Report No. K-TRAN: KU-07-4
FINAL REPORT

DEVELOPMENT OF RECOMMENDED RESISTANCE FACTORS FOR DRILLED SHAFTS IN WEAK ROCKS BASED ON O-CELL TESTS

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March 2010

A COOPERATIVE TRANSPORTATION RESEARCH PROGRAM BETWEEN:

KANSAS DEPARTMENT OF TRANSPORTATION
KANSAS STATE UNIVERSITY
UNIVERSITY OF KANSAS

KANSAS
DEPARTMENT OF TRANSPORTATION

1 Report No. K-TRAN: KU-07-4	2 Government Accession No.	3 Recipient Catalog No.
4 Title and Subtitle Development of Recommended Resistance Factors for Drilled Shafts in Weak Rocks Based on O-Cell Tests		5 Report Date March 2010 6 Performing Organization Code
7 Author(s) Xiaoming Yang, Jie Han, Ph.D., P.E., Robert L. Parsons, Ph.D., P.E.		8 Performing Organization Report No.
9 Performing Organization Name and Address The University of Kansas Department of Civil, Environmental and Architectural Engineering 2150 Learned Hall, 1530 W 15th Street Lawrence, Kansas 66045-7609		10 Work Unit No. (TRAIS) 11 Contract or Grant No. C1616
12 Sponsoring Agency Name and Address Kansas Department of Transportation Bureau of Materials and Research 700 SW Harrison Street Topeka, Kansas 66603-3745		13 Type of Report and Period Covered Final Report July 2006 - February 2010 14 Sponsoring Agency Code RE-0444-01
15 Supplementary Notes For more information write to address in block 9.		

16 Abstract

From October 1, 2007, the new bridges on federal-aid funded projects are mandated to be designed to meet American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications. LRFD is a simplified form of reliability-based design. By multiplying calibrated factors to load and resistance components, the designed structure will maintain a specific level of reliability (or probability of failure). By concept, the load and resistance factors should be calibrated by large number of test data; however, they are often unavailable in geotechnical engineering. Significant efforts are needed to calibrate load and resistance factors based on test data of good quality. In this study, 26 O-Cell test data were collected from Kansas, Colorado, Missouri, Ohio, and Illinois. Seven methods available in the literature were selected to estimate the load capacities of 25 out of 26 drilled shafts. The “FHWA 0.05D” method was found to yield the closest and conservative predictions of the nominal resistances to the representative values; therefore, it was adopted in this study when calibrating the resistance factors for Strength Limit State design. These test data were analyzed and used to calibrate side and base resistance factors for drilled shafts in weak rock.

Resistance factors were calibrated at two different target reliability indices: 2.3 (i.e., failure probability, Pf≈1/100) for shafts with greater redundancy and 3.0 (Pf ≈ 1/1000) for shafts with less redundancy. Side resistance factors were calibrated from two different datasets of measured resistance: total side resistance and layered unit side resistance. The resistance factors calibrated from layered unit side resistance are considered more reliable, therefore, they are recommended for design. The recommended resistance factors from this study are compared with those in AASHTO specifications. Some of those calibrated resistance factors from this study are considerably lower than those in AASHTO specifications. The main reasons for such lower resistance factors are mainly attributed to the low efficiency of the FHWA design method and the limited quality and number of O-Cell test data. These resistance factors may be improved by increasing the size and the quality of the test data in the future. At present, field load tests on drilled shafts are recommended as an alternative to using lower resistance factors, which will also accumulate more test data for future improvement.

17 Key Words	18 Distribution Statement
O-Cell, drilled shafts	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, Virginia 22161

19 Security Classification (of this report)	20 Security Classification (of this page)	21 No. of pages	22 Price
Unclassified	Unclassified	65	

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A Report on Research Sponsored By

THE KANSAS DEPARTMENT OF TRANSPORTATION
TOPEKA, KANSAS

March 2010

PREFACE

The Kansas Department of Transportation's (KDOT) Kansas Transportation Research and New-Developments (K-TRAN) Research Program funded this research project. It is an ongoing, cooperative and comprehensive research program addressing transportation needs of the state of Kansas utilizing academic and research resources from KDOT, Kansas State University and the University of Kansas. Transportation professionals in KDOT and the universities jointly develop the projects included in the research program.

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ABSTRACT

From October 1, 2007, the new bridges on federal-aid funded projects are mandated to be designed to meet American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications. LRFD is a simplified form of reliability-based design. By multiplying calibrated factors to load and resistance components, the designed structure will maintain a specific level of reliability (or probability of failure). By concept, the load and resistance factors should be calibrated by large number of test data; however, they are often unavailable in geotechnical engineering. Significant efforts are needed to calibrate load and resistance factors based on test data of good quality. In this study, 26 O-Cell test data were collected from Kansas, Colorado, Missouri, Ohio, and Illinois. Seven methods available in the literature were selected to estimate the load capacities of 25 out of 26 drilled shafts. The “FHWA 0.05D” method was found to yield the closest and conservative predictions of the nominal resistances to the representative values; therefore, it was adopted in this study when calibrating the resistance factors for Strength Limit State design. These test data were analyzed and used to calibrate side and base resistance factors for drilled shafts in weak rock.

Resistance factors were calibrated at two different target reliability indices: 2.3 (i.e., failure probability, P_f≈1/100) for shafts with greater redundancy and 3.0 (P_f ≈ 1/1000) for shafts with less redundancy. Side resistance factors were calibrated from two different datasets of measured resistance: total side resistance and layered unit side resistance. The resistance factors calibrated from layered unit side resistance are considered more reliable, therefore, they are recommended for design. The

recommended resistance factors from this study are compared with those in AASHTO specifications. Some of those calibrated resistance factors from this study are considerably lower than those in AASHTO specifications. The main reasons for such lower resistance factors are mainly attributed to the low efficiency of the FHWA design method and the limited quality and number of O-Cell test data. These resistance factors may be improved by increasing the size and the quality of the test data in the future. At present, field load tests on drilled shafts are recommended as an alternative to using lower resistance factors, which will also accumulate more test data for future improvement.

ACKNOWLEDGMENTS

This research is financially sponsored by the Kansas Department of Transportation (KDOT) through the K-TRAN program. Mr. Mark Hurt and Mr. Robert Henthorne at KDOT were the project monitors for this project. Mr. William (Bill) Ryan at Loadtest, Mr. Don King at King Construction, Mr. Pete Nix at DLZ Ohio, Inc., Mr. Lyle Simonton at Subsurface Contractors, Inc., and Dr. Michael Yang at Michael Baker Jr. provided part of the O-Cell test data. The authors would like to express their appreciations to all these individuals for their assistance and support.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1 - INTRODUCTION

1.1 Background

Since October 1st 2007, federal-funded projects including new bridges have been mandated to be designed to meet AASHTO LRFD Bridge Design Specifications. The transition from Allowable Stress Design (ASD) to Load and Resistance Factor Design (LRFD) has caused a challenge to geotechnical designers. KDOT engineers have indicated that the design of drilled shafts in weak rocks following the AASHTO LRFD specifications sometimes results in a considerably different design from that according to the original ASD. Designers also have had problems in applying load and resistance factors into their computer programs that are based on ASD.

Weak rock is widely distributed in the state of Kansas. Drilled shafts are the most commonly used foundation type for bridges in such rock formations. In most projects, KDOT has used a serviceability criterion of 0.25 inch settlement to design drilled shafts. To verify the reasonableness of design, Osterberg Cell (O-Cell) load tests have been performed in several projects in Kansas. The test results indicated that measured shaft capacities are often several times higher than those predicted by the FHWA design method (O'Neill and Reese, 1999). The AASHTO LRFD Bridge Design Specifications (AASHTO, 2006) do include recommended resistance factors for drilled shafts in weak rock (the terminology “intermediate geomaterial” used in the AASHTO specifications). However, these resistance factors were converted from typical factors of safety or nationwide load test database, which may not accurately reflect the local conditions and practice in Kansas. Therefore, a research project was funded by KDOT through the K-TRAN research program to evaluate and recalibrate the LRFD resistance factors for drilled shafts based on the properties of the weak rock formations in Kansas and other nearby states using O-Cell test data.

1.2 Scope and Objectives

The overall objectives of this research are to:

- collect O-Cell test data on drilled shafts in weak rocks from Kansas and other nearby states;
- analyze the data and calibrate the side and base resistance factors based on the FHWA design method; and
- develop a design procedure and example to illustrate the application of LRFD resistance factors with the software currently used by KDOT

1.3 Methodology

A literature review was conducted on the topics related to reliability-based design (RBD), LRFD, rock-socketed drilled shafts, and O-Cell tests. Twenty-six sets of O-Cell test data (see Appendix A) were collected from the states of Kansas, Colorado, Missouri, Ohio, and Illinois. Statistical analyses were performed on the O-Cell test data to select an appropriate method to determine measured nominal resistance from a load-displacement curve. Some test data lacking of rock property information were excluded from the calibration because predicted values of resistance cannot be calculated. The side and base resistance factors for drilled shafts in weak rocks were calibrated using the Monte Carlo method described in Transportation Research Circular E-C079 (Allen et al., 2005). Two design scenarios were considered: (a) Strength Limit State and (b) Service Limit State at a settlement of 0.25 inch.

1.4 Organization of this Report

This report is presented in five chapters and one appendix. Chapter One describes the background, scope and objectives, and methodology of this research. Chapter Two is a literature review. Chapter Three presents the statistical analyses on the O-Cell test data to select a reliable method to determine nominal resistance from a measured load-displacement curve. Chapter Four covers the calibration of side and base resistance factors for the Strength Limit State and the Service Limit State at a settlement of 0.25 inch. Chapter Five provides a design example of a vertically loaded drilled shaft socketed in weak rock for the Strength Limit State design and the Service Limit State design at a settlement of 0.25 inch. This example also illustrates how to perform LRFD with the design software Shaft v5.0 for the Strength Limit State. Chapter Six summarizes the key findings of this research. Details about the O-Cell test database are provided in Appendix A.

CHAPTER 2 - LITERATURE REVIEW

2.1 Reliability Based Design and LRFD

Civil engineers deal with uncertainties on a daily basis. In the traditional ASD, uncertainties are accounted for by a factor of safety *FS*. *FS* has a problem of lacking consistency because it does not consider the variability of design parameters in theory of statistics and probability. As Kulhawy and Phoon (1996) stated: "a larger factor of safety doesn't imply a smaller level of risk". From mid 20th century, with an increasing need of safety analysis for modern structures, researchers (e.g., Pugsley (1955) and Freudenthal (1961)) started to introduce a more rational tool to structural design: reliability based design (RBD). Unlike ASD, RBD treats load Q and resistance R components as random variables and targets the designed structure to a particular probability of failure P_f, as shown in Figure 2.1. In practice, reliability index β is more commonly used than P_f. The relationship between P_f and β depends on the distribution type of the performance function and can be estimated if a normal distribution is assumed (Figure 2.2).

