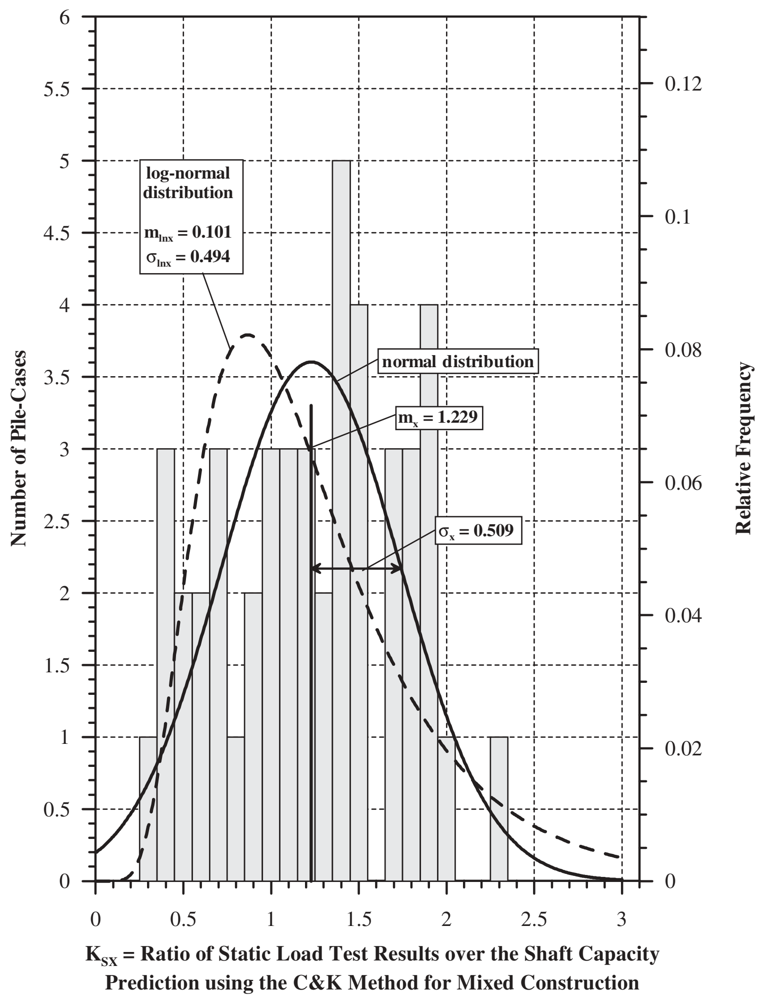


Figure 40. *Histogram and frequency distributions for K_{sx} for 46 cases of drilled shafts in rock.*

a very conservative safety factor (FS = 3.66) relative to the static analysis methods.

The major derived conclusions are therefore

1. The absolute value of a safety measure (factor of safety or resistance factor) by itself does not represent the economics of the method or the progressiveness of the code as suggested in Table 1.
2. An efficiency factor or a similar parameter is required in order to account for the bias of the analysis methods and provide an objective evaluation regarding the effectiveness of the capacity prediction method.
3. Databases are essential to assess any design methodology.
4. The reduction of the factor of safety during design based on the anticipated capacity verification method during construction is unreasonable and unsafe. Specifically, if one uses an FS = 2 for static analysis during design because a static load test is expected to be carried out during construction (see Table 1), the actual mean FS in these cases is about 1.5 (1.45 to 1.62 for α and λ methods, respectively).

3.5.3 Sensitivity Analysis and Factors Evaluation

The existing resistance factors of the AASHTO specifications for dynamic evaluation of driven piles are limited and connected to static evaluation methods. The recommended resistance factors, presented in Table 27, are novel in their approach and categorization. Detailed comparisons between the current AASHTO specifications and those recommended are, therefore, not possible. General comparison between the factors presented in Table 27 and those of other codes (e.g., Australia's) suggests that the proposed resistance factors are comparable.

The resistance factors for static analyses of driven piles, presented in Table 25, can be compared to the existing specifications with the application of the λ_v factor and neglecting the specific method of the recommended values. When compared, the proposed parameters are reasonably in agreement with, but demonstrate the weakness of, the existing specifications.

A sensitivity analysis along with a comparison between the parameters of different sources for static analyses of driven piles is presented in Figures 45 through 52. Figure 45 presents a summary of parameters from the existing LRFD code, the Standard (WSD) AASHTO code and the present recommended values. Figures 46 through 48 present a sensitivity analysis along with a comparison between the factors for selected cases. For example Figure 46 examines the dataset related to pipe piles in clay, analyzed using the α API method. The use of $\phi = 0.7$ for the α method in the existing LRFD AASHTO specifications is apparently based on a database (Barker et al., 1991) and seem to be incompatible with any other source. The dataset for pipe piles in clay (Figure 46) seem to be sensitive to the elimination of the extreme cases as shown by the relations between the resistance factors and target reliability for a set including 20, 19, 18, and 17 cases, associated with all data, data within the two standard deviation zone, 1.5 SD zone, and 1SD zone respectively. When examining the same design method for the databases of concrete piles and H piles, the sensitivity of the exclusion of cases does not exist once the extreme cases beyond the zone of two standard deviations are omitted.

Figures 49 through 52 relate to the analyses of driven piles in sand. The recommended factors seem to vary in relation to the existing FS according to the pile type; matching the existing WSD for pipe piles, while being substantially higher for concrete piles and lower for H piles. This demonstrates the effect of developing parameters with a consistent probability of failure compared to the parameters of the existing methodology. The new parameters may appear depending on the case conservative or unsafe compared to existing standards, while actually being consistent.

The recommended resistance factors for redundant drilled shafts, presented in Table 29, agree overall with those provided by the existing specifications. The categorization by construction methods in mixed subsurface (sand and clay) can be further evaluated in light of local practices. Specifying a construction method before bidding is permitted in some states and not in others. Unspecific bidding specifications eliminate the possibility of a design associated with a specific construction method. The practice of constructing single nonredundant drilled shafts is more common than in the case of driven piles. For nonredundant drilled shafts, the recommended resistance factors are lower than the common practice and need to be further evaluated in light of the possible consequences of failure.

3.5.4 Actual Probability of Failure

One advantage of using a large database is that the probability of failure (or the risk) can be directly calculated from the available data, rather than by using the calculated distribution function. The procedure is done by applying a certain resistance factor to the calculated resistance (capacity) and examining the number of cases that exceed the actual capacity (nominal strength). An example of the process as applied to some of the dynamic methods is presented in Table 31. It should be noted that the values presented in Table 31 are conservative, as a comprehensive calculation should account for the load factors (on the order of 1.35 depending on the DL to LL ratio); hence further decrease the probability of failure values provided in Table 31. The data in Table 31 suggests that the recommended factors presented in Table 27 would result in target reliabilities higher (lower p_f) than those calculated for using the distribution functions.

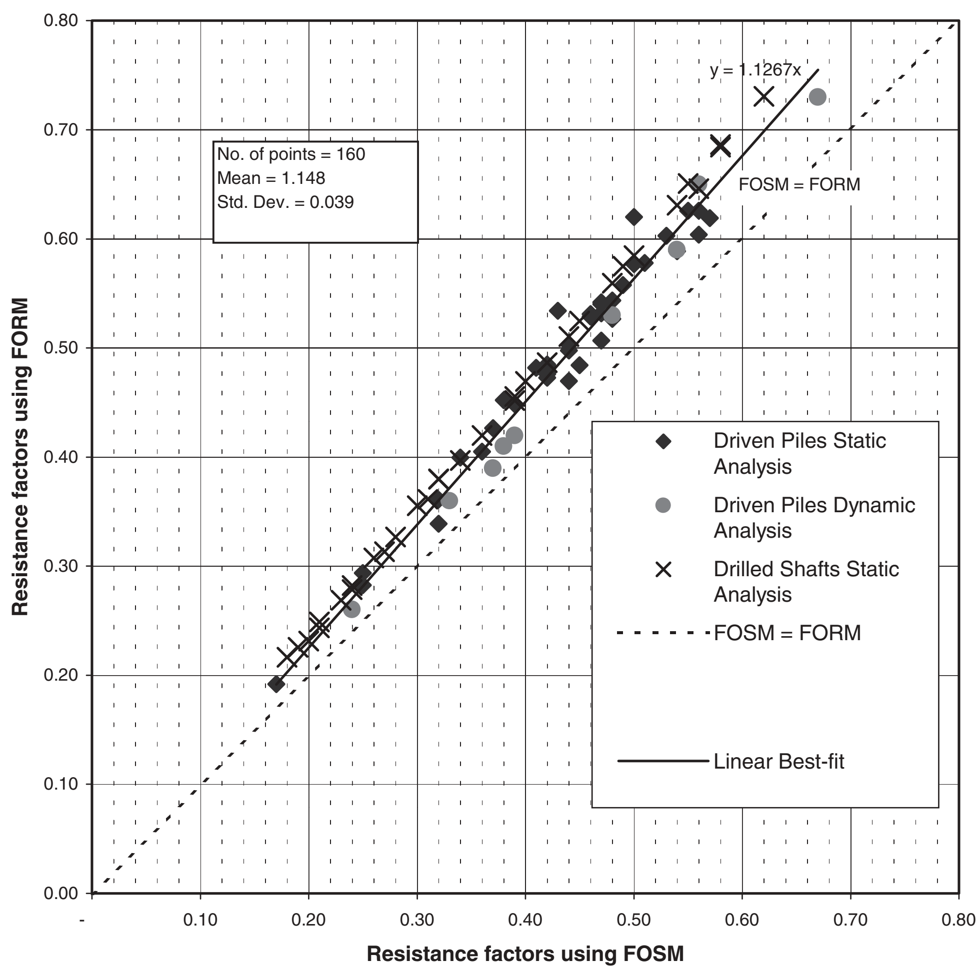


Figure 41. Comparison between resistance factors obtained using the First Order Second Moment (FOSM) vs. those obtained by using First Order Reliability Method (FORM) for a target reliability of $\beta = 2.33$.

TABLE 19 Statistical details of static analyses of driven piles, resistance factors, efficiency factors, equivalent and "actual" factors of safety