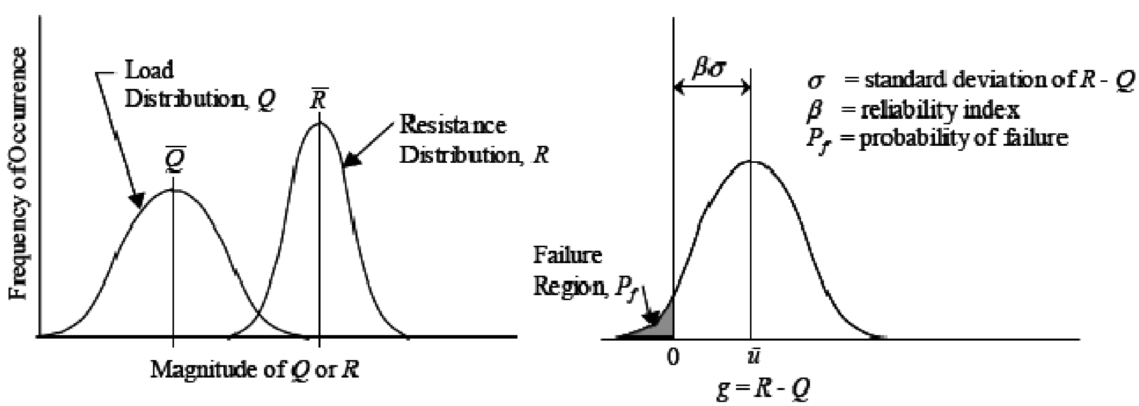


Figure 2.1: Probability of failure P_f and reliability index β (adapted from Withiam et al. 1998)

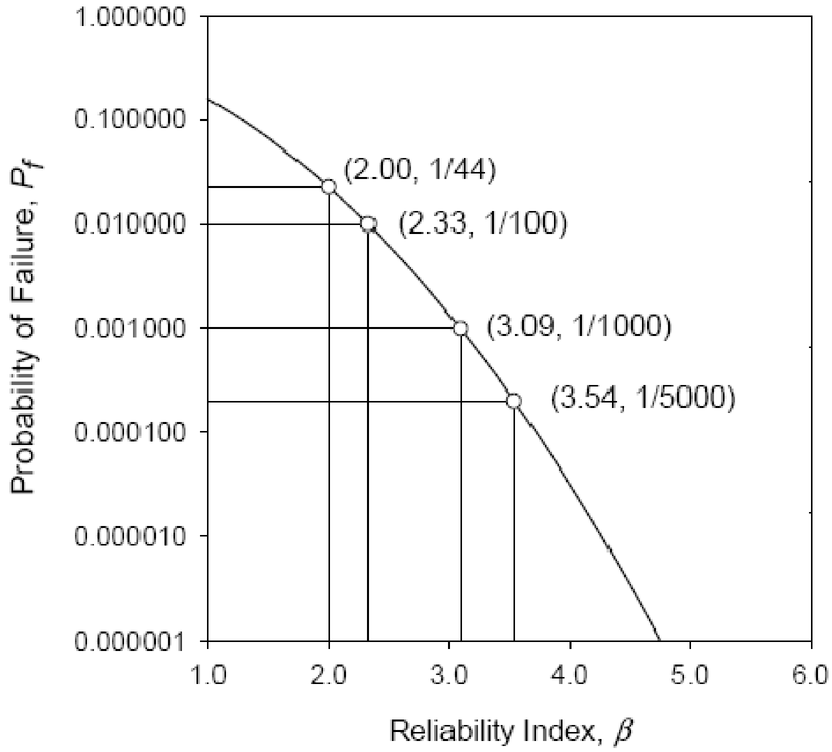


Figure 2.2: Relationship between β and P_f for a normally distributed function of *g* (adapted from Allen et al. 2005)

A rigorous reliability-based design requires a large number of statistical data and is too complicated to be performed. Some researchers (e.g., Hansen (1965) and Lind (1971)) then proposed simplified approaches, such as applying modification factors either to soil properties or to calculated load and resistance components. The latter is also known as load and resistance factor design (LRFD), which has been commonly adopted in North America (Kulhawy and Phoon 1996; 2002). The Strength Limit State design equation of LRFD is shown below:

$$\sum \eta_i \gamma_i Q_i = \phi R_n \qquad \text{Equation 2.1}$$

in which η_i is the load modifier – a factor relating to ductility, redundancy and operational importance, γ_i is the load factor for the i^th load component, Q_i is the i^th nominal load component, ϕ is the resistance factor, R_n is the nominal resistance (either ultimate resistance or resistance mobilized at a given deformation).

With presumed load factors, the resistance factors can be calibrated by either fitting to ASD or with statistical test data. Due to the lack of a reliable database, resistance factors in the early AASHTO specifications for foundation design were calibrated by fitting to ASD in order to keep generally consistent with past practice (Barker et al. 1991). Though some efforts have been made to calibrate the resistance factors using test data (Allen 2005; Paikowsky 2004), more calibration work is still needed.

Several reliability analysis methods are available for the calibration. Currently two of them have gained the widest acceptance: the first order reliability method (FORM) (Ellingwood et al. 1980; Hasofer and Lind 1974; Phoon et al. 1995) and the Monte Carlo method (Allen et al. 2005). In this study, the Monte Carlo method was adopted for its capability of dealing with a combination of different load and resistance components having different types of distributions. The calibration in this study followed the recommended procedures in Transportation Research Circular E-C079 (Allen et al. 2005).

2.2 Drilled Shafts in Weak Rock

Weak rock refers to rock with strength properties between soil and strong rock. Different agencies have given different definitions. The International Society of Rock Mechanics defines it as the rock with an unconfined compressive strength q_u from 0.5 to 25 MPa (10 to 500 ksf). Kulhawy and Phoon (1993) provided classifications of rocks from different reference sources and consider the rock with unconfined compressive strength from 0.5 to 20MPa (10 to 400 ksf) as weak rock. The FHWA design manual (O'Neill and Reese 1999) uses the term "intermediate geomaterial (IGM)" to define a geomaterial with a q_u value from 0.5 to 5MPa (10 to 100 ksf). The term IGM and the corresponding drilled shaft design method are also adopted in the AASHTO (2006) specifications.

Drilled shafts are often used for or as bridge foundations in rock. Earlier studies (e.g., Horvath and Kenney 1979; Rowe and Armitage 1987; Williams et al. 1980) have developed design methods for axially loaded drilled shafts socketed in rocks including weak rocks. Due to the non-uniform and discontinuous characteristics of weak rocks, it is difficult to accurately predict the shaft resistance in such rocks. O'Neill et al. (1996) studied the side resistance of drilled shafts embedded in very weak rocks (i.e., IGM) based on numerical analysis. Their proposed design method is currently adopted by FHWA (O'Neill and Reese 1999) and included in the AASHTO (2006) specifications. However, O-Cell tests conducted in Colorado indicated that the measured side resistance was two to three times higher than that predicted by this method (Abu-Hejleh et al. 2003). Miller (2003) also found that O'Neill and Reese's design method is over-conservative after evaluating six O-Cell test results from Missouri. Paikowsky et al (2004) calibrated the resistance factor for O'Neill and Reese's method based on 91 conventional load tests in IGM. The calibrated resistance factor for drilled shafts in IGM at a target reliability of 3.0 was 0.51 to 0.65 depending on the resistance type (side resistance or a combination of side and base resistance) and the construction technique (dry, slurry, casing or mixed).

Table 2.1 lists the design methods suggested by AASHTO LRFD Bridge Design Specifications (2006) and their corresponding resistance factors. It should be noted from AASHTO (2006) specifications that when a static load test is performed on site, a resistance factor up to 0.70 can be used. AASHTO (2006) also allows the use of other methods and resistance factors calibrated in a manner that is consistent with the development of resistance factors for AASHTO. Turner (2006) reviewed the AASHTO specifications on rock socketed drilled shafts and noticed that some of the resistance factors were calibrated inappropriately. He suggested that alternative design equations for side resistance should be considered and the most up-to-date research should be referenced.

Table 2.1: Summary of design methods and resistance factors included in AASHTO LRFD Bridge Design Specifications (2006)

Method / Soil Condition			ϕ
Nominal axial compressive resistance of single drilled shaft	Side resistance in IGMs	O'Neill and Reese (1999)	0.6
	Tip resistance in IGMs	O'Neill and Reese (1999)	0.55
	Side resistance in rock	Horvath and Kenney (1979)	0.55
		O'Neill and Reese (1999)	0.55
		Carter and Kulhawy (1988)	0.50
	Tip resistance in rock	Canadian Geotechnical Society (1985) Pressuremeter Method (Canadian Geotechnical Society, 1985) O'Neill and Reese (1999)	0.50

2.3 Osterberg-Cell Test on Drilled Shaft

The Osterberg Cell (O-Cell) test was invented by Jorj O. Osterberg and first used in the 1980s (Osterberg 1984). Unlike the conventional top-down load test, the load in this test is applied by a hydraulic cell, which is pre-installed in the shaft somewhere near the tip. This cell will simultaneously produce an upward force to the upper portion of the shaft and a downward force to the lower portion of the shaft at an equal magnitude, which can be used to estimate the side resistance and the base resistance of the shaft separately. O-Cell test is a cost-effective technique especially for testing large diameter drilled shafts. Figure 2.3 shows the comparison between the conventional load test and the O-Cell load test.

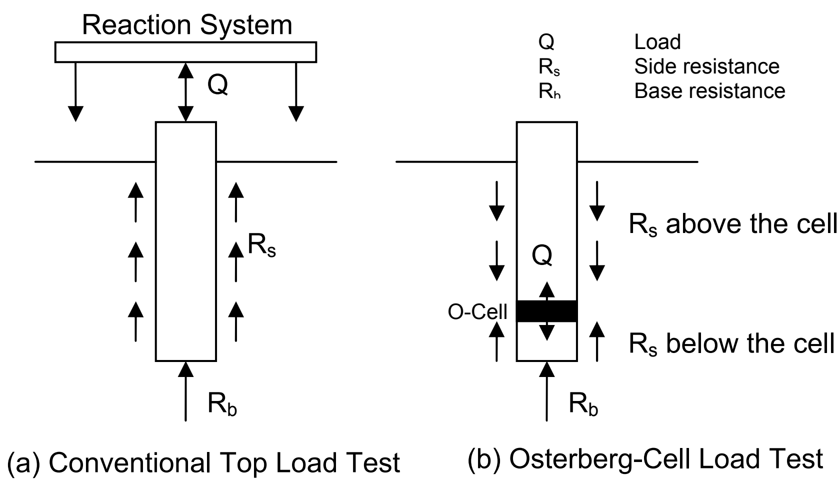


Figure 2.3: Comparison of conventional top load test and O-Cell load test

Figure 2.4 shows a typical test data from an O-Cell test. Upward and downward load-displacement curves are obtained during the test. In order to estimate the ultimate total capacity of the shaft, Osterberg (1998) developed a procedure to obtain an equivalent “top-down” load-displacement curve by assuming that the shaft is incompressible. The detailed procedures are illustrated in Figure 2.4. A reverse procedure can be taken to obtain the ultimate side and base resistance once the ultimate total capacity is determined from the equivalent “top-down” curve.