Soil Type	Pile Type	N Total No. of Cases	Design Method[1]	Details of Method[2] Application	No. of Cases ± 2 SD	Mean (λ)	COV	β=2.33				β=3.00			
								ϕ	ϕ/λ	FS	FS x λ	ϕ	ϕ/λ	FS	FS x λ
Clay	H-Piles	4	β-Method	11.5 B;T&P(2)	4	0.61	0.61	0.19	0.32	7.34	4.48	0.13	0.22	10.63	6.48
		17	λ-Method	11.5B; T&P(2) 2B; T&P(5)	16	0.74	0.39	0.37	0.50	3.80	2.82	0.29	0.39	4.97	3.68
		17	α-Tomlinson	2B; T&P(2)	17	0.82	0.40	0.40	0.49	3.51	2.88	0.31	0.37	4.61	3.78
		17	α-API	2B; T&P(5)	16	0.90	0.41	0.43	0.48	3.26	2.93	0.33	0.37	4.30	3.87
		9	SPT-97 mob		8	1.04	0.41	0.50	0.48	2.84	2.95	0.38	0.36	3.74	3.89
	Concrete Piles	19	λ-Method	2B; Hara (5h)	18	0.76	0.29	0.48	0.63	2.97	2.26	0.38	0.51	3.69	2.80
		19	α-API	2B; Hara (5h)	17	0.81	0.26	0.54	0.67	2.61	2.11	0.44	0.55	3.20	2.59
		8	β-Method	2B; Hara (5h)	8	0.81	0.51	0.32	0.39	4.45	3.60	0.23	0.28	6.14	4.97
		19	α-Tomlinson	2B; Hara (5h)	18	0.87	0.48	0.36	0.41	3.94	3.43	0.26	0.30	5.37	4.67
	Pipe Piles	20	α-Tomlinson	2B; T&P (1)	18	0.64	0.50	0.25	0.40	5.56	3.56	0.19	0.29	7.64	4.89
		20	α-API	2B; T&P (1)	19	0.79	0.54	0.29	0.36	4.95	3.91	0.20	0.26	6.96	5.50
		13	β-Method	2B; T&P (1)	12	0.45	0.60	0.14	0.32	9.81	4.41	0.10	0.22	14.16	6.37
		20	λ-Method	2B; T&P (1)	19	0.67	0.55	0.24	0.36	5.94	3.98	0.17	0.25	8.38	5.62
		13	SPT-97 mob	2B; T&P (1)	12	0.39	0.62	0.12	0.31	11.70	4.56	0.08	0.21	17.02	6.64
Sand	H-Piles	19	Nordlund	36; 11.5B,P(6)	19	0.94	0.40	0.46	0.49	3.08	2.89	0.35	0.37	4.04	3.80
		19	Meyerhof		18	0.81	0.38	0.42	0.51	3.41	2.76	0.32	0.39	4.43	3.59
		19	β-Method	36; 2B; P(5)	19	0.78	0.51	0.30	0.39	4.69	3.66	0.22	0.28	6.49	5.06
		19	SPT-97 mob		18	1.35	0.43	0.63	0.46	2.26	3.05	0.47	0.35	3.01	4.06
	Concrete Piles	37	Nordlund	36: 11.5B; P(6)	36	1.02	0.48	0.42	0.42	3.34	3.41	0.31	0.31	4.55	4.64
		37	β-Method	36; 2B; P(5)	35	1.10	0.44	0.50	0.46	2.82	3.10	0.38	0.34	3.76	4.13
		37	Meyerhof		36	0.61	0.61	0.19	0.32	7.34	4.48	0.13	0.22	10.63	6.48
		37	SPT97 mob		36	1.21	0.47	0.51	0.42	2.76	3.34	0.38	0.31	3.75	4.53
	Pipe Piles	20	Nordlund	36; 2B P(5)	19	1.48	0.52	0.56	0.38	2.51	3.71	0.41	0.27	3.49	5.16
		20	β-Method	36; 2B P(5)	20	1.18	0.62	0.36	0.31	3.89	4.59	0.25	0.21	5.67	6.69
		20	Meyerhof		20	0.94	0.59	0.31	0.33	4.55	4.27	0.22	0.23	6.52	6.13
		20	SPT-97 mob		19	1.58	0.52	0.60	0.38	2.34	3.70	0.44	0.28	3.26	5.14
Mixed Soils	H-Piles	22	α-Tomlinson/Nordlund/Thurman	36; 2B; P(5)	20	0.59	0.39	0.30	0.51	4.75	2.80	0.23	0.39	6.20	3.66
		37	α-API/Nordlund/Thurman	36; 2B; P(5)	34	0.79	0.44	0.36	0.45	3.98	3.14	0.27	0.34	5.33	4.21
		35	β-Method/Thurman	36; 2B; P(5)	32	0.48	0.48	0.20	0.42	7.08	3.40	0.15	0.31	9.65	4.63
		41	SPT-97		40	1.23	0.45	0.55	0.45	2.58	3.17	0.41	0.33	3.46	4.25
	Concrete Piles	34	α-Tomlinson/Nordlund/Thurman	36; 2B; P; Hara(5h)	33	0.96	0.49	0.39	0.41	3.62	3.48	0.29	0.30	4.96	4.76
		85	α-API/Nordland/Thurman	36; 11.5B; Sch; T&P(8)	80	0.87	0.48	0.36	0.41	3.94	3.43	0.26	0.30	5.37	4.67
		85	β-Method/Thurman	36; 11.5B; Sch; T&P(8)	80	0.81	0.38	0.42	0.51	3.41	2.76	0.32	0.39	4.43	3.59
		74	SPT-97 mob		71	1.81	0.50	0.72	0.40	1.98	3.58	0.52	0.29	2.72	4.93
		32	FHWA CPT		30	0.84	0.31	0.51	0.60	2.81	2.36	0.40	0.48	3.52	2.96
	Pipe Piles	13	α-Tomlinson/Nordlund/Thurman	36; 2B; P(5)	13	0.74	0.59	0.24	0.32	5.89	4.36	0.17	0.23	8.49	6.28
		34	α-API/Nordland/Thurman	36; 2B; P(5)	32	0.80	0.45	0.36	0.44	3.99	3.19	0.26	0.33	5.36	4.29
		31	β-Method/Thurman	36; 2B; P(5)	29	0.54	0.48	0.22	0.41	6.33	3.42	0.16	0.30	8.63	4.66
		34	SPT-97 mob		33	0.76	0.38	0.39	0.51	3.62	2.75	0.30	0.40	4.71	3.58

(1)See Table 6 for details;
(2)Numbers in parentheses refer to notations used for detailing soil parameter combinations (see Table 7b and Appendix C for more details), See Tables 7a and 8 for soil properties' correlations to SPT and CPT respectively, 36 = limiting friction angle, B = pile diameter 2B, 11.5B contributing zone to tip resistance.

TABLE 20 Statistical details of dynamic analyses of driven piles, resistance factors, efficiency factors, equivalent and "actual" factors of safety

Method		Time of Driving	No. of Cases	Mean (λ)	COV	$\beta = 2.33$				$\beta = 3.0$			
						ϕ	ϕ/λ	F.S.	F.S.xλ	ϕ	ϕ/λ	F.S	F.S.xλ
Dynamic Measurements	CAPWAP	General	377	1.368	0.453	0.59	0.43	2.40	3.28	0.43	0.31	3.29	4.51
		EOD	125	1.626	0.490	0.64	0.40	2.21	3.60	0.46	0.28	3.08	5.01
		EOD - AR < 350 & Bl. Ct. < 16 BP10cm	37	2.589	0.921	0.41	0.16	3.46	8.95	0.23	0.09	6.16	15.95
		BOR	162	1.158	0.339	0.65	0.56	2.18	2.52	0.51	0.44	2.78	3.22
	Energy Approach	General	371	0.894	0.411	0.42	0.47	2.52	2.26	0.32	0.36	4.43	3.96
		EOD	128	1.084	0.398	0.53	0.49	2.67	2.91	0.40	0.37	3.54	3.84
		EOD - AR < 350 & Bl. Ct. < 16 BP10cm	39	1.431	0.508	0.54	0.38	2.62	3.75	0.39	0.27	3.63	5.20
		BOR	153	0.785	0.369	0.41	0.52	3.46	2.71	0.32	0.41	4.43	3.48
Dynamic Equations	ENR	General	384	1.602	0.910	0.26	0.16	5.45	8.73	0.15	0.09	9.45	15.13
	Gates	General	384	1.787	0.475	0.73	0.41	1.94	3.47	0.53	0.30	2.67	4.78
	FHWA modified Gates	General	384	0.940	0.502	0.36	0.38	3.94	3.70	0.26	0.38	5.45	5.12
		EOD	135	1.073	0.534	0.38	0.36	3.73	4.00	0.27	0.25	5.25	5.63
		EOD Bl. Ct. < 16BP10cm	62	1.306	0.492	0.51	0.39	2.78	3.63	0.37	0.28	3.83	5.00
WEAP		EOD	99	1.656	0.724	0.39	0.24	3.63	6.02	0.25	0.24	5.67	9.38

Notes: Column heads: Mean = ratio of the static load test results (Davisson's Criterion) to the predicted capacity = $K_{sx} = \lambda$ = bias;
COV = Coefficient of Variation
Methods: ENR = Engineering News Record Equation
Time of Driving: EOD = end of driving; BOR = beginning of restrike; AR = area ratio; Bl. Ct. = blow count;
BP10cm = blows per 10cm

TABLE 21 Statistical details of static analyses of drilled shafts, resistance factors, efficiency factors, equivalent and "actual" factors of safety

Capacity Component	Soil Type	N Total No. of Cases	Design Method	Const. Method	No. of Cases ± 2 SD	Mean (λ)	COV	β = 2.33				β = 3.0			
								φ	φ/λ	F.S.	F.S. x λ	φ	φ/λ	F.S.	F.S. x λ
Skin Friction + End Bearing	Sand	34	FHWA	Mixed	32	1.71	0.60	0.55	0.32	2.58	4.41	0.38	0.22	3.73	6.37
		14		Casing	12	2.27	0.46	0.99	0.43	1.44	3.26	0.73	0.32	1.94	4.40
		14		Slurry	9	1.62	0.74	0.38	0.24	3.69	5.97	0.25	0.15	5.70	9.23
		34	R&W	Mixed	32	1.22	0.67	0.34	0.28	4.21	5.13	0.23	0.18	6.29	7.67
		14		Casing	12	1.45	0.50	0.58	0.40	2.45	3.56	0.42	0.29	3.37	4.89
		14		Slurry	9	1.32	0.62	0.41	0.31	3.49	4.61	0.28	0.21	5.09	6.72
	Clay	54	FHWA	Mixed	53	0.90	0.47	0.38	0.43	3.70	3.33	0.28	0.31	5.02	4.52
		14		Casing	13	0.84	0.50	0.33	0.40	4.23	3.56	0.24	0.29	5.82	4.89
		40		Dry	40	0.88	0.48	0.37	0.42	3.87	3.41	0.27	0.31	5.27	4.64
	Sand + Clay	48	FHWA	Mixed	44	1.19	0.30	0.73	0.61	1.94	2.31	0.58	0.49	2.42	2.88
		23		Casing	21	1.04	0.29	0.65	0.63	2.17	2.26	0.52	0.50	2.70	2.81
		13		Dry	12	1.32	0.28	0.85	0.64	1.67	2.21	0.68	0.52	2.07	2.73
		12		Slurry	10	1.29	0.27	0.84	0.65	1.68	2.16	0.69	0.53	2.06	2.66
		48	R&W	Mixed	44	1.09	0.35	0.60	0.55	2.36	2.57	0.47	0.43	3.02	3.29
		23		Casing	21	1.01	0.42	0.48	0.47	2.96	2.99	0.36	0.36	3.92	3.96
		13		Dry	12	1.20	0.32	0.71	0.59	2.01	2.41	0.56	0.47	2.53	3.04
		12		Slurry	10	1.16	0.25	0.79	0.68	1.79	2.07	0.65	0.56	2.18	2.53
	Rock	49	C&K	Mixed	46	1.23	0.41	0.60	0.48	2.38	2.93	0.45	0.37	3.13	3.86
		32		Dry	29	1.29	0.40	0.64	0.49	2.22	2.86	0.49	0.38	2.91	3.76
		49	IGM	Mixed	46	1.30	0.34	0.73	0.56	1.94	2.52	0.57	0.44	2.46	3.20
		32		Dry	29	1.35	0.31	0.81	0.60	1.75	2.36	0.65	0.48	2.19	2.96
Skin	Sand	11	FHWA	Mixed	11	1.09	0.51	0.43	0.39	3.33	3.63	0.31	0.28	4.61	5.02
		11	R&W	Mixed	11	0.83	0.54	0.30	0.37	4.67	3.88	0.22	0.26	6.55	5.44
	Clay	16	FHWA	Mixed	13	0.87	0.37	0.46	0.53	3.09	2.69	0.36	0.41	3.99	3.47
	Sand + Clay	16	FHWA	Mixed	14	1.25	0.29	0.78	0.63	1.81	2.26	0.63	0.50	2.25	2.81
		16	R&W	Mixed	14	1.24	0.41	0.60	0.48	2.36	2.93	0.46	0.37	3.11	3.86
	All Soils	40	FHWA	Mixed	39	1.08	0.41	0.52	0.48	2.71	2.93	0.40	0.37	3.57	3.86
		27	R&W	Mixed	25	1.07	0.48	0.45	0.42	3.18	3.41	0.33	0.31	4.34	4.64
	Rock	17	C&K	Mixed	16	1.18	0.46	0.51	0.43	2.76	3.26	0.38	0.32	3.73	4.40
		17	IGM	Mixed	16	1.25	0.37	0.66	0.53	2.15	2.69	0.51	0.41	2.78	3.47