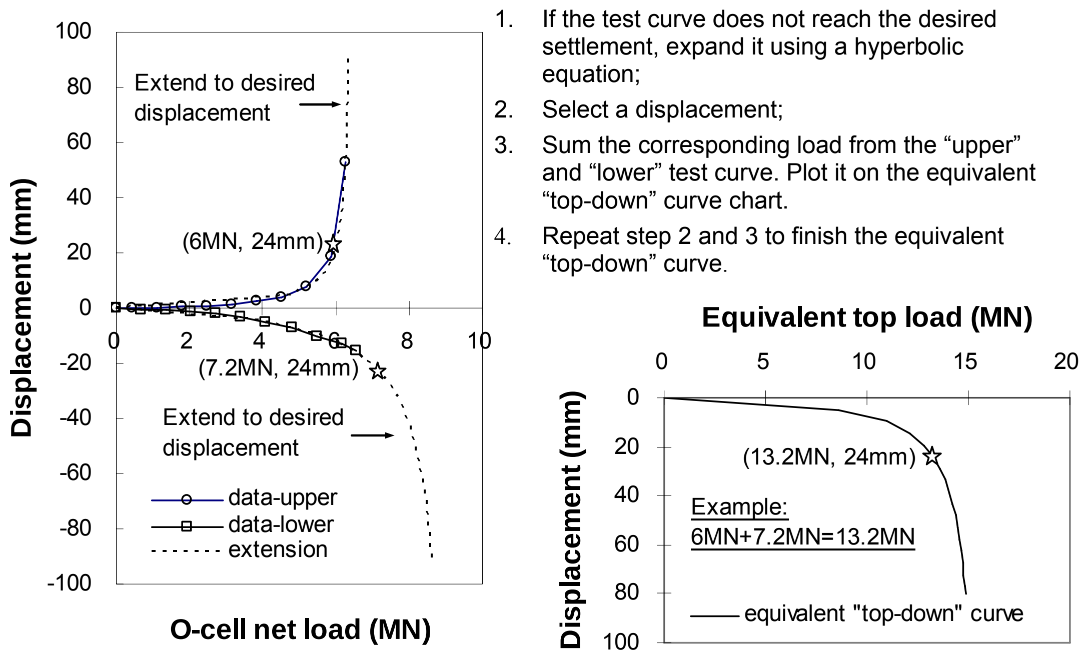


Figure 2.4: Construction process of equivalent “top-down” load-displacement curve

One major issue with the O-Cell test is that whether the equivalent “top-down” curve can reflect the real response of the shaft under axial loading. Some researchers (Ogura, et al. (1996) and Kwon et al. (2005)) carried out O-Cell tests together with conventional load tests at the same site for comparison purpose. In general, it is found that the conventional load test tends to measure a larger settlement especially under a working load, which is believed to be caused by the difference in the shaft compression from that in an O-Cell test. To consider this issue, some modifications were proposed (e.g., Kwon et al. 2005; Ooi et al. 2004) to Osterberg’s “top-down” curve. LOADTEST Inc., the only contractor performing O-Cell test in the US, also has its own model to consider the shaft compression difference (LOADTEST Inc. 2001). This method was adopted in this study for the interpretation of the O-Cell test data.

CHAPTER 3 - STATISTICAL ANALYSES ON O-CELL TEST DATA

3.1 Introduction

Resistance factors for drilled shaft design can be calibrated for either ultimate resistance (or load capacity) or the resistance at a certain displacement (e.g., 0.25 inch). The calibration of the ultimate resistance requires an appropriate method to determine this resistance from a load-displacement curve. This chapter covers the selection of such a method. Review of seven available methods (“Creep limit”, “FHWA 0.05D”, “Davisson’s”, “Brinch-Hansen’s 80%”, “Butler and Hoy’s”, “Fuller and Hoy’s” and “Chin’s”) was first performed in this study. The ultimate side, base, and total resistances were determined using these seven methods for 25 out of 26 collected O-Cell test data collected from the states of Kansas, Colorado, Missouri, Ohio, and Illinois in this study. Statistical analyses were performed to evaluate these methods.

3.2 Review of Methods to Determine Ultimate Shaft Resistance

Table 3.1 lists seven methods which were proposed in the past to determine ultimate load capacities of piles or drilled shafts from load-displacement curves obtained from load tests.

Table 3.1: Seven methods to determine ultimate resistance from a load-displacement curve

Name of method	Reference
Creep Limit	(LOADTEST Inc. 2001)
FHWA 0.05D	(O'Neill and Reese 1999)
Brinch-Hansen’s 80%	(Brinch-Hansen 1963)
Butler and Hoy’s	(Butler and Hoy 1977)
Fuller and Hoy’s	(Fuller and Hoy 1970)
Davisson’s	(Davisson 1972)
Chin’s	(Chin 1970)

O-Cell tests are typically performed according to the requirements of ASTM D1143 (Quick Test Methods). Seven methods as listed in Table 3.1 were selected in this study to determine the nominal load capacity from a load-displacement curve. LOADTEST Inc (2001), the only contractor performing O-Cell tests in the US, uses the “Creep Limit” method. “Creep” here means the displacement developed between the last two displacement readings at each load step, for example, the displacement difference between 4 minutes’ and 8 minutes’ readings. “Creep limit” refers to as the load at which the “creep” has a significant increase. FHWA (O'Neill and Reese 1999) suggested the use of the load corresponding to a displacement of 5% shaft diameter (FHWA 0.05D) if the plunging of the curve is not reached. Other methods include Brinch-Hansen’s 80%, Butler and Hoy’s, Chin’s, Fuller and Hoy’s, and Davisson’s criterion. Paikowsky (2004) examined five different methods and used the mean value as the representative capacity for conventional load tests. By comparison, Paikowsky (2004) suggested the FHWA 0.05D method as the method for the resistance factor calibration. Ooi (2004) also compared different extrapolation equations and capacity criteria and found that the most reliable methods were Chin’s (1970) hyperbolic equation for extrapolation of the load-displacement curve and Davisson’s method for determination of the nominal capacity.

In this study, Chin’s equation (Eq. 3.1) was used for the curve extrapolation except for Brinch-Hansen’s 80% method (Eq. 3.2) because Brinch-Hansen’s 80% method has its own extrapolation equation.

$$\frac{s}{P} = C_1 s + C_2 \qquad \text{Equation 3.1}$$

$$\frac{\sqrt{s}}{P} = C_1 s + C_2 \qquad \text{Equation 3.2}$$

where P and s are the measured load and the settlement, respectively and C_1 and C_2 are the regression coefficients.

3.3 Interpretation of Ultimate Resistance

The seven methods discussed above were applied to all the collected O-Cell tests in Appendix A with the following exceptions. Test Nos. 9 – 14 were collected without any “creep” data, so they were interpreted by six methods except the “creep limit” method. Test No.19 is a triple-cell test with three O-Cells installed along the shaft, which is the single practice in the United States so far, therefore, the method illustrated in Figure 2.4 is not applicable to this kind of test. As a result, Test No. 19 was excluded from this statistical analysis but included the calibration of resistance factors. Figures 3.1 to 3.3 present the calculated side resistance for all the other 25 shafts based on these seven methods. It is shown that these methods yielded results with certain differences. The “Creep Limit” method consistently predicted the lowest ultimate resistance value and Chin’s method always resulted in the highest value. The ultimate resistance based on the “Creep Limit” method was determined using the measured data without any extrapolation. This method is over-conservative especially when the test is terminated before a full mobilization of either side or base resistance. Chin’s method mathematically calculates the ultimate load capacity when the hyperbolic curve reaches an infinite displacement; therefore, it over-predicts the load capacity. As a result, these two methods were not included in the subsequent analysis. The mean value of the resistance from the remaining five methods was taken as the representative capacity of the shaft. Figures 3.1, 3.2, and 3.3 present the calculated ultimate side resistance, ultimate base resistance, and ultimate total resistance for all 25 shafts, respectively. As shown in Figure 3.3, the calculated ultimate total resistance ranges from 3.6 to 291.9 MN (809 to 65622 kips).

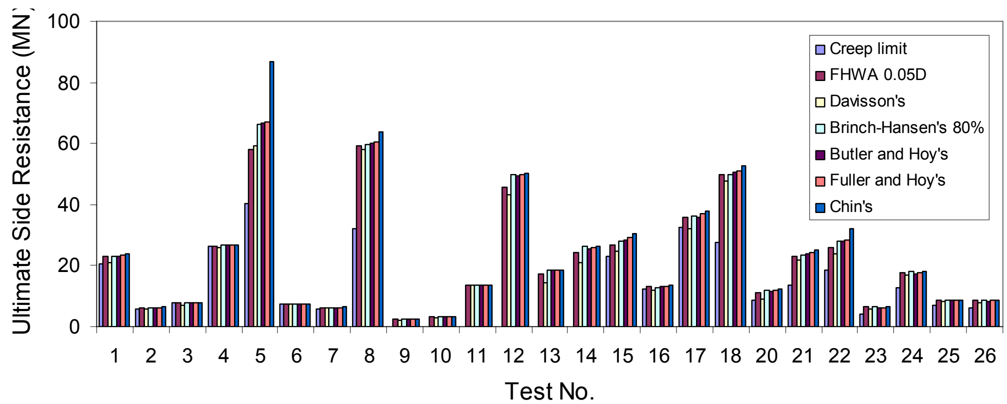


Figure 3.1: Calculated ultimate side resistance

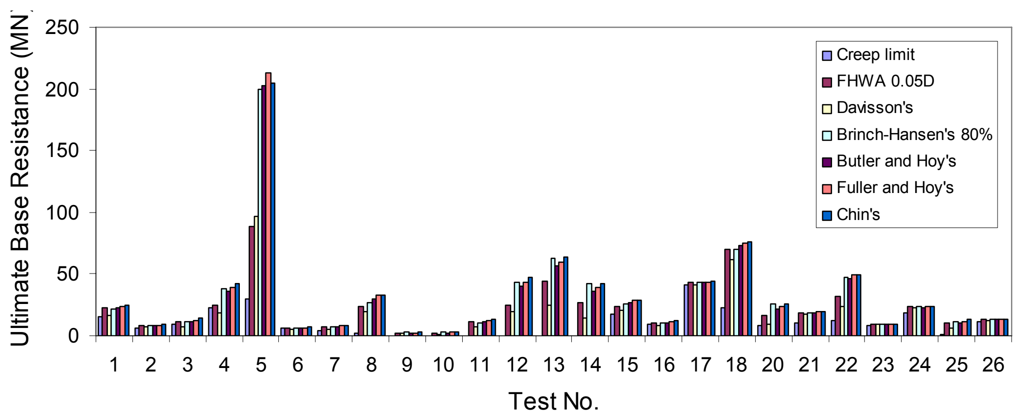


Figure 3.2: Calculated ultimate base resistance

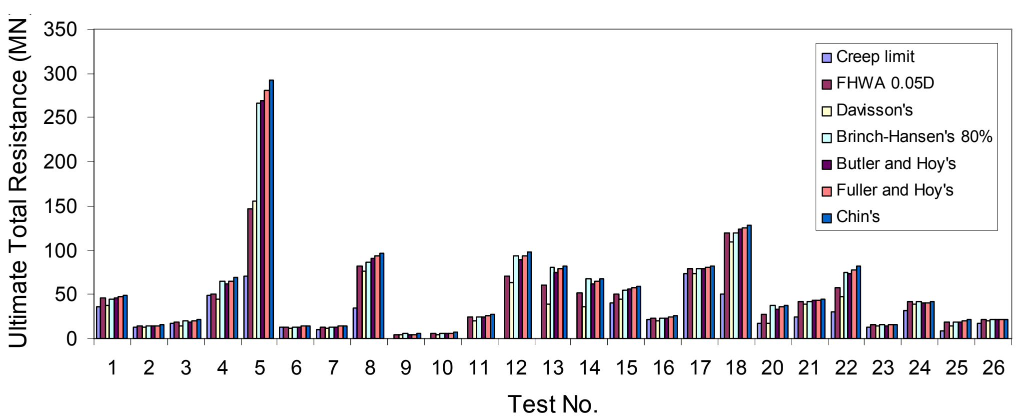


Figure 3.3: Calculated ultimate total resistance

3.4 Statistical Analyses

A concept “bias” was used in this study to evaluate the remaining five methods (i.e., FHWA 0.05D, Davisson’s, Brinch-Hansen’s 80%, Butler and Hoy’s, and Fuller and Hoy’s criteria). “Bias” is defined as the ratio of the resistance by each interpretation criterion over the representative capacity (Paikowsky 2004). This approach is similar to what Paikowsky (2004) did except that the side and base resistance were examined separately in this study.

The calculated “bias” values for all five methods were statistically analyzed and the results are provided in Table 3.2. It is shown that except Davisson’s method, the other four methods are relatively reliable and comparable. The comparison shows that the FHWA 0.05D method yielded a side resistance prediction equal to the representative value with a low standard deviation of 0.03. Since Butler and Hoy’s method had the lowest COV values for all three capacities, it is considered the most reliable method from the statistical point of view. However, Butler and Hoy’s method overestimated the capacities as compared with the representative values. FHWA 0.05D method yielded the closest and conservative mean values of the side, base, and total load capacity to representative values. Therefore, the FHWA 0.05D method was selected in this study for future resistance factor calibration.

Table 3.2: Statistical results based on five interpretation criteria

	FHWA 0.05D			Davisson’s			Brinch-Hansen’s 80%			Butler and Hoy’s			Fuller and Hoy’s		
	Side	Base	Total	Side	Base	Total	Side	Base	Total	Side	Base	Total	Side	Base	Total
m	1.00	0.95	0.97	0.92	0.72	0.82	1.03	1.11	1.07	1.01	1.07	1.04	1.03	1.15	1.09
σ	0.03	0.13	0.09	0.05	0.16	0.11	0.03	0.11	0.07	0.02	0.08	0.05	0.02	0.09	0.06
COV	0.03	0.14	0.09	0.06	0.22	0.14	0.03	0.10	0.07	0.02	0.07	0.05	0.02	0.08	0.06

Note: m=mean value; σ=standard deviation; COV=coefficient of variation.

3.5 Summary

Twenty-six O-Cell test data were collected from the states of Kansas, Colorado, Missouri, Ohio, and Illinois in this study for drilled shafts in rocks. Seven methods available in the literature were selected to estimate the load capacities of 25 out of 26 drilled shafts. Calculated load capacities from five methods (FHWA 0.05D, Davisson's, Brinch-Hansen's 80%, Butler and Hoy's, and Fuller and Hoy's methods) were used for statistical analyses. The comparison showed that Butler and Hoy's method is the most reliable method. However, the FHWA 0.05D method provided the closest and conservative predictions of the ultimate resistance to the representative values. Therefore, the ultimate resistance determined by FHWA 0.05D method is recommended for resistance factor calibration for Strength Limit State design.

CHAPTER 4 - CALIBRATION OF RESISTANCE FACTORS BASED ON O-CELL TEST DATA

4.1 Scope

This chapter covers the calibration of resistance factors for drilled shafts in weak rock based on O-Cell test data. In this study, the FHWA design method (O'Neill and Reese 1999) for the intermediate geomaterial (IGM) was selected as the design method for weak rock. Although the design method limits the rock unconfined compressive strength q_u from 0.5 to 5 MPa (10 to 100 ksf), the upper limit, as indicated by O'Neill et al. (1996), was selected arbitrarily. Kulhawy and Phoon (1993) provided classifications of rock material strength from different reference sources. From that list, it appears that the rock with unconfined compressive strength from 0.5 to 20MPa can be considered as weak rock. In order to include enough test data necessary for a reliability analysis, the FHWA design method was extended to predict drilled shaft resistance in weak rock ($0< q_u \leq 20$MPa), with some extrapolation in this study. The O-Cell test database contains 26 test data collected from the state of Kansas, Colorado, Missouri, Ohio, and Illinois (see Appendix A). Seven of them were excluded from the calibration due to inadequate information to predict the shaft capacity. The remaining test data were used to calibrate the resistance factors for side and base resistance following the Monte Carlo method suggested by FHWA (Allen et al. 2005). Both the Strength Limit State and Service Limit State (0.25in) design were considered.

4.2 FHWA Design Method

The FHWA design method for cohesive intermediate geomaterials was based on O'Neill and Reese's research report (O'Neill et al. 1996). Design equations for smooth socket were used in this study because they are more commonly used in practice than rough socket equations. Details about this method can be found in Chapter 5 of this report, where a design example is provided. The O-Cell data used in calibration were from weak rock with a wider range of unconfined compressive strengths (0.4MPa to 20.2MPa) than those for IGMs (0.5MPa to 5MPa). This decision was made based on the fact of limited available data for IGMs, which were not sufficient to conduct a reliability analysis. By expanding the range of weak rock included in the analysis, a reliability analysis became possible. In addition, Kulhawy and Phoon (1993) provided classifications of rock material strength from different reference sources. From that list, it appears that the rock with unconfined compressive strength from 0.5 to 20MPa can be considered as weak rock. Since O'Neill and Reese's method is limited to the IGMs with an unconfined compressive strength of 0.5 to 5MPa (somewhat arbitrary upper limit as indicated by O'Neill et al. (1996)), extrapolations are necessary for estimation of side resistance of drilled shafts in the rock beyond this range. The extension of this range may increase the variability of the calculated results. However, it also increases the number of datasets, which in turn increase the reliability of the calibrated resistance factors. Since the variability of the calculated results from the wider range is included in the calibrated resistance factors, they are still valid. It is also the beauty of the reliability method. The extrapolations involved one main parameter in O'Neill and Reese's design method: the *α* coefficient for side resistance. The *α* coefficient for side resistance is used in the following equation:

$$f_a = \alpha q_u \qquad \text{Equation 4.1}$$

where f_a is the ultimate unit side resistance along the drilled shaft and

q_u is the unconfined compressive strength of weak rock.

The α coefficient is determined using the following equation:

$$\alpha = (5 - 8.8\lambda)(q_u / \sigma_p)^{\lambda - 1} \qquad \text{Equation 4.2}$$

where $\lambda = (15 - \sigma_n / \sigma_p)/27$,

σ_n is the normal stress at the borehole wall before loading the shaft,

σ_p is the atmospheric pressure, and

q_u is the unconfined compressive strength of the rock.

Since λ is always less than 1, a higher q_u value would yield a lower α value, which is conservative for a weak rock having a higher q_u value. In other words, the extension of O'Neill and Reese's design method to weak rock with a q_u greater than 5 MPa would yield a conservative side resistance value.

In this study, the following assumptions were also made: (a) cohesive weak rock, (b) smooth rock sockets, (c) closed joints, (d) if the elastic modulus of the core sample was not available, the suggested relationships by O'Neill et al. (1996), $E_i = 250q_u$, for Argrillaceous geomaterial and $E_i = 115q_u$ for Calcareous rock were adopted, and (e) concrete had slump of 175mm and unit weight of 20.4 kN/m^3 if no information was available.

4.3 Information Needed for Resistance Factor Calibration

The calibration in this study followed the recommended procedures in Transportation Research Circular E-C079 (Allen et al. 2005). Strength I limit state and

Service I limit state were considered in this study. Since only live load and dead load components are involved in both situations, the limit state design equation can be written as:

$$g = \gamma_{LL} LL + \gamma_{DL} DL - \phi R \geq 0 \qquad \text{Equation 4.3}$$

where g is the safety margin;

LL and DL are the nominal live and dead loads,

γ_{LL} and γ_{DL} are the live load and dead factors,

R is the nominal resistance, and

ϕ is the resistance factor.

To calibrate the resistance factors, the statistical characteristics of load components are necessary. In this study, both live and dead loads were assumed to be normally distributed. The statistical characteristics and load factors of live and dead loads listed in Table 4.1 were used in the calibration in this study. The statistical parameters are from Nowak's study (Nowak 1999). Load factors for Strength I limit and Service I limit design were adopted from AASHTO LRFD Specifications (2006).

Table 4.1: Statistical characteristics and load factors of live and dead loads

	Bias	COV	Load factor	
			Strength I	Service I
Live load	$\lambda_{LL} = 1.15$	$COV_{LL} = 0.2$	$\gamma_{LL} = 1.75$	$\gamma_{LL} = 1.00$
Dead load	$\lambda_{DL} = 1.05$	$COV_{DL} = 0.1$	$\gamma_{DL} = 1.25$	$\gamma_{DL} = 1.00$

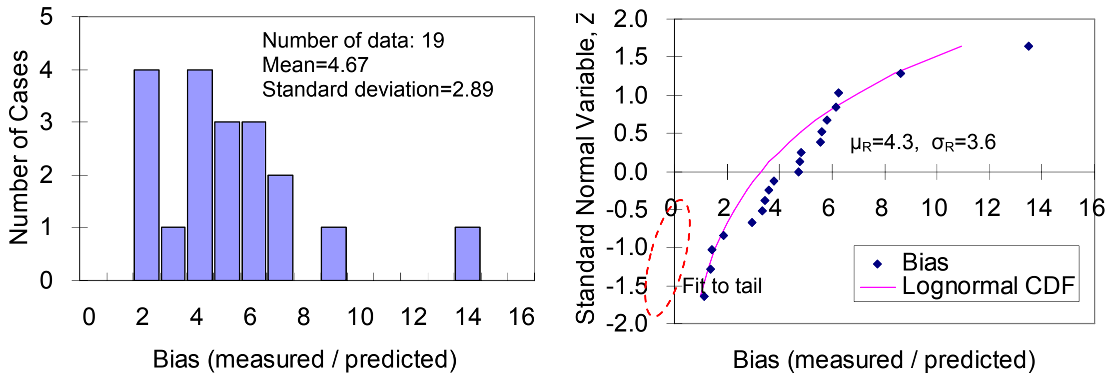
Note: Bias is the mean value of the measured/predicted load.

COV is the Coefficient of Variation, which is the ratio of the standard deviation over the mean value

Measured resistance from the O-Cell tests played an essential role in the calibration of resistance factors. For the Strength Limit State design, the measured ultimate side resistance was used. As discussed in Chapter 3, the side resistance at a displacement equal to 5% of the shaft diameter was taken as the ultimate side resistance. For the Service Limit State design, the measured resistance at a displacement of 0.25 inch was used.

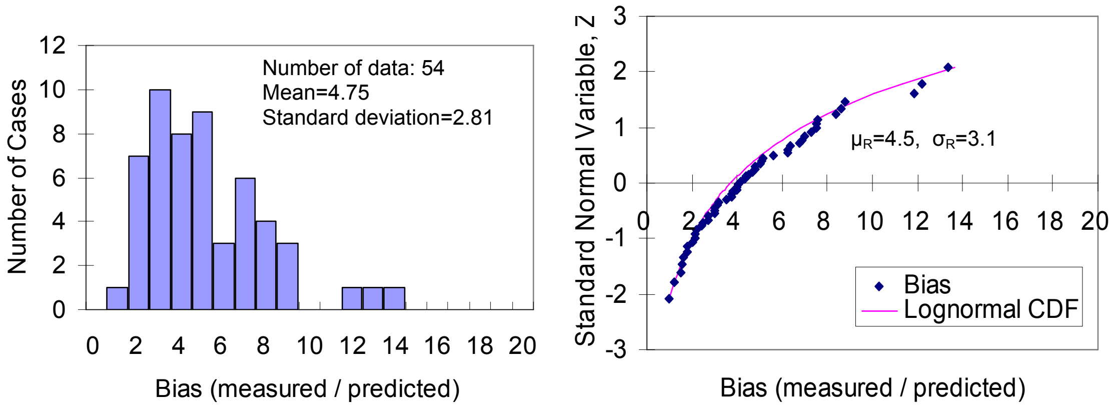
In O-Cell tests, side resistance can be obtained in two ways: (a) the total side resistance derived from the O-Cell upper load-displacement curve and (b) the unit side resistance in each layer from the strain gauge data. Strain gauges are used to measure the axial stress distributions along the shafts in the O-Cell tests. The unit side resistance in the layer between two strain gauges can be determined from the axial stress difference divided by the shaft surface area between the two gauges. Typically, each shaft has several strain gauges installed at the boundary between different rock formations; therefore, the second approach has more data points and makes the analyses more reliable. Predicted values of side resistance were calculated using O'Neill and Reese's method. Accordingly, side resistance in each sub-layer was also calculated in this study.