TABLE 22 Resistance factors and associated factors of safety along with efficiency measures for sample methods

Category	Pile Type or Construction	Soil Type or State	Method of Analysis	$\beta = 2.33$ $\beta = 3.00$ $\gamma_L = 1.75$, $\gamma_D = 1.2$, DL/LL = 2			
				ϕ resistance factor	ϕ/λ efficiency	FS factor of safety	FS x λ actual mean FS
Static Methods Driven Piles	PPC	Clay	α - API	0.54 0.44	0.67 0.55	2.61 3.20	2.11 2.59
	PPC	Sand	β	0.50 0.38	0.46 0.34	2.82 3.76	3.10 4.13
	Pipe	Mixed	α - API Nordlund/Thurman	0.36 0.26	0.44 0.33	3.99 5.36	3.19 4.29
Dynamic Methods Driven Piles	All	BOR	CAPWAP	0.65 0.51	0.56 0.44	2.18 2.78	2.52 3.22
	All	EOD	Energy Approach	0.53 0.40	0.49 0.37	2.67 3.54	2.91 3.84
	All	EOD	FHWA mod Gates	0.38 0.27	0.36 0.25	3.73 5.25	4.00 5.63
Static Methods Drilled Shafts	Mixed	All	R&W skin	0.45 0.33	0.42 0.31	3.18 4.34	3.41 4.64
	Mixed	Rock	C&K total	0.60 0.45	0.48 0.37	2.38 3.13	2.93 3.86
	Mixed	Sand & Clay	FHWA skin	0.78 0.63	0.63 0.50	1.81 2.25	2.26 2.81

Notes: *Top line of column: $\beta = 2.33$; **Bottom line of column: $\beta = 3.00$; $\gamma_L = 1.75$; $\gamma_D = 1.2$; DL/LL = 2.

TABLE 23 Detailed resistance factors for pullout of driven piles—based on static analyses

Pile Type	Soil Type	Design Method	No.	λ	COV	Resistance Factor		ϕ/λ	
						Redundant $\beta = 2.33$	Non-redundant $\beta = 3.00$	Redundant $\beta = 2.33$	Non-redundant $\beta = 3.00$
Pipe	Clay	α-API	9	1.11	0.71	0.28	0.18	0.25	0.16
		α-Tomlinson	9	0.95	0.57	0.33	0.23	0.35	0.24
		λ-Method	9	0.72	0.52	0.27	0.20	0.38	0.36
	Sand and Mixed	β-Method	7	0.52	0.54	0.19	0.14	0.37	0.27
		SPT-97 mob	7	1.18	1.33	0.08	0.04	0.07	0.03
		αAPI/Nordlund	7	0.80	0.60	0.26	0.18	0.33	0.23
H	Clay	α-API	3	0.76	0.57	0.26	0.18	0.34	0.24
		α-Tomlinson	3	0.64	0.54	0.23	0.17	0.36	0.27
	Sand	β-Method	8	0.23	0.36	0.12	0.10	0.52	0.43
		SPT-97 mob	8	0.43	0.32	0.25	0.20	0.58	0.47

TABLE 24 Resistance factors as a function of number of load tests per site, site variability and target reliability

Site Variation	N	Mean (λ) (Bias)	SD	C.O.V.	Target Reliability β		
					2.00	2.33	3.00
Low	1	1	0.18	0.18	0.86	0.80	0.67
	2	1	0.13	0.13	0.96	0.89	0.78
	3	1	0.10	0.10	1.00	0.94	0.83
	4	1	0.09	0.09	1.03	0.97	0.86
	5	1	0.08	0.08	1.04	0.99	0.88
Medium	1	1	0.27	0.27	0.73	0.65	0.53
	2	1	0.19	0.19	0.85	0.78	0.66
	3	1	0.16	0.16	0.90	0.84	0.72
	4	1	0.14	0.14	0.94	0.88	0.76
	5	1	0.12	0.12	0.97	0.90	0.79
High	1	1	0.36	0.36	0.61	0.54	0.42
	2	1	0.25	0.25	0.75	0.68	0.55
	3	1	0.21	0.21	0.82	0.75	0.63
	4	1	0.18	0.18	0.86	0.80	0.67
	5	1	0.16	0.16	0.90	0.83	0.71

Note: N = Number of load tests

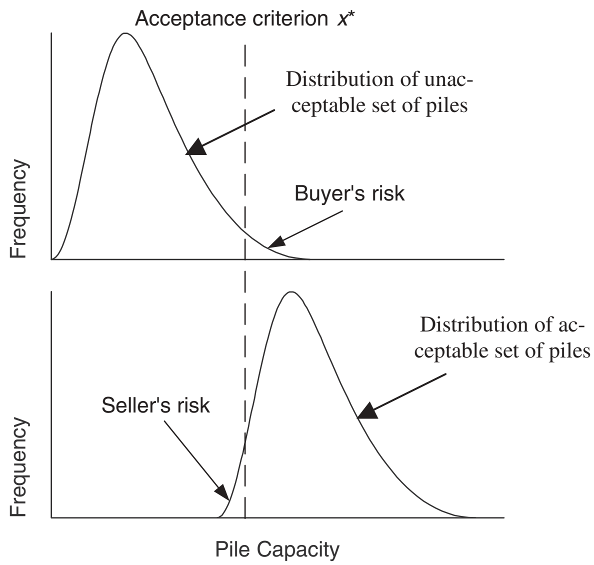


Figure 42. Frequency distributions of test results taken from sets of unacceptable and acceptable piles, showing contractor's (seller's) and owner's (buyer's) risks (schematic).

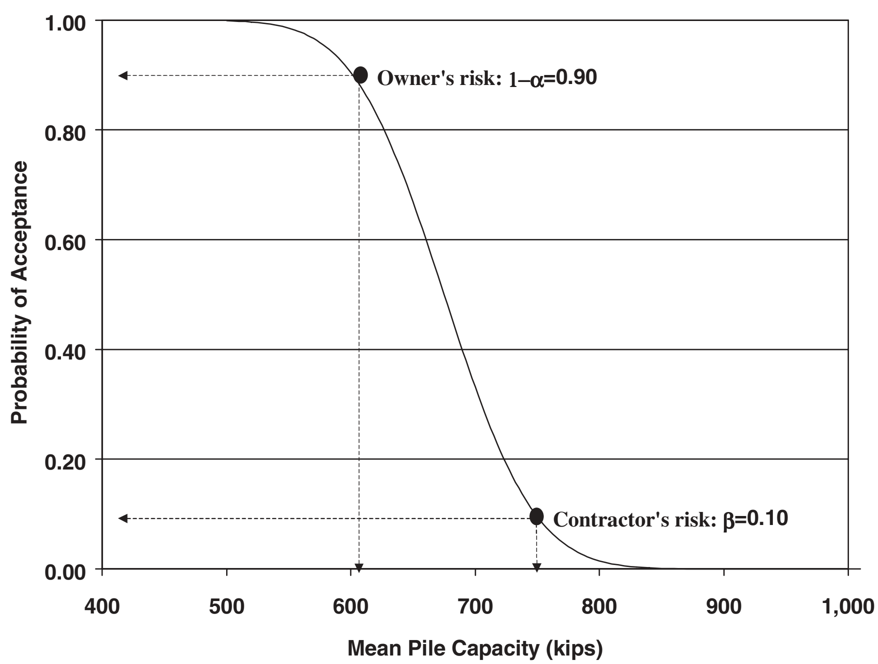


Figure 43. Operating characteristics curve for an acceptance sampling plan to ensure the average axial capacity of a set of piles.

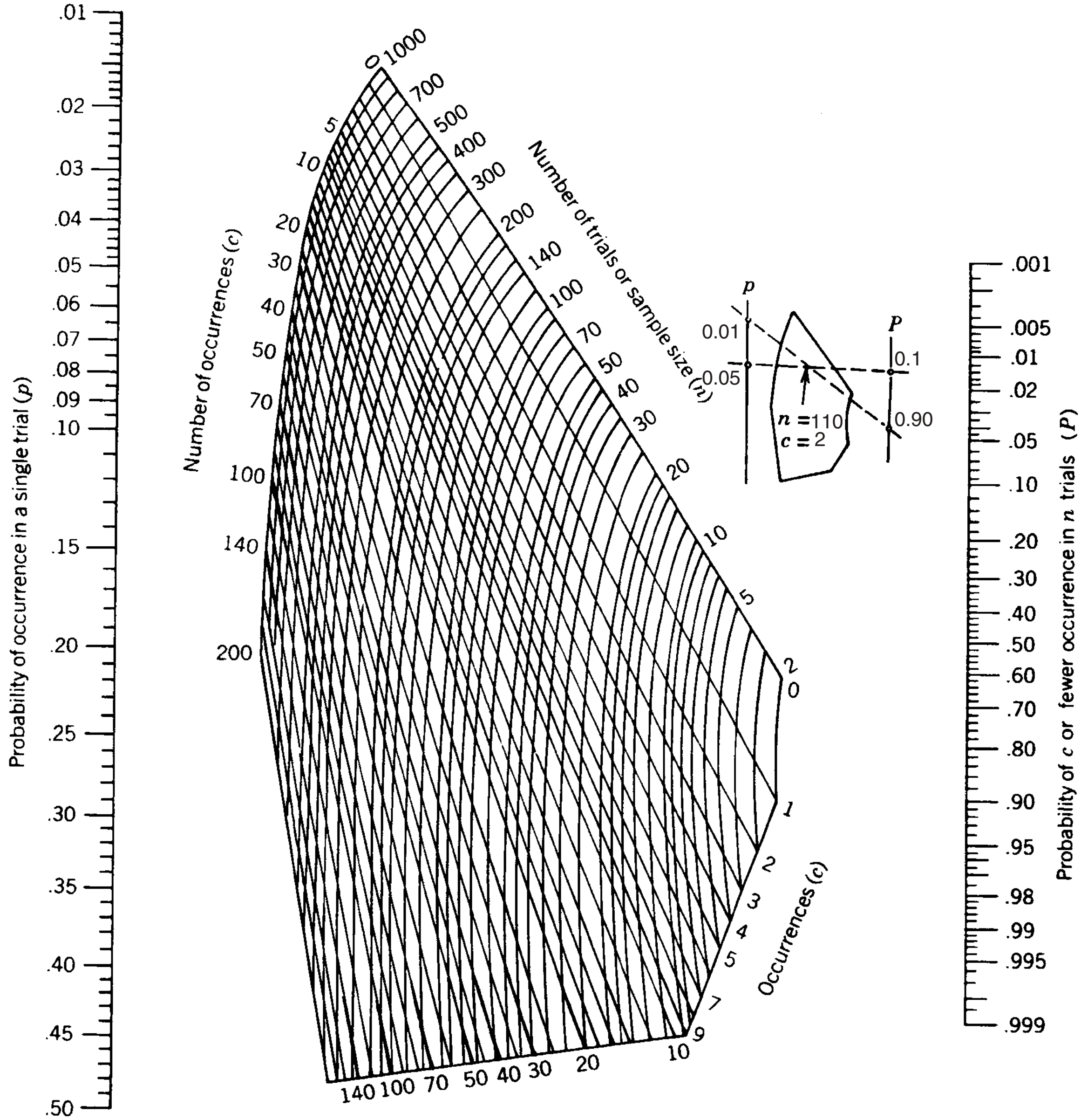


Figure 44. Binomial nomograph for determining sample size, n, *and permitted number of defectives,* c, *for contractor's risk* α *and owner's risk* β *(Montgomery 1991). The procedure for using the nomograph to design a sampling plan is to (1) draw a line connecting* α *on the right-hand rule with the corresponding* p_1 *on the left-hand rule, (2) draw a similar line connecting (1-*β*) and* p_2*, and (3) the point of intersection of the two lines gives the required sample size, n, and the maximum number of defectives permitted within the sample for acceptance.*

TABLE 25 Recommended resistance and efficiency factors for static analyses of driven piles

Pile Type	Soil Type	Design Method	Resistance Factor ϕ		ϕ/λ	
			Redundant	Non-redundant	Redundant	Non-redundant
Concrete Pile	Mixed	SPT-97 mob	0.70	0.50	0.40	0.29
	Clay	α-API	0.50	0.40	0.67	0.55
		λ-Method			0.63	0.55
	Sand	β-Method			0.46	0.34
		SPT-97 mob			0.42	0.31
	Mixed	FHWA CPT			0.60	0.48
		β-Method/Thurman	0.40	0.30	0.51	0.39
		αTomlinson/Nordlund/Thurman			0.41	0.30
	Sand	Nordlund			0.42	0.31
	Clay	α-Tomlinson	0.35	0.25	0.41	0.30
	Mixed	α-API/Nordlund/Thurman			0.41	0.30
	Sand	Meyerhof	0.20	0.15	0.32	0.22
Pipe Pile	Sand	SPT-97 mob	0.55	0.45	0.38	0.28
		Nordlund			0.38	0.27
	Mixed	SPT-97 mob	0.40	0.30	0.51	0.40
		α-API/Nordlund/Thurman	0.35	0.25	0.44	0.31
	Sand	β-Method			0.31	0.21
	Clay	α-API	0.30	0.20	0.36	0.26
	Sand	Meyerhof			0.33	0.23
	Mixed	αTomlinson/Nordlund/Thurman	0.25	0.15	0.32	0.23
		β-Method/Thurman			0.41	0.30
	Clay	α-Tomlinson			0.40	0.29
		λ-Method			0.36	0.25
H Piles	Mixed	SPT-97 mob	0.55	0.45	0.45	0.33
	Sand	SPT-97 mob			0.46	0.35
		Nordlund	0.45	0.35	0.49	0.37
		Meyerhof			0.51	0.39
	Clay	α-API			0.48	0.37
		α-Tomlinson	0.40	0.30	0.49	0.37
		λ-Method			0.50	0.39
	Mixed	α-API/Nordlund/Thurman	0.35	0.25	0.45	0.34
		αTomlinson/Nordlund/Thurman	0.30		0.51	0.39
	Sand	β-Method			0.39	0.28
	Mixed	β-Method/Thurman	0.20	0.15	0.42	0.31

Notes: ϕ/λ = efficiency factor, evaluating the relative economic performance of each method (higher ratios indicate a more economical solution).
λ = bias = K_{sx} = Mean of measured over predicted.
ϕ/λ values relate to the exact calculated ϕ and λ and not to the assigned ϕ values in the table
Redundant = Five piles or more under one pile cap ($\beta = 2.33$ $p_f = 1.0\%$)
Non-Redundant = Four or fewer piles under one pile cap ($\beta = 3.0$ $p_f = 0.1\%$)

TABLE 26 Recommended resistance factors for static analysis of nontapered driven piles under pullout

Soil Type	Design Method	Pile Type	ϕ (resistance factor)	
			Redundant $\beta = 2.33$	Non-Redundant $\beta = 3.00$
Clay	α-API, λ αTomlinson	H, Pipe, PPC	0.25[1]	0.20
Sand	β	H	0.15	0.10
		Pipe, PPC	0.25	0.20
	SPT-97	H, Pipe, PPC	0.25	0.20
Mixed	α-API/Nordlund	H, Pipe, PPC	0.20	0.15

[1]Higher values may be applicable for PPC piles but no sufficient data were available to support this.

TABLE 27 Recommended resistance and efficiency factors for dynamic analyses of driven piles

Method		Case	ϕ (resistance factor)		ϕ/λ	
			Redundant	Non-Redundant	Redundant	Non-Redundant
Dynamic Measurements	Signal Matching (CAPWAP)	EOD	0.65	0.45	0.40	0.28
		EOD, AR<350, Bl. Ct.<16BP10cm	0.40	0.25	0.16	0.09
		BOR	0.65	0.50	0.56	0.44
	Energy Approach	EOD	0.55	0.40	0.49	0.37
		BOR	0.40	0.30	0.52	0.41
Dynamic Equations	ENR	General	0.25	0.15	0.16	0.09
	Gates	General	0.75	0.55	0.41	0.30
	FHWA modified	General	0.40	0.25	0.38	0.28
WEAP		EOD	0.40	0.25	0.24	0.15

Notes: COV = Coefficient of Variation
Column heads: ϕ/λ = efficiency factor, evaluating the relative economic performance of each method (higher ratios indicate a more economical solution); ϕ/λ values relate to the exact calculated ϕ and λ and not to the assigned ϕ values in the table; Redundant = Five piles or more under one pile cap.($\beta = 2.33$ $p_f = 1.0\%$); λ = bias = K_{SX} = Mean of measured/predicted; Non-Redundant = Four or less piles under one pile cap ($\beta = 3.0$ $p_f = 0.1\%$)
Method: ENR = Engineering News Record Equation.
Case: EOD = End of Driving; BOR = Beginning of Restrike; AR = Area ratio; Bl.Ct. = blow count; BP10cm = blows per 10cm

TABLE 28 Recommended number of dynamic tests to be conducted during production

Site Variability.		Low		Medium		High	
No. of Piles	Method	EA	CAPWAP	EA	CAPWAP	EA	CAPWAP
	Time	EOD	BOR	EOD	BOR	EOD	BOR
≤ 15		4	3	5	4	6	6
16 - 25		5	3	6	5	9	8
26 - 50		6	4	8	6	10	9
51 – 100		7	4	9	7	12	10
101 – 500		7	4	11	7	14	12
> 500		7	4	12	7	15	12

Notes: Site variability – see section 3.4.3, item 4 for the determination of site variability.
EA = Energy Approach Analysis; CAPWAP = Signal Matching Analysis;
EOD = End of Driving; BOR = Beginning of Restrike

TABLE 29 Recommended resistance factors for drilled shafts

Shaft Resistance	Soil Type	Design Method	Construction Method	ϕ (resistance Factors)		ϕ/λ	
				Redundant	Non-Redundant	Redundant	Non-Redundant
Total Resistance	Sand	R&W	All	0.50	0.40	0.36	0.29
		FHWA				0.38	0.31
	Clay	FHWA	All	0.40	0.30	0.43	0.31
	Sand + Clay	FHWA	Slurry & Dry	0.85	0.70	0.63	0.52
			Casing	0.65	0.50	0.63	0.52
		R&W	Slurry & Dry	0.75	0.60	0.65	0.52
			Casing	0.50	0.35	0.47	0.36
	Rock	C&K	All	0.60	0.60	0.48	0.37
		IGM	All	0.75	0.75	0.56	0.44
Skin Resistance	All Soils	FHWA	All	0.45	0.35	0.48	0.40
		R&W				0.42	0.33
	Rock	C&K	All	0.50	0.35	0.43	0.32
		IGM		0.65	0.50	0.53	0.41

Notes: ϕ/λ = efficiency factor, evaluating the relative economic performance of each method (higher ratios indicate a more economical solution); ϕ/λ values relate to the exact calculated ϕ and λ and not to the assigned ϕ values in the table.
Redundant = Five piles or more under one pile cap ($\beta = 2.33$ $p_f = 1.0\%$)
Non-Redundant = Four or fewer piles under one pile cap ($\beta = 3.0$ $p_f = 0.1\%$)
λ = bias = K_{SX} = mean of measured/predicted
FHWA = Reese and O'Neill (1988); R&W = Reese and Wright (1977);
C&K = Carter and Kulhawy (1988); IGM = O'Neill and Reese (1999).

TABLE 30 Recommended resistance factors for static load tests

ϕ (Resistance Factor)			
No. of Load Tests Per Site	Site Variability		
	Low	Medium	High
1	0.80	0.70	0.55
2	0.90	0.75	0.65
3	0.90	0.85	0.75
≥ 4	0.90	0.90	0.80

Note: Site variability: see section 3.4.3 item 4 for the determination of site variability.

Recommended ϕ values for $\beta = 2.33$

Pile Type	Method		
	α	α	λ
Concrete	0.50	0.35	0.50
Pipe	0.30	0.25	0.25
H	0.45	0.40	0.40

1. Suggest to omit β Method in clay. Not considered Nordlund in clay
2. FHWA CPT mixed soil concrete piles $\phi = 0.50$

Existing LRFD ϕ values

α Method	0.70 λv
λ Method	0.55 λv
β Method and Nordlund applied for clay	0.50 λv
End Bearing Skempton	0.70 λv
WSD	FS = 3.5

No./Mean of Prediction (data $\pm$ 2 SD)

Pile Type	Method					
	α API		α Tomlinson		λ	
Concrete	17	0.81	18	0.87	18	0.76
Pipe	19	0.79	18	0.64	19	0.67
H	16	0.90	17	0.82	16	0.74
Total	52	0.83	51	0.81	53	0.72

Actual Mean FS for driven piles in clay

α Method = 0.82 x 3.5 = 2.87
λ Method = 0.72 x 3.5 = 2.52

For Comparison
CAPWAP - EOD 126 cases Mean = 1.63
BOR 162 Mean = 1.16

Actual FS EOD = 1.63 x 2.25 = 3.66
Actual FS BOR = 1.16 x 2.25 = 2.61

Figure 45. Data summary for parameter evaluation of driven piles in clay.

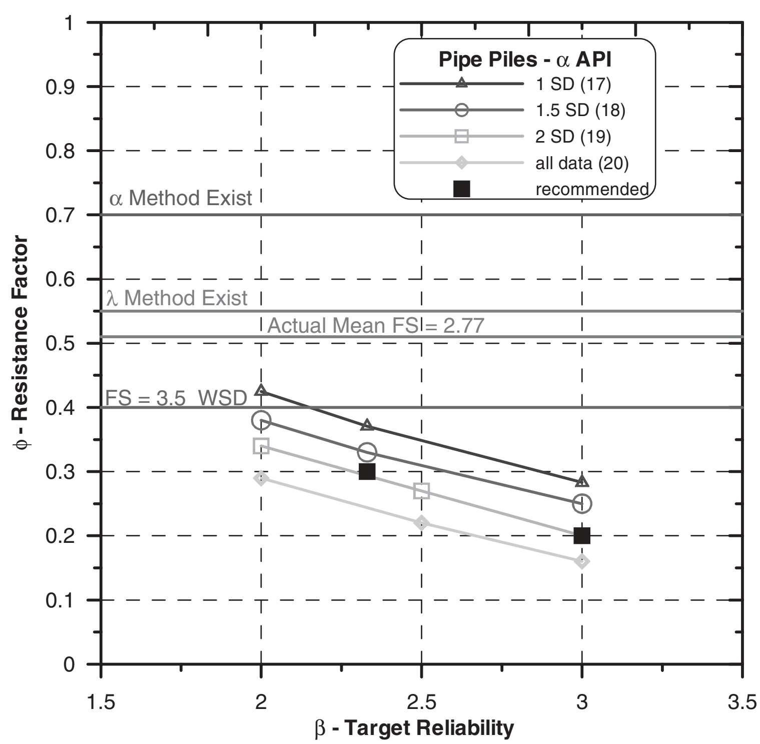


Figure 46. Sensitivity analysis examining the recommended parameters for the design of pipe piles in clay using α API method.