The bias λ for each drilled shaft was calculated by dividing the measured side resistance from the O-Cell data by the corresponding predicted resistance using O'Neill and Reese's method. Note the concept of this bias is different from the one used in Chapter 3. The same name is used here only to keep it consistent with that used in the literature. Statistical analysis on the λ values was then performed. Figures 4.1 to 4.6 show the histograms and the cumulative distribution function (CDF) curves of the bias values of the total side resistance, unit side resistance and base resistance for Strength Limit State and Service Limit State, respectively. Based on the shapes of all the histograms, the bias of resistance can be assumed to be log-normally distributed. Figures 4.1(a), 4.2(a), 4.3(a) and 4.4(a) show that the mean biases for the side resistance range from 4.67 to 5.63 and the mean biases for base resistance range from 4.16 to 4.67. These high biases imply that the FHWA method significantly underestimated the actual capacities of drilled shafts in weak rock in both limit states. One of the reasons for the underestimated side resistance may be attributed to the assumption of smooth sockets, which is commonly used in practice. In reality, sockets may be rough or between smooth and rough. Unfortunately, not enough information was available in the O-Cell test reports to make such an assessment. The coefficient of variation (COV) of the bias for all the six situations ranged from 0.69 to 1.30, which indicates that FHWA design method has relatively low efficiency in determining side and base resistance of drilled shafts in weak rock. Note the mean values and the standard deviation values in Figures 4.1(a) to 4.6(a) were not used in the calibration directly. Instead, the mean value μ_R and the standard deviation σ_R in Figures 4.1(b) to 4.6(b) were used to fit the CDF curves following a "fit to tail" strategy recommended by Allen et al. (2005). Detailed procedures to develop the standard normal variables z and the CDF plots can be found in Transportation Research Circular E-C079 (Allen et al. 2005). The "best fit" lognormal distribution parameters used in the calibration were summarized in Table 4.2.



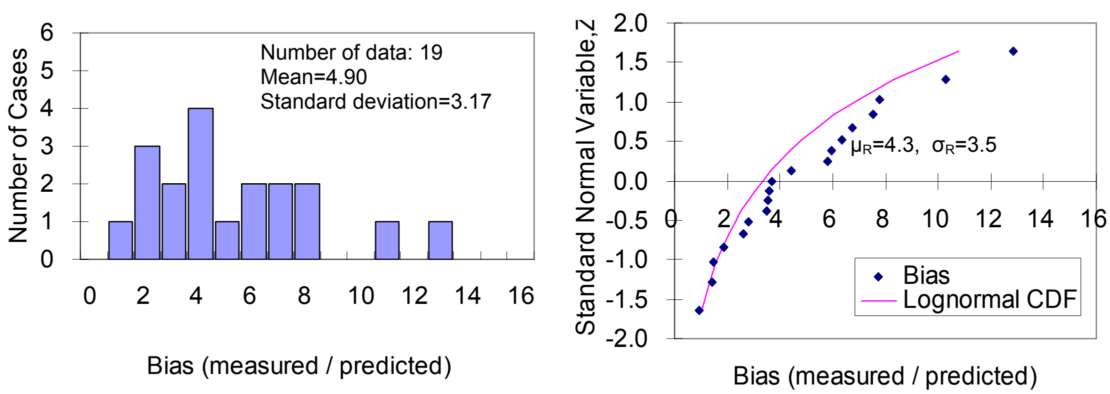
(a) (b)

Figure 4.1: Histogram and CDF plot of the bias of total side resistance - strength limit



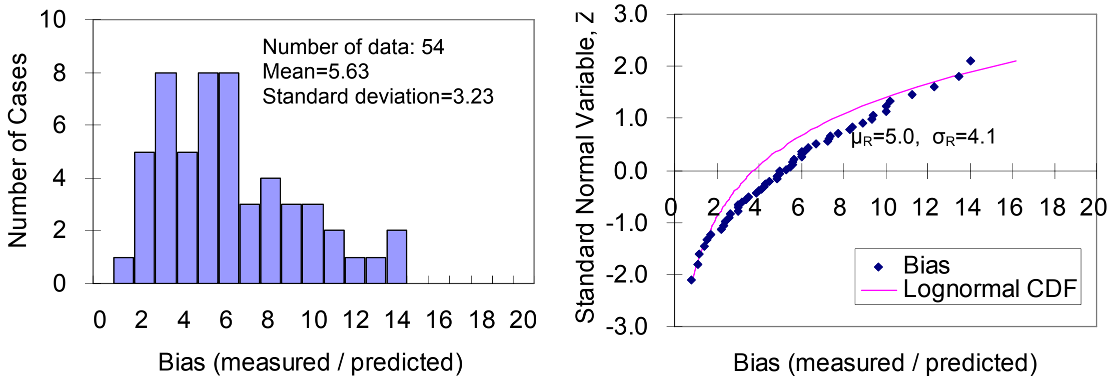
(a) (b)

Figure 4.2: Histogram and CDF plot of the bias of layered side resistance - strength limit



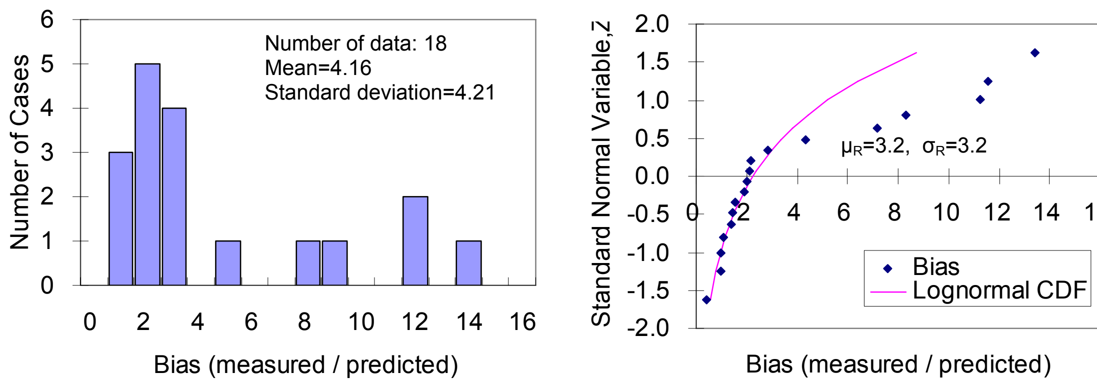
(a) (b)

Figure 4.3: Histogram and CDF plot of the bias of total side resistance - service limit



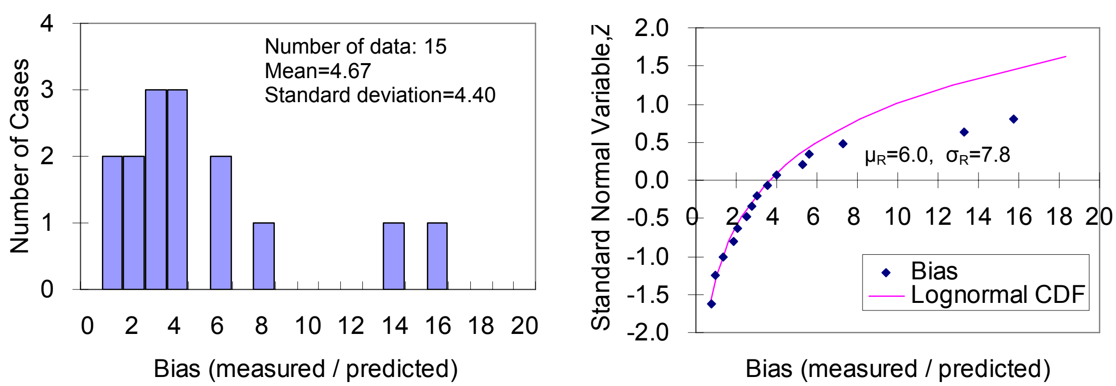
(a) (b)

Figure 4.4: Histogram and CDF plot of the bias of layered side resistance - service limit



(a) (b)

Figure 4.5: Histogram and CDF plot of the bias of total base resistance - strength limit



(a) (b)

Figure 4.6: Histogram and CDF plot of the bias of total base resistance - service limit

Table 4.2: Distribution parameters for the Monte Carlo simulation

Situation			μ_R	σ_R	COV_R
Side resistance	Strength Limit State	Total	4.3	3.6	0.84
		"sub-region"	4.5	3.1	0.69
	Service Limit State	Total	4.3	3.5	0.81
		"sub-region"	5.0	4.1	0.82
Base resistance	Strength Limit State	-	3.2	3.2	1.00
	Service Limit State	-	6.0	7.8	1.30

4.4 Monte Carlo Simulation

Prior to the Monte Carlo simulation, a target reliability index β_T and a ratio of dead load to live load (DL/LL) must be selected. To be consistent with the AASHTO LRFD Specifications, $\beta_T = 3.0$ for a common design and $\beta_T = 2.3$ for a shaft group with five or more drilled shafts were considered. Paikowsky et al. (2004) found out that the calibrated resistance factor is not sensitive to the change of DL/LL. So in this study the DL/LL ratio of 2.0 was selected.

In the Monte Carlo simulation, the program generated 50,000 groups of random numbers. Each group consisted of 3 random numbers a, b, and c. a, b, and c are normally distributed from 0 to 1. The program then calculated random live load LL_{rnd}, dead load DL_{rnd}, and resistance R_{rnd} using the following equations:

$$LL_{rnd} = LL \cdot \lambda_{LL} \cdot (1 + a \cdot COV_{LL}) \qquad \text{Equation 4.4}$$

$$DL_{rnd} = DL \cdot \lambda_{DL} = LL \cdot \frac{DL}{LL} \cdot \lambda_{DL} \cdot (1 + b \cdot COV_{DL}) \qquad \text{Equation 4.5}$$

$$R_{rnd} = EXP(\mu_{\ln} + c \cdot \sigma_{\ln}) \qquad \text{Equation 4.6}$$

where $$\mu_{\ln} = Ln(\mu_R \cdot \frac{LL \cdot \gamma_{LL} + DL \cdot \gamma_{DL}}{\phi}) - 0.5\sigma_{\ln}$$

$$\sigma_{\ln} = \left\{Ln\left[COV_R{}^2 + 1\right]\right\}^{0.5}$$

The above equations were rewritten from those in the Transportation Research Circular E-C079 (Allen et al. 2005). ϕ is a trial resistance factor. LL and DL are the nominal live load and dead load. γ_{LL} , γ_{DL} λ_{LL} , λ_{DL} , COV_{LL} and COV_{DL} are explained in Table 4.1. The values for μ_R and COV_R are listed in Table 4.2. Note the only unknown variable is the nominal live load LL . In the Monte Carlo simulation, the magnitude of the nominal load would not affect the result so that LL was simply set to one. From each group of random loads and resistance, the safety margin was calculated using Equation 4.7.

$$g = R_{rnd} - LL_{rnd} - DL_{rnd} \qquad \text{Equation 4.7}$$

The probability of failure is the number of the failed cases ($g < 0$) divided by the total number of the cases generated:

$$P_f = \frac{N_{g<0}}{50{,}000} \qquad \text{Equation 4.8}$$

Finally, the reliability index β estimated by an Excel function was obtained

$$\beta = -NORMSINV(P_f) \qquad \text{Equation 4.9}$$

NORMSINV is the inverse of the standard normal cumulative distribution and available in MS Excel spreadsheet. The mathematical expression of this function is complicated and omitted here. Details of this function can be found in the Circular E-C079 (Allen et al. 2005). If the calculated β value is different from the target reliability index β_T , the trial resistance factor must be changed and iterations are necessary until $\beta = \beta_T$. The corresponding resistance factor is the one calibrated from this procedure.

4.5 Calibrated Resistance Factors for Drilled Shaft Design

The resistance factors calibrated (round to 0.05) from the O-Cell test data were summarized in Table 4.3. Since the side resistance factor from layered unit side resistance was calibrated from 54 data points, it is more reliable than that from the total side resistance (only 19 data points). In fact, drilled shafts are always designed based on layered rock properties, so the resistance factor from layered unit side resistance data are recommended for designers to use. Table 4.3 also indicates that the resistance factors for the Strength Limit State are higher than those for the Service Limit State. This result is because the load factors used in the Service Limit State design are less than those used in the Strength Limit State design. More deduction has to be made to nominal resistance since load components are not amplified.

It should also be noted that the resistance factors calibrated here are much less than those recommended by the AASHTO. The main reason for the lower resistance factors is the low efficiency of the FHWA design method, as indicated by the wide distributed biases especially for the base resistance. Another possible reason is the limited volume of database, from which it is hard to differentiate outlier datasets from the ordinary ones. Future efforts are needed to increase the size and improve the quality of the test database. At present, performing load tests on test shafts is also recommended to improve the efficiency of design. The AASHTO allows resistance factors of no more than 0.70 to be applied to the measured total resistance from field load tests.

Table 4.3: Calibrated resistance factors

Situation			φ (β_T=3.0)	φ (β_T=2.3)	AASHTO (β_T=3.0)
Side resistance	Strength Limit State	Total	0.50	0.80	0.6
		Layered	0.70	1.00	
	Service Limit State	Total	0.35	0.60	1.00
		Layered	0.40	0.65	
Base resistance	Strength Limit State	-	0.25	0.45	0.55
	Service Limit State	-	0.15	0.35	1.00

4.6 Summary

This chapter provides the details regarding the calibration of side and base resistance factors for drilled shaft using the O-Cell test data. Strength I Limit State and Service I Limit State were considered in this study. Side and base resistance factors were calibrated at two different target reliability indices: 3.0 and 2.3, corresponding to different design scenarios. Side resistance factors were calibrated from two different sources of measured resistance: total side resistance and layered unit side resistance. Since the layered unit side resistance source had more datasets, the resistance calibrated from the layered measured unit side resistance is considered more reliable, thus recommended for designers to use. Some calibrated resistance factors are considerably less than those in the AASHTO specifications. The lower resistance factors are caused by the low efficiency of the FHWA design method. In other words, the FHWA design method is not so reliable (based on the test data collected) in predicting the load capacity of the drilled shafts in weak rock, thus a low resistance factor has to be applied to account for the variation between the predicted capacity and the actual capacity of a drilled shaft. It is also partly resulted from the limited size of the available test database. This result can be further improved by increasing the size and improving the quality of the database in the future. At present, as an alternative of using low resistance factors, field load tests on the drilled shafts are also recommended.

CHAPTER 5 - DESIGN EXAMPLE

5.1 Scope

In this chapter, a design example is provided to explain the LRFD design of drilled shafts in weak rock using the calibrated resistance factors obtained in this study. The example consists of a Strength Limit State design and a Service Limit State design. The design was carried out by both hand calculation and design software Shaft V5.0.

5.2 Design Scenario

A single drilled shaft is to be designed in cohesive weak rock (the profile shown in Figure 5.1). Specify the concrete unit weight $\gamma = 130$ pcf, Young's modulus $E_c = 4000$ ksi, and slump =7inch (175mm). The vertical nominal (unfactored) live load and dead load are LL = 100 kips and DL = 400 kips. The drilled shaft will be designed by (1) Strength Limit State and (2) Service Limit State at a settlement of 0.25 inch.

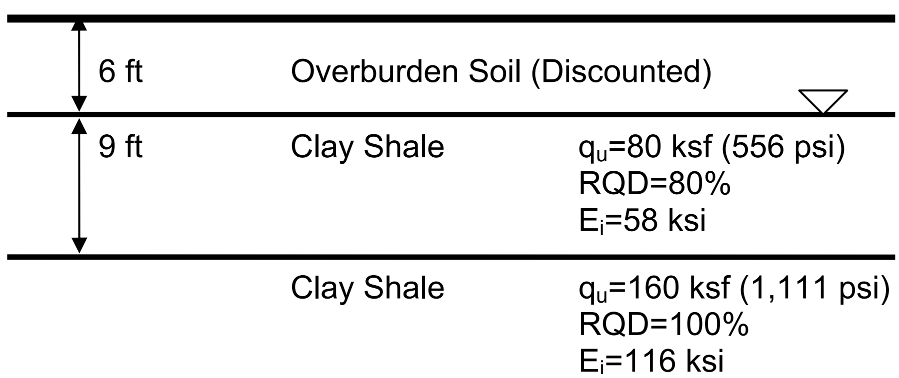


Figure 5.1: Geotechnical profile

5.3 Design Example – Strength Limit State Design

The procedure for the Strength Limit State design is illustrated in Figure 5.2. The formula are used in this design example can be found in the literature (O'Neill and

Reese 1999). Hand calculation and input and output from Shaft V. 5.0 software are presented below.

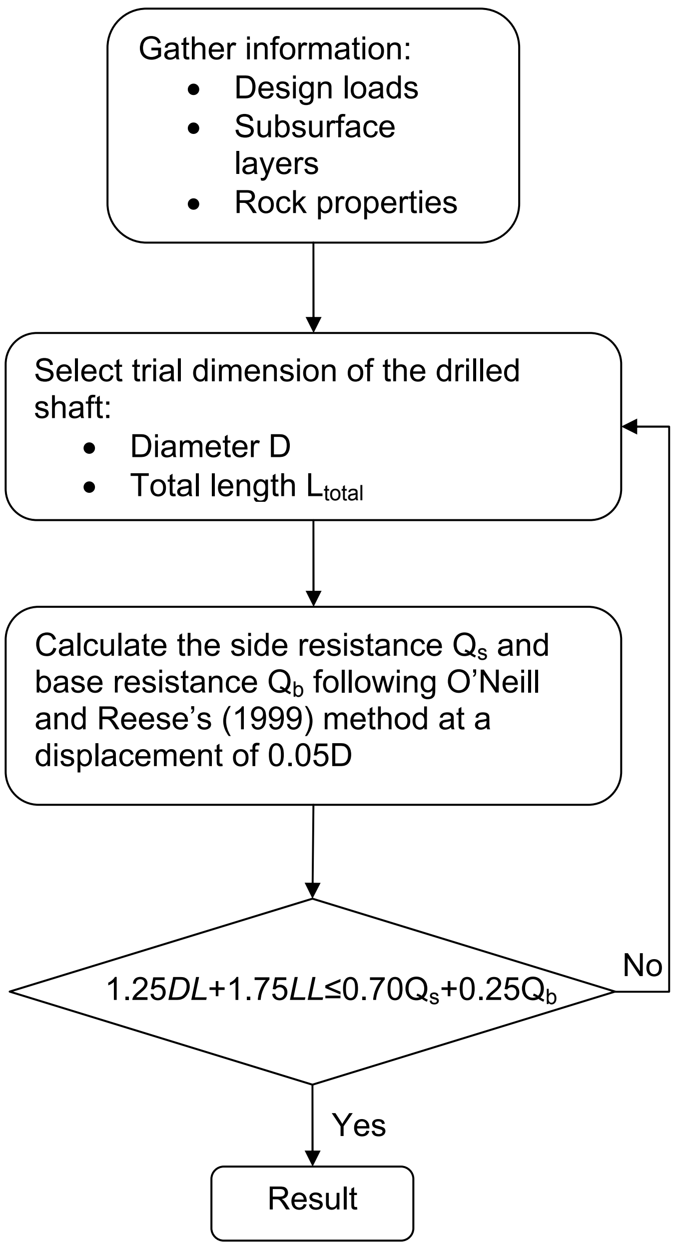


Figure 5.2: Procedure of strength limit state design

5.3.1 Hand Calculation

1. In a Strength Limit State design, the factored load is 1.25 DL + 1.75 LL =1.25 $\times$ 400 + 1.75 $\times$ 100 = 675 ton.
2. A trial dimension has to be assumed. Here we try shaft diameter D = 4 ft and shaft length L_{total} = 20 ft. The shaft length in rock L = 20 – 6 = 14 ft by ignoring the overburden soil.
3. Calculate the normal stress between the concrete and the borehole wall σ_n for each layer:
   a. Layer 1: z_c = 10.5 ft (3.2 m) (depth from the top of the concrete to the mid-depth of this layer). From Figure 5.3, M = 0.98. The normal stress between the concrete and the borehole wall $\sigma_n = M\gamma'_c z_c$ = 0.98 $\times$ 130 $\times$ 6 + 0.98 $\times$ (130 – 62.4) $\times$ 4.5ft = 1,062 psf = 7.37 psi.
   b. Layer 2: z_c =17.5ft (5.3 m) (depth from the top of the concrete to the mid-depth of this layer). From Figure 5.3, M = 0.9. The normal stress between the concrete and the borehole wall $\sigma_n = M\gamma'_c z_c$ = 0.9 $\times$ 130 $\times$ 6 + 0.9 $\times$ (130 – 62.4) $\times$ 11.5 = 1,402 psf = 9.74 psi.

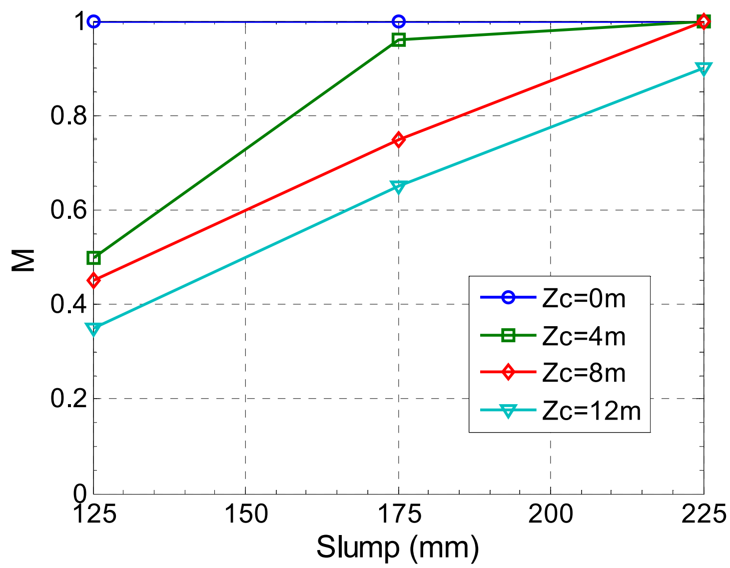


Figure 5.3: Factor M vs. concrete slump (O'Neill and Reese 1999)

4. Calculate α.

   Layer 1: $\lambda = (15 - \sigma_n/\sigma_p)/27 = (15 – 7.37/14.7)/27 = 0.54$. $\alpha = (5 - 8.8\ \lambda)(q_u/\sigma_p)^{\lambda-1} = (5 - 8.8\times0.54) \times (556/14.7)^{0.54-1} = 0.047$.

   Layer 2: $\lambda = (15 - \sigma_n/\sigma_p)/27 = (15 – 9.74/14.7)/27 = 0.53$. $\alpha = (5 - 8.8\ \lambda)(q_u/\sigma_p)^{\lambda-1} = (5 - 8.8 \times 0.53) \times (1{,}111/14.7)^{0.53-1} = 0.044$.

5. Calculate E_m, f_a and f_{aa}.

   Layer 1: $f_a = \alpha q_u = 0.047 \times 80 = 3.76$ ksf = 26.1 psi. Since RQD = 80%, from Tables 5.1 and 5.2, assuming close joints, $f_{aa} = 0.92 \times 26.1 = 24.0$ psi, $E_m = 0.8E_i = 0.8 \times 58 = 46.4$ ksi.

   a. Layer 2: $f_a = \alpha q_u = 0.044 \times 160 = 7.04$ ksf = 48.9 psi. Since RQD = 100%, from Tables 5.1 and 5.2, assuming close joints, $f_{aa} = f_a = 48.9$ psi, $E_m = E_i = 116$ ksi.