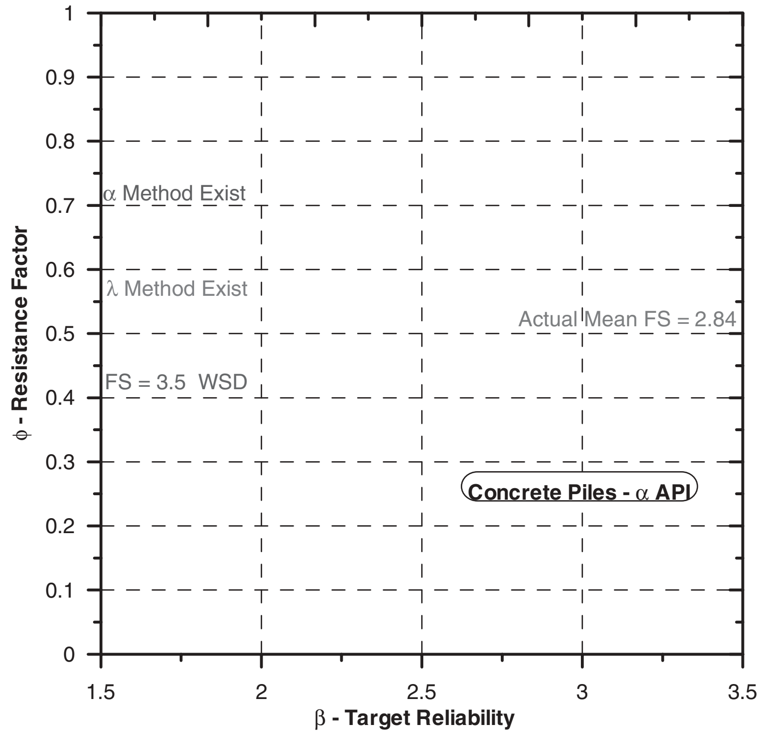


Figure 47. Sensitivity analysis examining the recommended parameters for the design of concrete piles in clay using α *API method.*

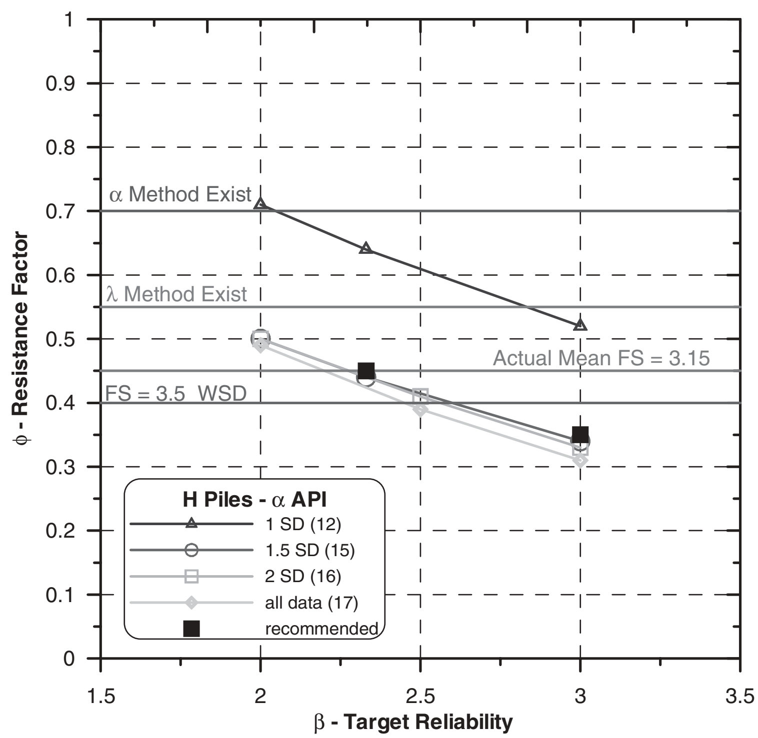


Figure 48. Sensitivity analysis examining the recommended parameters for the design of H piles in clay using α *API method.*

Recommended ϕ values for $\beta = 2.33$

Pile Type	Method			
	Nordlund	Meyerhof	β	SPT 97
Concrete	0.40	0.20	0.5	0.50
Pipe	0.55	0.30	0.3	0.55
H	0.45	0.45	0.3	0.55

FHWA CPT mixed soil concrete piles $\phi = 0.50$

Existing LRFD ϕ values skin and end bearing

CPT	$0.55\ \lambda v$
SPT	$0.45\ \lambda v$
WSD	FS = 3.5

Actual mean FS for driven piles in sand

Range:
pipe piles SPT 97 = 1.58 x 3.5 = 5.53
Meyerhof PPC = 0.61 x 3.5 = 2.14

No./Mean of prediction (data ± 2 SD)

Pile Type	Method							
	Nordlund		Meyerhof		β		SPT 97	
Concrete	36	1.02	36	0.61	35	1.10	36	1.21
Pipe	19	1.48	20	0.94	20	1.18	19	1.58
H	19	0.94	18	0.81	19	0.78	18	1.35
Total	74	1.18*	74	0.75*	74	1.04*	73	1.34*

* - large variation between pile types

Figure 49. Data summary for parameter evaluation of driven piles in sand.

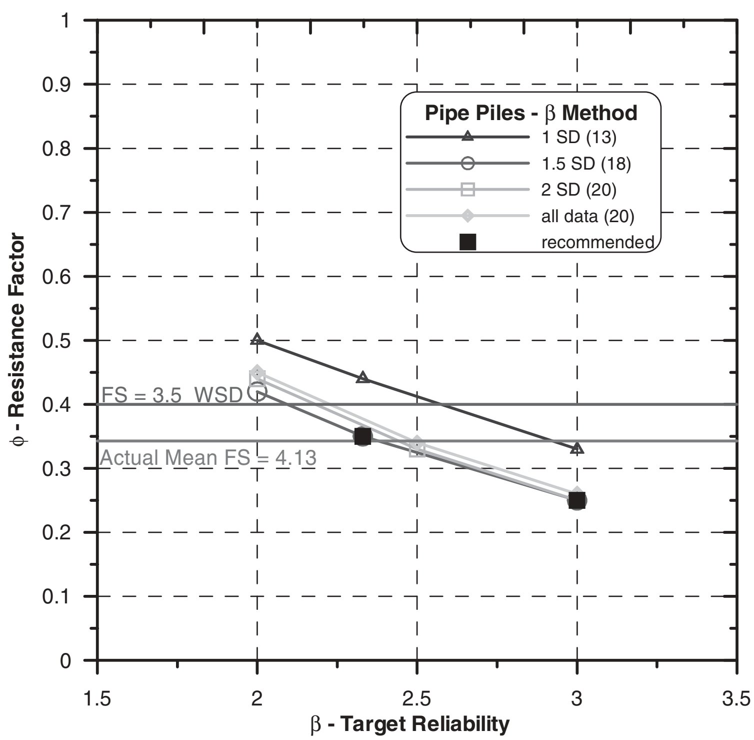


Figure 50. Sensitivity analysis examining the recommended parameters for the design of pipe piles in sand using the β method.

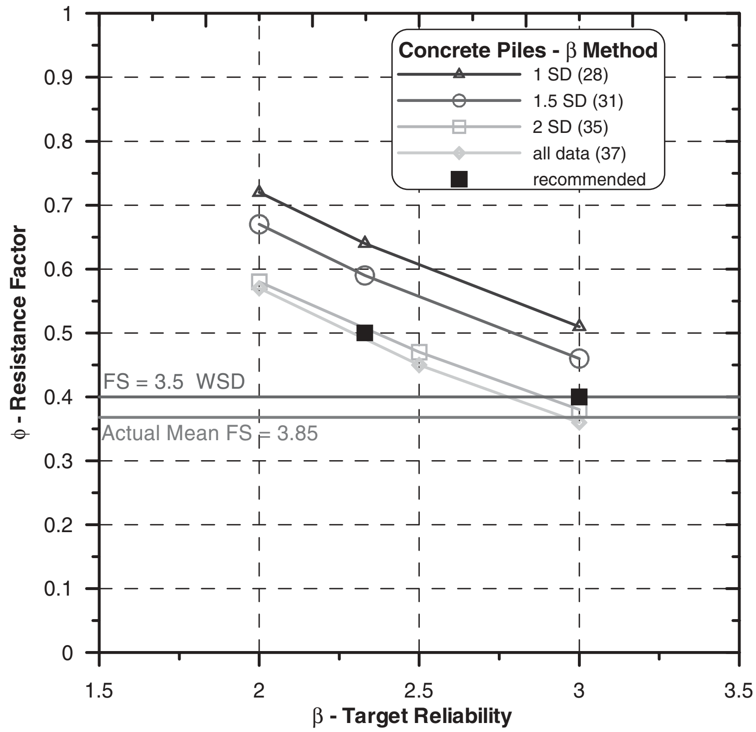


Figure 51. Sensitivity analysis examining the recommended parameters for the design of concrete piles in sand using the β method.

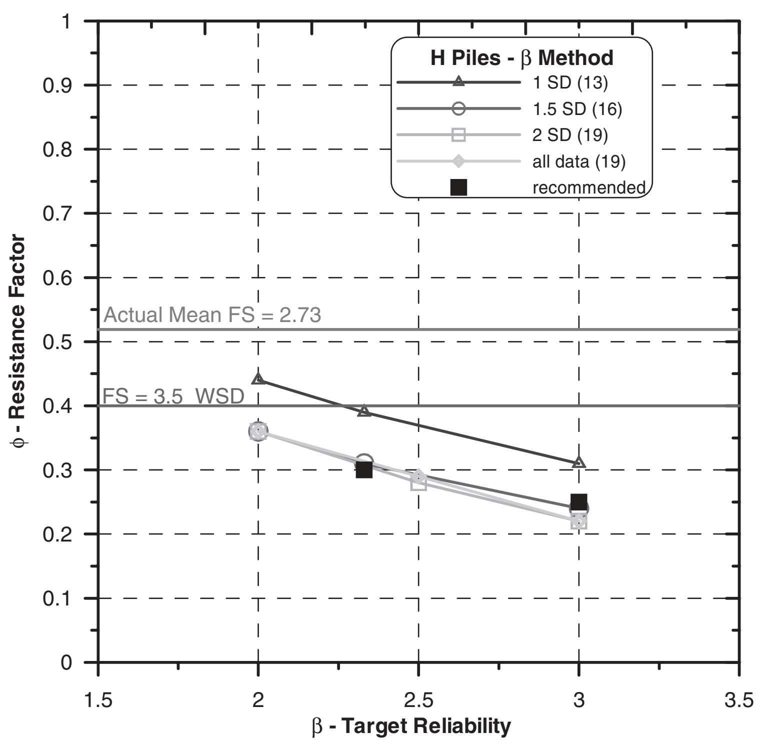


Figure 52. Sensitivity analysis examining the recommended parameters for the design of H piles in sand using the β method.

TABLE 31 Calculated probability of failure [$p = (\%)$] based on direct utilization of database PD/LT 2000 for selected prediction methods

Resistance factor ϕ	CAPWAP General	CAPWAP BOR	CAPWAP EOD AR > 350 BL ct. > 16 BP10cm	Energy Approach EOD	FHWA Mod Gates General
0.5	0.27	0	2.70	1.56	10.42
0.4	0	0	0	0	3.13
0.33	0	0	0	0	0.78
# of cases used	377	162	37	128	384

CHAPTER 4

CONCLUSIONS AND SUGGESTED RESEARCH

4.1 CONCLUSIONS

The data and analyses presented in this report lead to the following major conclusions:

1. The compilation of large databases allows for the quantitative assessment of pile capacity evaluation methods during both design and construction. In addition, databases (combined with the application of mechanics principles) allow the determination of the controlling parameters of capacity evaluation methods that require calibration. Databases are essential therefore for the examination of any design methodology and hence enable the testing of the factors of safety used by WSD (i.e., their validity and effect on costs) and the development of other methodologies, such as LRFD.
2. LRFD facilitates a design methodology that is more suitable for geotechnical applications than WSD. The ability to determine design factors while quantifying the significance of their outcome is a powerful tool in engineering. The development of load and resistance factors utilizing reliability-based calibration and databases is a major necessary step toward objective quantification of the LRFD parameters. More so, it allows a meaningful utilization of the LRFD principles (in contrast to parameter fitting to WSD) and sets a base for future developments that will further rationalize design and lead to more economical construction.
3. The findings presented are the first of their kind in the development of resistance factors for LRFD design methodologies in geotechnical engineering. A review of the recommended resistance factors must be compatible with the fundamental principle of the methodology, i.e. engineering design with a consistent level of reliability. Existing LRFD codes worldwide were developed, by and large, to be compatible with previous WSD parameters based on different concepts. While radical changes cannot be expected in construction practices, a shift in both directions (more and less conservative depending on the specifics) should be expected and accepted when adopting a true LRFD design.