Table 5.1: Estimation of E_m/E_i based on RQD (O'Neill and Reese, 1999)

RQD (percent)	E_m/E_i	
	Closed joints	Open joints
100	1.00	0.60
70	0.70	0.10
50	0.15	0.10
20	0.05	0.05

Table 5.2: Adjustment of f_a for presence of soft seams (O'Neill and Reese, 1999)

E_m/E_i	f_{aa}/f_a
1.0	1.0
0.5	0.8
0.3	0.7
0.1	0.55
0.05	0.45

6. f_{aa} (weighed avg.) = $(24.0 \times 9 + 48.9 \times 5)/14 = 32.9$ psi.

   E_m (weighed avg.) = $(46.4 \times 9 + 116 \times 5)/14 = 71.3$ ksi.

7. $\Omega = 1.14(L/D)^{0.5} - 0.05[(L/D)^{0.5} - 1]\log_{10}(E_c/E_m) - 0.44 = 1.14 \times (14/4)^{0.5} - 0.05 \times [(14/4)^{0.5} - 1] \times \log_{10}(4{,}000/71.3) - 0.44 = 1.62$.

   $\Gamma = 0.37(L/D)^{0.5} - 0.15[(L/D)^{0.5} - 1]\log_{10}(E_c/E_m) + 0.13 = 0.37 \times (14/4)^{0.5} - 0.15 \times [(14/4)^{0.5} - 1] \times \log_{10}(4{,}000/71.3) + 0.13 = 0.59$.

8. $\Theta_f = E_m\Omega w_t/(\pi L\Gamma f_{aa}) = 71{,}300 \times 1.62/(3.14 \times 14 \times 0.59 \times 32.9)\ w_t = 135.4\ w_t$, w_t is the displacement of the shaft measured from the top (in ft), here we use "FHWA 0.05D" criterion, $w_t = 0.05 \times 4 = 0.2$ ft, then Θ_f (avg.) = $135.4 \times 0.2 = 27.1$.

9. Find n for each layer from Figure 5.4:

   Layer 1: $qu/\sigma p = 556/14.7 = 37.8$, $Em/\sigma n = 46400/7.37 = 6{,}296$, n=0.

   Layer 2: $qu/\sigma p = 1{,}111/14.7 = 75.6$, $E_m/\sigma_n = 116{,}000/9.74 = 11{,}910$, n=0.

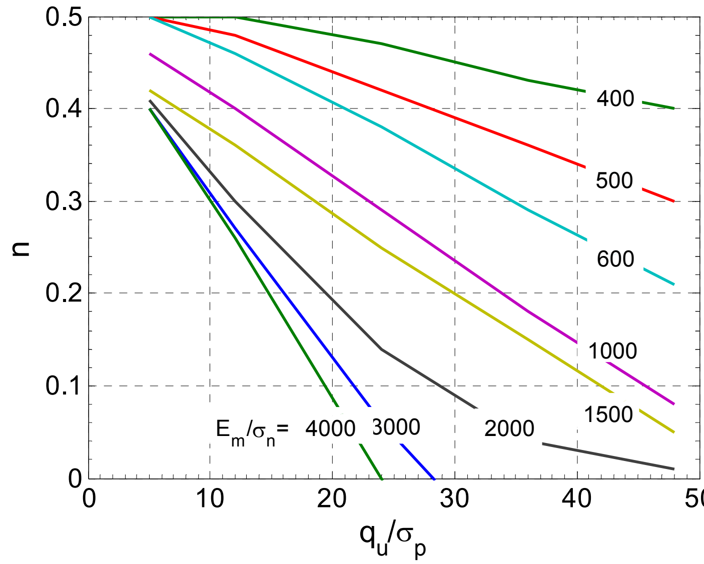


Figure 5.4: Factor n for smooth socket (O'Neill and Reese, 1999)

10. n (weighed avg.) = 0 < Θ_f.
11. Then $K_f = n + [(\Theta_f - n)(1 - n)]/(\Theta_f - 2n + 1) = 0 + [(27.1 - 0) \times (1 - 0)]/(27.1 - 0 + 1) = 0.96$
12. Unit side resistance $f = K_f f_{aa}$ (weighed avg.)= 0.96 × 32.9 = 31.6 psi = 2.28 tsf
13. Calculate unit base resistance.

$$q_b = 0.0134 E_{m(base)} \frac{\frac{L}{D}}{\left(\frac{L}{D}+1\right)} \left\{ \frac{200\left[\left(\frac{L}{D}\right)^{0.5} - \Omega\right]\left[1+\frac{L}{D}\right]}{\pi L \Gamma} \right\}^{0.67} w_t^{\,0.67}$$

= 0.0134 × 116,000 × [3.5/(3.5 + 1)][200 × ($3.5^{0.5}$ – 1.62) × (1 + 3.5)/3.14/14/0.59]$^{0.67}$ × [0.05 × 4]$^{0.67}$ = 1,753 = 126.2 tsf.

14. Calculate nominal side base and total resistance:

$Q_s = \pi LDf$ = 3.14 × 14ft × 4 × 2.28 = 401 ton

$Q_b = [\pi D^2/4]q_b = [3.14 \times (4)^2/4](126.2)$ = 1585 ton

15. Calculate factored resistance using the resistance factors of 0.70 for side and 0.25 for base resistance. $0.70Q_s + 0.25Q_b = 0.70 \times 401 + 0.25 \times 1{,}585 = 677$ ton > factored load (equals to 675 ton). The trial dimension of the drilled shaft is OK.

5.3.2 Shaft V5.0

Using the design software Shaft V5.0, it is much more convenient to perform the iteration design.

1. In the Strength Limit State design, the factored load is 1.25DL + 1.75LL =1.25 x 400 + 1.75 x 100 = 675 ton.
2. Use the "Data" menu to input parameters into the program (Figures 5.5 to 5.7). A preferred diameter (4 ft) has to be input at this time. The program will determine the minimum shaft length later.