4.2 SUGGESTED RESEARCH—KNOWLEDGE-BASED DESIGNS

4.2.1 Statement of Problem

Variability in the parameters used in the design, site conditions, construction quality, and previous experience are all important factors. The present study bypassed some of the difficulties by calibrating specific design methods and correlations as a unit. A more complete design process based on LRFD can recognize the contribution of different factors—such as subsurface variability, site-specific technology, and previous experience—as well as amount and type of testing during construction. A framework for such an approach is presented here; further development, however, will require additional databases, e.g., for the correlation between soil parameters and field tests, as well as subjective judgments.

4.2.2 Framework for LRFD Design for Deep Foundations

In order to fully exploit the potential of the LRFD methodology for geotechnical purposes the aforementioned issues must be addressed. Many of the affecting factors are in fact being considered in the design (e.g., previous experience) but need a framework to allow future progress. A proposed solution is to establish knowledge-based factors for both the design and the construction (independent) capacity evaluation methods. These factors can be accounted for by a modifying constant ξ to be multiplied by the resistance factor.

$$\xi\phi R > \gamma L \qquad (40)$$

Where:

$\xi = \xi_1 \xi_2 \xi_3 \xi_4 \leq \xi_{limit}$
ξ_1 = factor adjusting for the variability of site conditions
ξ_2 = factor adjusting for the quality of soil parameter estimates
ξ_3 = factor adjusting for construction quality control
ξ_4 = factor adjusting for previous site or construction experience

ξ_{limit} = an upper limit on the factor that will be determined from computing the components of ξ (and judgment). The limit should have some real value larger than 1.0, such as 1.10.

A short description of the knowledge-based factors follows.

ξ_1 applies to the spatial variation of soil properties, stratification across a site, and the extent to which that variation has been categorized by the subsurface investigation program. The factor is relevant for deep foundations capacity evaluation during both design and construction. Low values mean that the site is more erratic than normal or that little exploration and testing has been done. Average values reflect normally variable soil conditions adequately investigated. High values mean that the site is more uniform than normal and that an extensive program of boring and testing has been conducted. The extent of exploration can be evaluated via number of borings per substructure unit.

ξ_2 applies only to deep foundations capacity methods employing calculations based on soil parameters. This factor accounts for the manner in which soil parameters are estimated from field and laboratory test data and the exactness of those estimates. Low values mean that the correlation between soil parameters and the measurements they are based upon is poor (e.g., when estimating undrained shear strength of soft to medium clay from SPT values), and thus the confidence in the accuracy of the soil parameter values is low. Average values reflect normally variable soil conditions adequately investigated. High values mean that the correlation is good (e.g., when parameters are estimated based on laboratory test of undisturbed samples or direct in situ testing like a field vane test), and thus the confidence in parameter values is high.

ξ_3 applies to the extent of measures taken to control construction quality and testing the integrity and capacity of the constructed deep foundations. Low values mean that few measures are to control construction quality; and no static, dynamic, or integrity testing results are available. High values mean that extensive measures are taken to control construction quality, and multiple pile testing results are available (e.g., integrity and capacity of drilled shafts, static and dynamic tests of driven piles). The high quality control also relates to the number of tested deep foundations as a ratio of the number of piles installed per substructure.

ξ_4 is to be used during the design to account for previous experience accumulated either on a specific construction site or from a specific construction technology. Low values are used if no previous experience is known at the site and a new unfamiliar construction technology is used. High values mean that previous deep foundation testing results similar to the one designed (type and installation) are available.

BIBLIOGRAPHY*

AASHTO. 1994. *LRFD Bridge and Construction Specifications.* AASHTO, Washington, DC.
AASHTO. 1996/2000. *LRFD Bridge Design Specifications: 2nd LRFD Edition—2000 Interim Revisions*. AASHTO, Washington, DC.
AASHTO. 1997. *Standard Specifications for Highway Bridges: 16th Edition* (1996 with 1997 interims). AASHTO, Washington, DC.
AASHTO. 2001. *LRFD Bridge Design Specifications.* AASHTO, Washington, DC.
American Concrete Institute (ACI). 1995, *Building Code Requirements for Reinforced Concrete.* American Concrete Institute, Detroit, MI.
American Institute of Steel Construction (AISC). 1994. Load and Resistance Factor Design. *Manual of Steel Construction,* American Institute of Steel Construction, Chicago, IL.
American Petroleum Institute (API). 1989. Draft Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms—Load and Resistance Factor Design. API RP2A-LRFD. American Petroleum Institute, Dallas, TX.
ASCE. 1993. *Minimum Design Loads for Buildings and Other Structures ASCE 7-93.* (Formerly ANSI A58.1). ASCE, Reston, VA.
AUSTROADS. 1992. *AUSTROADS Bridge Design Code.* National Office, AUSTROADS, Surry Hills, NSW, Australia.
Ayyub, B.M. 1999. *Cost-Benefit Analysis of Risk-Informed Design of Structural Systems.* American Society of Mechanical Engineers, the Research Committee on Risk Technology, Washington, DC.
Ayyub, B., and Assakkaf, I. 1999. *LRFD Rules for Naval Surface Ship Structures: Reliability-Based Load and Resistance Factor Design Rules.* Naval Surface Warfare Center, Carderock Division, U.S. Navy.
Ayyub, B., Assakkaf, I., and Atua, K. 1998a. *Development of LRFD Rules for Naval Surface Ship Structures: Reliability-Based Load and Resistance Factor Design Rules, Part III—Stiffened and Gross Panels.* Naval Surface Warfare Center, Carderock Division, U.S. Navy.
Ayyub, B., Assakkaf, I., and Atua, K. 2000. Reliability-Based Load and Resistance Factor Design (LRFD) of Hull Girders for Surface Ships. *Naval Engineers Journal,* Vol. 112, No. 4, July 2000, pp. 279–296.
Ayyub, B., Assakkaf, I., Atua, K., Engle, A., Hess, P., Karaszewski, Z., Kihl, D., Melton, W., Sielski, R., Sieve, M., Waldman, J., and White, G. 1998b. *Reliability-based Design of Ship Structures: Current Practice and Emerging Technologies.* T&R Report R-53, Society of Naval Architects and Marine Engineers.
Ayyub, B., and McCuen, R. 1997. *Probability, Statistics and Reliability for Engineers.* CRC Press, FL.
Baecher, G. 2001. Contribution to a progress research report as part of Project NCHRP 24-17, LRFD Deep Foundations Design, unpublished document.
Barker, R., Duncan, J., Rojiani, K., Ooi, P., Tan, C., and Kim, S. 1991. *NCHRP Report 343: Manuals for the Design of Bridge Foundations.* TRB, National Research Council, Washington, DC.
Becker, D. 1996. Eighteenth Canadian Geotechnical Colloquium: Limit States Design for Foundations. Part I. An overview of the foundation design process. *Canadian Geotechnical Journal*, Vol. 33, No. 6, December, pp. 956–983.
Benjamin, J. R., and Cornell, C. A. 1970, *Probability, Statistics, and Decision for Civil Engineers*. McGraw-Hill, New York.
Bermingham, P., and White, J. 1995. Pyrotechnics and the Accurate Prediction of Statnamic Peak Loading and Fuel Charge Size. *Proceedings First International Statnamic Seminar*, September 27–30, Vancouver, BC, Canada, pp. 1–12.
Bowles, J. 1996. *Foundation Analysis and Design.* McGraw-Hill, New York.
Brooms, B. 1964a. “Lateral Resistance of Piles in Cohesive Soils.” *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. SM2, March, pp. 27–63.
Brooms, B. 1964b. “Lateral Resistance of Piles in Cohesionless Soils.” *Journal of the Soil Mechanics and Foundations Division*, ASCE, Vol. SM3, May, pp.123–156.
Butler, H.D., and Hoy, H.E. 1977. *Users Manual for the Texas Quick-Load Method for Foundation Load Testing.* FHWA-IP-77-8. FHWA, Office of Development, Washington, DC.
Canadian Geotechnical Society. 1992. *Canadian Foundation Engineering Manual,* 3rd Edition. Bi-Tech Publishers, Ltd., Richmond, British Columbia, Canada.
Carter, J. P., and Kulhawy, F. H. 1988. *Analysis and Design of Foundations Socketed into Rock.* Electric Power Research Institute, Report Number EPRI EL-5918.
Construction Industry Research and Information Association (CIRIA). 1977. *Rationalization of Safety and Serviceability Factors in Structural Codes: CIRIA Report 63.* Construction Industry Research and Information Association, SWIP 3AU, Report 63, London, England.
Chellis, R.D. 1961. *Pile Foundation,* 2nd ed. McGraw Hill, NY.
Davisson, M. 1972. High Capacity Piles. In *Proceedings, Soil Mechanics Lecture Series on Innovations in Foundation Construction,* ASCE, Illinois Section, Chicago, IL, pp. 81–112.
DeBeer, E. 1970. Proefondervindellijke bijdrage tot de studie van het grandsdraagvermogen van zand onder funderinger op staal. English version. *Geotechnique,* Vol. 20, No. 4, pp. 387–411.
DiMaggio, J., Saad, T., Allen, T., Passe, P., Goble, G., Christopher, B., DiMillio, A., Person, G., and Shike, T. 1998. FHWA Summary Report of the Geotechnical Engineering Study Tour (GEST). FHWA International Technology Scanning Program, December, FHWA.
DiMillio, A. 1999. *A Quarter Century of Geotechnical Research.* FHWA-RD-98-139, FHWA.
Ellingwood, B., Galambos, T., MacGregor, J., and Cornell C. 1980. *Development of a Probability-Based Load Criterion for American National A58.* National Bureau of Standards Publication 577. Washington, DC.
Ellingwood, B., Galambos, T. 1982. Probability-Based Criteria for Structural Design. *Structural Safety*, Vol.1, pp. 15–26.
Ellingwood, B., Galambos, T., MacGregor, J., and Cornell, C. 1982a. Probability Based Load Criteria—Assessment of Current Design Practices. *Journal of the Structural Division,* ASCE, Vol. 108, No. ST5, pp. 959–977.
Ellingwood, B., Galambos, T., MacGregor, J., and Cornell, C. 1982b. Probability Based Load Criteria—Load Factors and Load

*Bibliographic material not verified by TRB.

Combinations. *Journal of the Structural Division,* ASCE, Vol. 108, ST5, pp. 978–997.

Esrig, M. I., and Kirby, R. C. 1979. Soil Capacity for Supporting Deep Foundation Members in Clay. ASTM STP. No. 670, pp. 27–63.

Eurocode 7. 1997. Geotechnical Design—Part I: General Rules. European Committee for Standardization, Central Secretariat, Brussels.

FHWA. 1988. *Geotechnical Guideline No. 13.* FHWA, 37 pp.

Fragaszy, R., Higgins, J., and Lawton, E. 1985. *Development of Guidelines for Construction Control of Pile Driving and Estimation of Pile Capacity: Phase I.* Washington DOT Report WA/RD-68.1, Olympia, Washington, 83 pp.

Freudenthal, A. 1947. Safety of Structures. *Transactions of the ASCE.* Vol. 112, pp. 125–180.

Galambos, T., and Ravindra, M. 1978. Properties of Steel for Use in LRFD. *Journal of Structural Engineering,* Vol. 104, No. 9, pp. 1459–1468.

Gates, M., 1957. Empirical Formula for Predicting Pile Bearing Capacity. *Civil Engineering,* Vol. 27, No. 3, pp. 65–66.

Geosciences Testing and Research, Inc. 1997. *Dynamic Pile Testing Report, Central Artery/Tunnel Project C07D2, I-90/Airport Interchange Arrivals Tunnel—Phase I, East Boston, Massachusetts.* Geosciences Testing and Research, Inc. North Chelmsford, MA.

Geosciences Testing and Research, Inc. (GTR). 1998. *Dynamic Pile Testing Report, Central Artery/Tunnel Project C07D2, I-90/Airport Interchange Toll Plaza, East Boston, Massachusetts.* Geosciences Testing and Research, Inc., North Chelmsford, MA.

Goble, G. 1999. *NCHRP Synthesis of Highway Practice 276: Geotechnical Related Development and Implementation of Load and Resistance Factor Design (LRFD) Methods.* TRB, National Research Council, Washington, DC.

Goble, G., Likens, G., and Rausche, F. 1970. Dynamic Studies on the Bearing Capacity of Piles—Phase III. Report No. 48. *Division of Solid Mechanics, Structures, and Mechanical Design.* Case Western Reserve University.

Goble, G., and Rausche, F. 1976. Wave Equation Analysis of Pile Driving-WEAP Program. Vol. 1–4. FHWA #IP-76-14.1 through #IP-76-14.4.

GRL, Inc. 1999. *Pile-Driving Analyzer, PAK Users Manual.* Goble, Rausche, Likins and Associates, Inc.

Grant, E., and Leavenworth, R. 1996. *Statistical quality control.* 7th ed., McGraw-Hill, New York, NY, 1996. pp. 764.

Hajduk, E., Paikowsky, S., Hölscher, P., and Barends, F. 2000. Accelerations of a Driven Pile and the Surrounding Soil. *Proceedings of the 6th International Conference on the Applications of Stress-Wave Theory to Piles,* S. Niyama, and J. Beim eds., September 11–13, São Paulo, Brazil, Balkema, Rotterdam-the Netherlands, pp. 541–548.

Hamilton, J., and Murff, J. 1992. Selection of LRFD Resistance Factors for Pile Foundation Design. American Society of Civil Engineers Structures Congress, April 13–15, San Antonio, Texas, *Proceedings Structures Congress '92,* ASCE.

Hannigan, P., Goble, G., Thendean, G., Likins, G., and Rausche, F. 1995. *Design and Construction of Driven Pile Foundation.* FHWA, Washington, DC.

Hannigan, R., Goble, G., Thendean, G., Likins, G., and Rausche, F. 1996. *Design and Construction of Driven Piles Foundations—Volume II (Working Draft).* U.S. DOT, FHWA. Washington, DC.

Hansen, B. 1953. *Earth Pressure Calculation.* Danish Technical Press, Copenhagen, Denmark.

Hansen, B. 1956. *Limit Design and Safety Factors in Soil Mechanics.* Bulletin No. 1, Danish Geotechnical Institute, Copenhagen, Denmark.

Hansen, B. 1966. *Code of Practice for Foundation Engineering.* Bulletin No. 22, Danish Geotechnical Institute, Copenhagen, Denmark.

Hasofer, A. M., and Lind, N. C. 1974. An Exact and Invariant First-Order Reliability Format. *Journal of Engineering Mechanics*, Vol. 100. No. EM1, pp. 111–121.

Hölscher, P. 1995. *Dynamical Response of Saturated and Dry Soils.* Delft University Press, Delft, The Netherlands.

Hölscher, P., and Barends, F. 1996. In-situ Measurement of Soil-Motion near the Toe of a Dynamically Loaded Pile. In *Proceedings, 5th International Conference of the Application of Stress-Wave Theory to Piles, September 11–13, 1996 Orlando, FL* (F.C. Townsend, M. Hussein, and M. McVay, eds.), pp. 26–36.

Japanese Geotechnical Society. 1998. *Foundation Design Standards in the World—Toward Performed-Based Design.* Research Committee on Present & Future of Japanese Foundation Design and Soil Investigation in View of International Equivalency.

Kulhawy, F., and Phoon, K. 1996. Engineering Judgment in the evolution from Deterministic to Reliability-Based Foundation Design. *Proceedings of the 1996 Conference on Uncertainty in the Geologic Environment, UNCERTAINTY'96*. Part 1 (of 2), July 31–Aug. 31, Madison, WI, ASCE, NY, pp. 29–48.

Kulhawy, F., and Mayne, P. 1990. Manual on Estimating of Soil Properties for Foundation Design. Electric Power Research Institute, Palo Alto, CA.

Kuo, C., McVay, M., and Birgisson, M. 2002. Calibration of Load and Resistance Factor Design: Resistance Factors for Drilled Shaft Design, TRB, *Transportation Research Record,* No. 1808, pp.108–111, Annual Meeting, January 13–17, Washington, DC.

Lai, P., and Graham, K. 1995. Static Pile Bearing Analysis Program. SPT94. http://www.dot.state.fl.us/structures/manuals/spt94.zip

Long, J.; Bozkurt, D.; Kerrigan, J.; Wysockey, M. 1999. Value of Methods for Predicting Axial Pile Capacity. TRB *Transportation Research Record No. 1663,* pp. 57–63, 76th Annual Meeting, January, Washington, DC.

Madsen, H.O., Krenk, S. and Lind, N.C. 1986. *Methods of Structural Safety.* Prentice Hall, Englewood Cliffs, NJ.

Mansour, A.E., Wirsching, P.H., Ayyub, B.M., and White, G.J. 1994. *Probability Based Ship Design Implementation of Design Guidelines for Ships, Ship Structures Committee Draft Report.* U.S. Coast Guard, Washington, DC.

Massachusetts. *Code of Massachusetts Regulations (CMR)*. State Building Code—780, 1997. Article 12, section 1214.3.1, Driving Formula, CMR 5th edition.

Massachusetts Highway Department. 1988. *Standard Specifications for Highways and Bridges.* Massachusetts Highway Department.

McVay, M., Birgisson, B., Nguyen, T., and Kuo, C. 2002. Uncertainty in LRFD phi, ϕ, Factors for Driven Prestressed Concrete Piles, TRB, *Transportation Research Record 1808,* pp. 99–107, Annual Meeting, January 13–17, Washington, DC.

McVay, M., Birgisson, B., Zhang, L., Perez., A, and Putcha, S. 2000. Load and Resistance Factor Design (LRFD) for Driven Piles Using Dynamic Methods—A Florida Perspective. *Geotechnical Testing Journal,* Vol. 23, No. 1, pp. 55–66.

McVay, M., and Townsend, F. 1989. *Short Course/Workshop Guide for PC Software Support for Design and Analysis of Deep and Shallow Foundations from In-situ Tests (or PL-AID Manual).* University of Florida, Gainesville, Florida.

Melchers, R. E. 1987. *Structural Reliability Analysis and Prediction.* Ellis Horwood Limited, UK.

Meyerhof, G. 1970. Safety Factors in Soil Mechanics. *Canadian Geotechnical Journal*, Vol. 7, No. 4, pp. 349–355.

Meyerhof, G. 1976. Bearing Capacity And Settlement Of Pile Foundations. American Society of Civil Engineers*, Journal of the Geotechnical Engineering Division*, Vol. 102, No. 3, March, pp. 195–228.

Meyerhof, G. 1994. Evolution of Safety Factors and Geotechnical Limit State Design. *Second Spencer J. Buchanan Lecture*, Texas A and M University, Nov. 4, pp. 32.

Middendorp, P., and Bielefeld, M. 1995. Statnamic Load Testing and the Influence of Stress Wave Phenomena. *Proceedings of the First International Statnamic Seminar, Vancouver, BC, Canada*, September 27–30, pp. 207–220.

Middendorp, P., and van Weel, P. 1986. Application of Characteristic Stress-Wave Method in Offshore Practice. *Proceedings of Numerical Methods in Offshore Piling: 3rd International Conference, Nantes,* May 21–22, Editions Technip.

Moses, F. 1985. *Implementation of a Reliability-Based API RP2A Format, Final Report.* API PRAC 83-22. American Petroleum Institute, Dallas, TX.

Moses, F. 1986. *Development of Preliminary Load and Resistance Factor Design Document for Fixed Offshore Platforms, Final Report.* API-PRAC 95-22, American Petroleum Institute, Dallas, TX.

Moses, F., and Verma, D. 1987. *NCHRP Report 301: Load Capacity Evaluation of Existing Bridges.* TRB, Washington, DC.

National Research Council. 1995. *Probabilistic Methods in Geotechnical Engineering.* National Academies Press, Washington, DC, pp. 96.

National Research Council of Canada. 1977. *National Building Code of Canada.* Ottawa, Canada.

Nordic Committee on Building Regulations. 1978. *Recommendations for Loading and Safety Regulations for Structural Design.* Nordic Committee for Safety of Structures, NKB Report No. 36. Copenhagen, Denmark.

Nordlund, R. L. 1963. Bearing Capacity of Piles in Cohesionless Soils. *Journal of Soil Mechanics and Foundation Engineering,* JSMFE, Vol. 89, SM 3, pp 1–36.

Nottingham, L., and Schmertmann, J. 1975. *An Investigation of Pile Capacity Design Procedures*. Final Report D629 to Florida Department of Transportation, Department of Civil Engineering, University of Florida, pp 159.

Nowak, A. 1999. *NCHRP Report 368: Calibration of LRFD Bridge Design Code.* TRB, Washington, DC.

Olsen, R., and Flaate, K. 1967. Pile Driving Formulas for Friction Piles in Sand. *Journal of Soil Mechanics and Foundation Engineering,* JSMFE, Vol. 93, SM6, Nov. 1967, 279–297.

O'Neill, M. 1995. LRFD Factors for Deep Foundations through Direct Experimentation. In *Proceedings of US/Taiwan Geotechnical Engineering Collaboration Workshop.* Sponsored by the National Science Foundation (USA) and the National Science Council (Taiwan, ROC), A.B. Huang, Y.S. Fang, P.W. Mayne, and S.G. Paikowsky eds., Taipei, January 9–11, pp. 100–114.

O'Neill, M., and Reese, L. 1999. Drilled Shafts: Construction Procedures and Design Methods, Volume II Chapters 9–19, Appendices. U.S. DOT, FHWA, Washington, DC.

O'Neill, M., Townsend, F., Hassan, K., Buller, A., and Chang, P. 1996. *Load Transfer for Drilled Shafts in Intermediate Geomaterials*, FHWA-RD-95-172, FHWA.

Ontario Ministry of Transportation and Communication. 1992. *Ontario Highway Bridge Design Code and Commentary.* 3rd ed.

Orr, T. 2002. Eurocode 7— A Code for Harmonized Geotechnical Design. *Proceedings of the International Workshop on Foundation Design Codes and Soil Investigation in View of International Harmonization and Performance Based Design,* IWS Kamakura 2002, Tokyo, Japan, April 10–12, Y. Honjo, O. Kusakabe, K. Matsui, M. Kouda, and G. Pokharel eds., Balkema, the Netherlands, pp. 3–15.

Orr, T., and Farrell, E. 1999. *Geotechnical Design to Eurocode 7*. Springer-Verlag, UK, pp. 175

Osterberg, J. 1992. *The Osterberg Cell for Load Testing Drilled Shafts And Driven Piles.* FHWA Report SA-94-035, Washington, DC.

Paikowsky, S. 1982. Use of Dynamic Measurements to Predict Pile Capacity Under Local Conditions. M.Sc. Thesis, Dept. of Civil Engineering Technion-Israel Institute of Technology, Haifa, Israel.

Paikowsky, S. 1984. *Use of Dynamic Measurements for Pile Analysis. Including PDAP-Pile-Driving Analysis Program.* GZA Inc., Newton, Massachusetts.

Paikowsky, S. 1995. Using Dynamic Measurements for the Capacity Evaluation of Driven Piles. *Civil Engineering Practice, Journal of the Boston Society of Civil Engineers Section/ASCE.* Vol. 10, No. 2, pp. 61–76.

Paikowsky, S., and Chernauskas, L. 1992. Energy Approach for Capacity Evaluation of Driven Piles. *Proceedings of the 4th International Conference on the Application of Stress-Wave Theory to Piles,* F.B. Barends ed., September 21–24, The Hague, The Netherlands, pp. 595–601.

Paikowsky, S., and Chernauskas, L. 1996. Soil Inertia and the Use of Pseudo Viscous Damping Parameters*. Proceedings of the 5th International Conference on the Application of Stress-Wave Theory to Piles,* F.C. Townsend, M. Hussein, and M.C. McVay eds., September 11–13, Orlando, FL, pp. 203–216.

Paikowsky, S., and Hajduk, E. 1999. *Design and Construction of an Instrumented Test Pile Cluster:* Research Report submitted to the Massachusetts Highway Department, September, 1999. University of Massachusetts—Lowell, MA.

Paikowsky, S., and Hajduk, E. 2000. *Theoretical Evaluation and Full Scale Field Testing Examination of Pile Capacity Gain with Time:* Research Report submitted to the Massachusetts Highway Department, December, University of Massachusetts—Lowell, MA.

Paikowsky, S., and Hart, L. 2000. *Development and Field Testing of Multiple Deployment Model Pile (MDMP): Research Report to be submitted to the Federal Highway Administration, April, 2000.* FHWA-RD-99-194. FHWA.

Paikowsky, S., and LaBelle, V. 1994. Examination of the Energy Approach for Capacity Evaluation of Driven Piles. *Proceedings of the International Conference on Design and Construction of Deep Foundations,* December 6–8, Orlando, FL. FHWA, Vol. II, pp. 1133–1149.

Paikowsky, S., LaBelle, V., and Hourani, N. 1996. Dynamic Analyses and Time Dependent Pile Capacity. *Proceedings of the 5th International Conference on the Application of Stress-Wave Theory to Piles,* F.C. Townsend, M. Hussein, and M.C. McVay eds., September 11–13, Orlando, FL, pp. 325–339.

Paikowsky, S., LaBelle, V., and Mynampaty, R. 1995. *Static and Dynamic Time Dependent Pile Behavior:* Research Report submitted to the Massachusetts Highway Department, November, University of Massachusetts—Lowell, MA.

Paikowsky, S., Operstein, V., and Bachand, M. 1999. *Express Method of Pile Testing by Static Cyclic Loading.* Research Report submitted to the Massachusetts Highway Department, October, University of Massachusetts—Lowell, MA.

Paikowsky, S., Regan, J., and McDonnell, J. 1994. *A Simplified Field Method for Capacity Evaluation of Driven Piles.* FHWA-RD-94-042, FHWA.

Paikowsky, S., and Stenersen, K. 2000. The Performance of the Dynamic Methods, their Controlling Parameters and Deep Foundation Specifications. Keynote lecture in Stress Wave 2000, *Proceedings of the Sixth International Conference on the Application of Stress-Wave Theory to Piles*, S. Niyama, and J. Beim eds., September 12, São Paulo, Brazil, Balkema, Rotterdam—the Netherlands, pp. 281–304.

Paikowsky, S., and Whitman, R. 1990. The Effect of Plugging on Pile Performance and Design. *Canadian Geotechnical Journal,* Vol. 27, No. 4, pp. 429–440.

Paikowsky, S., Whitman, R., and Baligh, M. 1989. A New Look at the Phenomenon of Offshore Pile Plugging. *Marine Geotechnology,* Vol. 8, No. 3, pp. 213–230.

Payer, H., Huppmann, H., Jochum, C., Madsen, H., Nittinger, K., Shibata, H., Wild, W., and Wingender, H. 1994. Plenary Panel Discussion on How Safe is Safe Enough? *Proceedings of the 6th International Conference on Structural Safety and Reliability,* G.I. Schueller, M. Shinozuka and J.T.P. Yao eds., ICOSSAR '93, Innsbruck, August 9–13, Balkema, Rotterdam, The Netherlands, pp. 57–74.

Phoon, K.K., and Kulhawy, F.H. 1996. On Quantifying Inherent Soil Variability. *Uncertainty in the Geologic Environment: from Theory to Practice* Proceedings of Uncertainty '96, C.D. Shackelford, P.P. Nelson, and M.J.S. Roth eds., Madison, WI, July 31–August 3, ASCE, New York, pp. 326–340.

Phoon, K., Kulhawy, F. and Grigoriu, M. 1995. *Reliability-Based Design of Foundations for Transmission Line Structures.* Report TR-105000. Electrical Power Research Institute (EPRI), Palo Alto, Texas.

Rausche, F. 2000. Personal communication. February 2000.

Rausche, F., Goble, G., and Likens, G. 1975. *Bearing Capacity of Piles from Dynamic Measurements, Final Report.* Ohio DOT-05-75. Ohio Department of Transportation.

Rausche, F., Thendean, F., Abou-matar, H., Likins, G., and Goble, G. 1997. *Determination of Pile Driveability and Capacity from Penetration Tests. Vol. 1–3, Final Report.* FHWA-RD-96-179, FHWA.

Reese, L. C., and O'Neill M. W. 1988. *Drilled Shaft: Construction Procedures and Design Methods.* FHWA-HI-88-042, FHWA.

Reese, L.C., Wang, S.C., and Arrellaga, J. 1998. APILE Plus 3.0 Manual. EnSoft, Inc. Austin, TX.

Reese, L. C., and Wright S. J. 1977. Construction Procedures and Design for Axial Loading, Drilled Shaft Manual HDV-22.

Rosenblueth, E., and Esteva, L. 1972. *Reliability Basis for Some Mexican Codes.* ACI Publication SP-31. American Concrete Institute, Detroit, MI.

Siu, W., Parimi, S., and Lind, N. 1975. Practical Approach to Code Calibration. *Journal of the Structural Division,* Vol. 101, No. ST7, pp. 1469–1480.

Smith, E. 1960. Pile Driving Analysis by the Wave Equation. *Journal of Soil Mechanics and Foundations,* ASCE, Vol. 86, No. SM4, August 1960, pp. 35–61.

Standards Association of Australia. 1995. *Australian Standards, Piling-Design and Installation.* Standards Association of Australia, Homebush, NSW, Australia.

Tang, W. 1993. Recent Developments in Geotechnical Reliability. *Proceedings of the Conference on Probabilistic Methods in Geotechnical Engineering* Li, K., and Lo, S-C. eds., Canberra, Australia, February 10–12, Balkema, Rotterdam, The Netherlands, pp. 3–28.

Tang Q., Drumm, E.C., and Bennett, R.M. 1994. Response to Drilled Shaft Foundations in Karst During Construction Loading, *Proceedings of the International Conference on Design and Construction of Deep Foundations,* December 6–8, Orlando, FL, FHWA, Vol. III, pp. 1296–1309.

Tang, W., Woodford, D., and Pelletier, J. 1990. Performance Reliability of Offshore Piles. *Proceedings of the 22nd Annual Offshore Technology Conference.* May 7–10, Houston, TX, Paper No. OTC 6379, Offshore Technology Conference, Richardson, TX, Vol. 3, p. 299–308.

Terzaghi, K. 1942. Discussion of the Progress Report of the Committee on the Bearing Value of Pile Foundations. *Proceedings, ASCE.* Vol. 68, pp. 311–323.

Terzaghi, K., and Peck, R.B. 1967. *Soil Mechanics in Engineering Practice.* Wiley.

Thoft-Christensen, P., and Baker, M. 1982. *Structural Reliability Theory and Its Application.* Springer-Verlag, New York.

Tomlinson, M.J., 1986. *Foundation Design and Construction.* Longman Scientific and Technical, Essex, England.

Tomlinson, M.J. 1995. *Foundation Design and Construction.* 6th Edition. Longman Scientific and Technical, Essex, England.

Trautmann, C., and Kulhawy, F. 1996. "Estimation of In-situ Test Uncertainty." *Uncertainty in Geological Environment: From Theory to Practice,* pp. 269–86.

U.S. Army Corps of Engineers. 1992. *Bearing Capacity of Soil, Technical Engineering and Design Guides.* Adapted by ASCE—2000, No.7, EM 1110-1-1905. ASCE Press, New York.

Vijayvergiya, V.N., and Focht Jr., J.A. 1972. A New Way to Predict Capacity of Piles In Clay. *Proceedings of the 4th Annual Offshore Technology Conference,* Houston, TX, Offshore Technology Conference, Richardson, TX, Vol. 2, pp. 856–874.

Wellington, A. 1892. Discussion of "The Iron Wharf at Fort Monroe, Va. By J.B. Cuncklee." *Transactions,* ASCE Vol. 27, paper No. 543, Aug. 1892, pp. 129–137.

Whitman, R. 1984. Evaluating Calculated Risk in Geotechnical Engineering. The Seventeenth Terzaghi Lecture. *Journal of Geotechnical Engineering,* Vol. 110, No. 2, pp. 145–188.

Wirsching, P. H. 1984. Fatigue Reliability for Offshore Structures. *Journal of Structural Engineering,* Vol. 110, No. 10, pp 2340–56.

Withiam, J., Voytko, E., Barker, R., Duncan, M., Kelly, B., Musser, S., and Elias, V. 1998. *Load and Resistance Factor Design (LRFD) of Highway Bridge Substructures.* FHWA HI-98-032. FHWA.

Wu, T., Tang, W., Sangrey, D., and Baecher, G., 1989. Reliability of Offshore Foundations—State of the Art. *Journal of Geotechnical Engineering,* Vol. 115, No. 2, pp. 157–178.

Zeitlen, J.G., and Paikowsky, S.G. Nov. 1982. "New Design Correlations for Piles in Sand." Discussion in ASCE Journal of GE, 108, No. GT11, pp. 1515–1518.

Zhang, L., Silva-Tulla, F., and Grismala, R. 2002. Ultimate Resistance of Laterally Loaded Piles in Cohesionless Soils, *Proceedings of the International Deep Foundations Congress 2002: An International Perspective on Theory, Design, Construction, and Performance.* February 14–16, 2002 Orlando, Florida, Geotechnical Special Publication No. 116, ASCE, pp.1364–1375.

Zhang, L., Tang, W., and Ng, C. 2001. Reliability of Axially Loaded Driven Pile Groups. *Journal of Geotechnical and Geoenvironmental Engineering,* Vol. 127, No. 12, pp. 1051–1060.

APPENDIXES

Appendixes A through D are published on the accompanying CD (CRP-CD-39).
The appendixes and their authorship are as follows:

- Appendix A: Surveys—State of Practice Design and Construction (Ching L. Kuo, Bjorn Birgisson, Michael McVay, and Samuel G. Paikowsky)
- Appendix B: Load and Resistance Factor Design (LRFD) for Dynamic Analyses of Driven Piles (Samuel G. Paikowsky and Kirk L. Stenersen)
- Appendix C: Static Analyses of Driven Piles and Drilled Shafts (Ching L. Kuo, Bjorn Birgisson, Michael McVay, and Samuel G. Paikowsky)
- Appendix D: Design Examples (Ching L. Kuo, Samuel G. Paikowsky, Kirk Stenersen, Roiy Guy, Bjorn Birgisson, and Michael McVay)

Abbreviations used without definitions in TRB publications:

AASHO	American Association of State Highway Officials
AASHTO	American Association of State Highway and Transportation Officials
APTA	American Public Transportation Association
ASCE	American Society of Civil Engineers
ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
ATA	American Trucking Associations
CTAA	Community Transportation Association of America
CTBSSP	Commercial Truck and Bus Safety Synthesis Program
FAA	Federal Aviation Administration
FHWA	Federal Highway Administration
FMCSA	Federal Motor Carrier Safety Administration
FRA	Federal Railroad Administration
FTA	Federal Transit Administration
IEEE	Institute of Electrical and Electronics Engineers
ITE	Institute of Transportation Engineers
NCHRP	National Cooperative Highway Research Program
NCTRP	National Cooperative Transit Research and Development Program
NHTSA	National Highway Traffic Safety Administration
NTSB	National Transportation Safety Board
SAE	Society of Automotive Engineers
TCRP	Transit Cooperative Research Program
TRB	Transportation Research Board
U.S.DOT	United States Department of Transportation