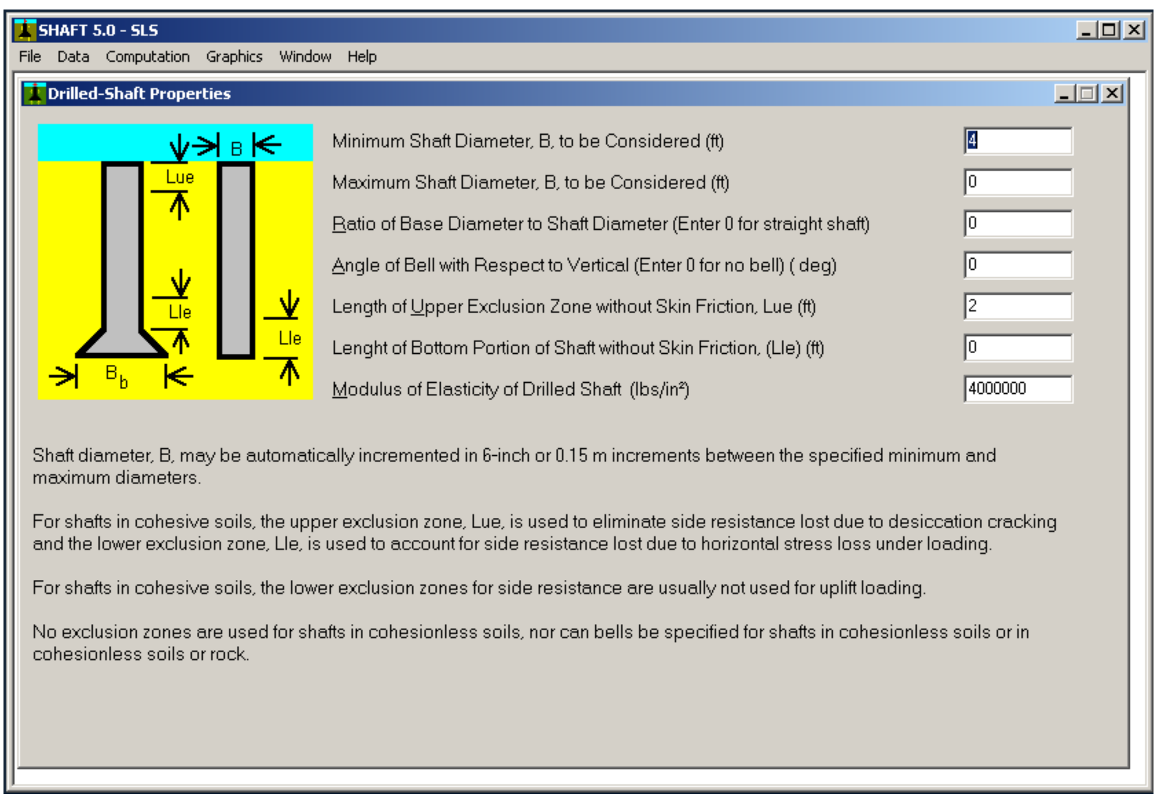


Figure 5.5: Input shaft properties

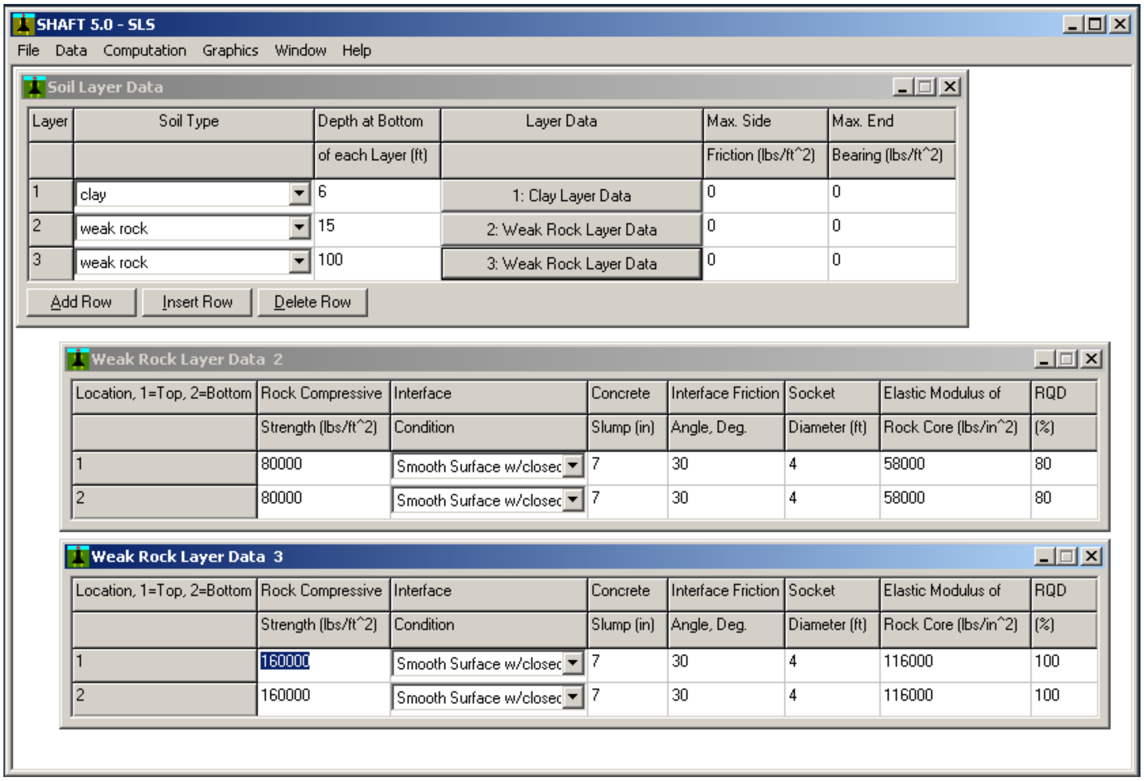


Figure 5.6: Input ground properties

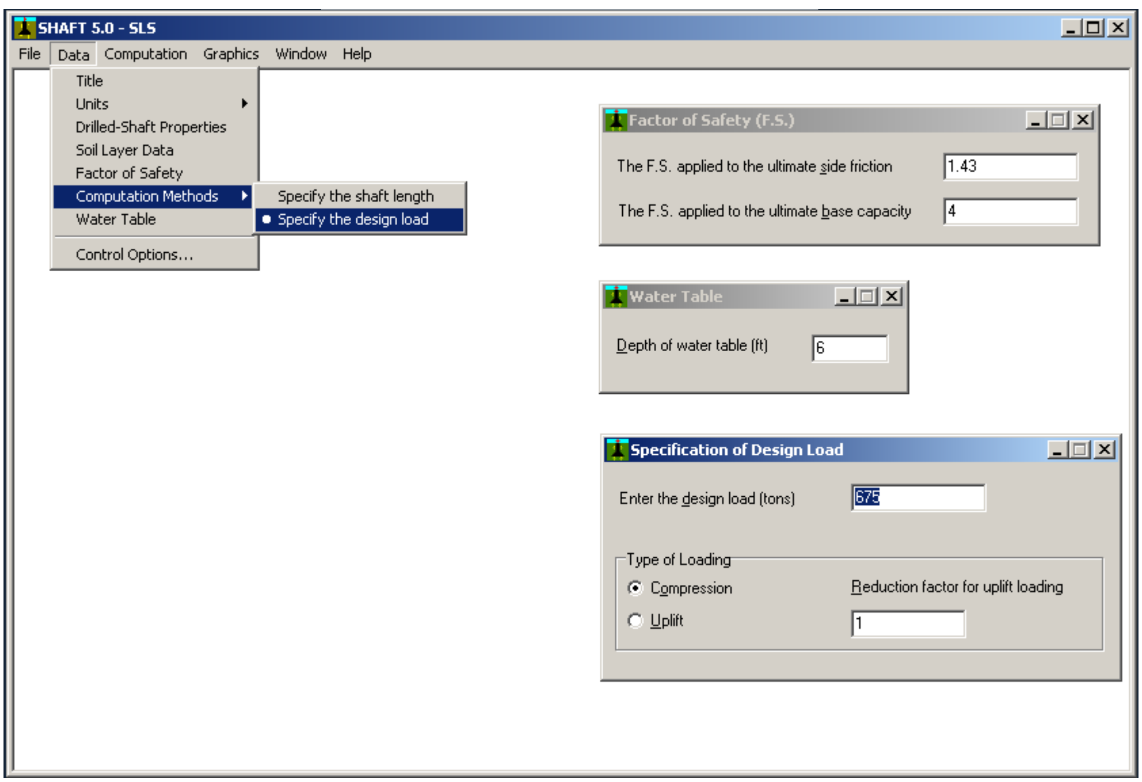


Figure 5.7: Input factors of safety, design load, and depth of water table

3. Shaft V5.0 is designed to use allowable stress design (ASD). To perform a LRFD, reciprocals of the resistance factors have to be calculated and input into the Factor of Safety window (Figure 5.7). In this case, a factor 1/0.7 = 1.43 is input for side friction and 1/0.25 = 4 for base capacity (0.7 and 0.25 are the resistance factors for side and base resistance, respectively). The factored load (675 ton) is input as the design load.
4. Goto “Computation” → “Run Analysis” to run the calculation. The program will increase the shaft length by steps until the factored resistance exceeds the design load.
5. Goto “Computation” → “Edit Output Text” to check the inputs and the results (Figure 5.8). In the “predicted results”, ultimate side, base and total resistance vs. shaft length are listed. Column “QDN” is the factored total resistance. The last shaft length the program calculated is the minimum shaft length that can carry the design load. In this case the minimum shaft length calculated by Shaft V5.0 is 19 ft (Figure 5.8).

```
SLS.sfo - Notepad
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PREDICTED RESULTS
-----------------

QS      = ULTIMATE SIDE RESISTANCE;
QB      = ULTIMATE BASE RESISTANCE;
WT      = WEIGHT OF DRILLED SHAFT (FOR UPLIFT CAPACITY ONLY);
QU      = TOTAL ULTIMATE RESISTANCE;
QBD     = TOTAL ALLOWABLE LOAD USING A FACTOR OF SAFETY
          APPLIED TO THE ULTIMATE BASE RESISTANCE;
QDN     = TOTAL ALLOWABLE LOAD USING FACTORS OF SAFETY
          APPLIED TO THE ULTIMATE SIDE RESISTANCE AND
          THE ULTIMATE BASE RESISTANCE.

LENGTH  VOLUME     QS       QB       QU       QBD      QDN      QU/VOLUME
(FEET)  (CU.YDS) (TONS)   (TONS)   (TONS)   (TONS)   (TONS)   (TONS/CU.YDS)
  3.0    1.40      0.00   586.68   586.68   146.67   146.67    420.12
  4.0    1.86      0.00   655.70   655.70   163.92   163.92    352.16
  5.0    2.33      0.00   690.21   690.21   172.55   172.55    296.56
  6.0    2.79      0.00   690.21   690.21   172.55   172.55    247.13
  7.0    3.26     33.48   690.21   723.69   206.03   195.97    222.10
  8.0    3.72     66.96   729.03   796.00   249.22   229.08    213.76
  9.0    4.19    100.44   782.76   883.20   296.13   265.93    210.82
 10.0    4.65    133.92   850.80   984.72   346.62   306.35    211.55
 11.0    5.12    167.40   932.78  1100.18   400.60   350.26    214.87
 12.0    5.59    200.89  1011.20  1212.09   453.69   393.28    217.00
 13.0    6.05    234.37  1081.22  1315.59   504.67   434.20    217.41
 14.0    6.52    267.85  1138.84  1406.68   552.56   472.01    215.86
 15.0    6.98    301.33  1180.70  1482.03   596.50   505.89    212.26
 16.0    7.45    367.69  1216.91  1584.60   671.91   561.35    212.76
 17.0    7.91    434.05  1248.55  1682.60   746.18   615.67    212.63
 18.0    8.38    500.41  1276.50  1776.90   819.53   669.06    212.07
 19.0    8.84    566.76  1300.12  1866.88   891.79   721.37    211.09
```

Figure 5.8: Calculation output for Strength Limit State design

5.4 Design Example – Service Limit State Design

Similar to the design procedure for the Strength Limit State design, a design procedure for the Service Limit State design is illustrated in Figure 5.9. A hand calculation for the same design example based on the settlement limit of 0.25 inch is provided below.

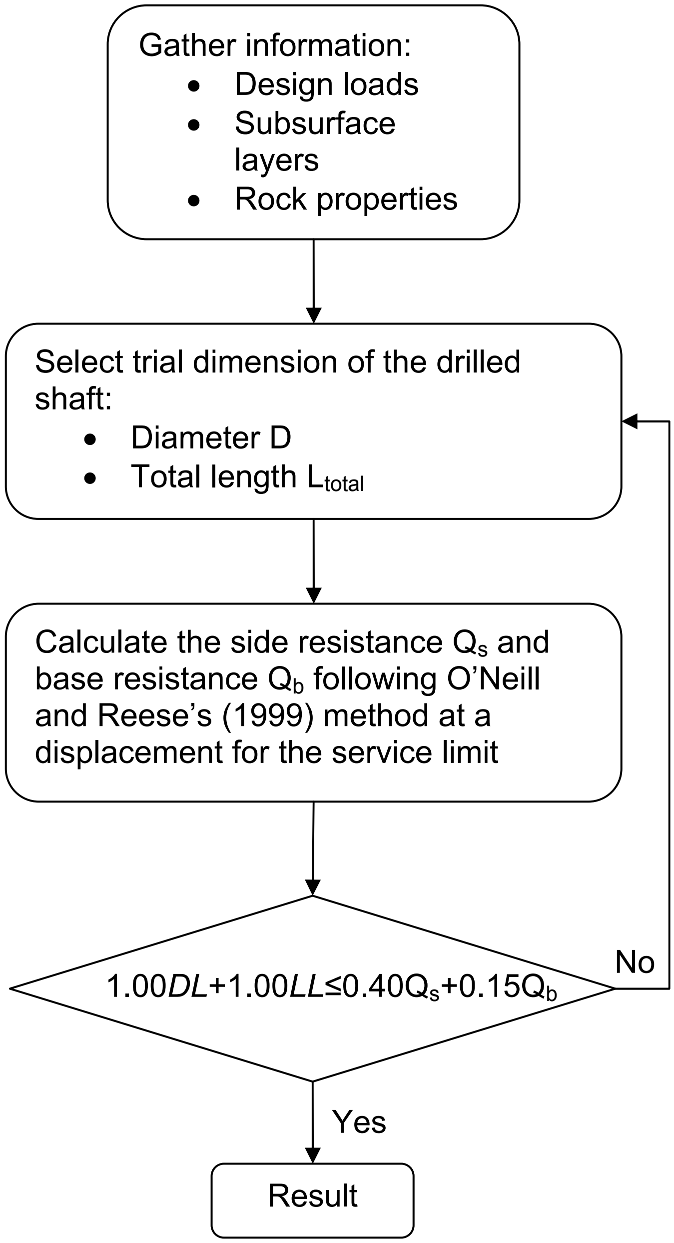

Figure 5.9: Procedure of strength limit state design

5.4.1 Hand Calculation

1. In a Strength Limit State design, the factored load is 1.00DL + 1.00LL =1.00 × 400 + 1.00 x 100 = 500 ton.

2. A trial dimension has to be assumed: shaft diameter D = 6 ft and shaft length L_{total} = 35 ft for this example. The shaft length in rock L = 35 – 6 =29 ft by ignoring the overburden soil.

3. Calculate the normal stress between the concrete and the borehole wall σ_n for each layer

   Layer 1: z_c = 10.5ft (3.2m) (depth from the top of the concrete to the mid-depth of this layer). From Figure 5.3, M = 0.98. The normal stress between the concrete and the borehole wall $\sigma_n = M\gamma'_c z_c = 0.98 \times 130 \times 6 + 0.98 \times (130 - 62.4) \times 4.5$ = 1,062 psf = 7.37 psi.

   Layer 2: z_c = 25ft (7.6m) (depth from the top of the concrete to the mid-depth of this layer). From Figure 5.3, M = 0.77. The normal stress between the concrete and the borehole wall $\sigma_n = M\gamma'_c z_c = 0.77 \times 130 \times 6 + 0.77 \times (130 - 62.4) \times 19$ = 1,590 psf = 11.04 psi.

4. Calculate α

   Layer 1: $\lambda = (15 - \sigma_n/\sigma_p)/27 = (15 - 7.37/14.7)/27 = 0.54$. $\alpha = (5 - 8.8\lambda)(q_u/\sigma_p)^{\lambda-1} = (5 - 8.8 \times 0.54) \times (556/14.7)^{0.54-1} = 0.047$.

   Layer 2: $\lambda = (15 - \sigma_n/\sigma_p)/27 = (15 - 11.04/14.7)/27 = 0.53$. $\alpha = (5 - 8.8\lambda)(q_u/\sigma_p)^{\lambda-1} = (5 - 8.8 \times 0.53)(1{,}111/14.7)^{0.53-1} = 0.044$.

5. Calculate E_m, f_a and f_{aa}

   Layer 1: $f_a = \alpha q_u = 0.047 \times 80$ = 3.76 ksf = 26.1 psi. Since RQD = 80%, from Tables 5.1 and 5.2, assuming close joints, f_{aa} = 0.92(26.1psi) = 24.0 psi, $E_m = 0.8E_i = 0.8 \times 58$ = 46.4 ksi.

Layer 2: $f_a = \alpha q_u = 0.044 \times 160 = 7.04$ ksf = 48.9 psi. Since RQD = 100%, from Tables 5.1 and 5.2, assuming close joints, $f_{aa} = f_a = 48.9$ psi, $E_m = E_i = 116$ ksi.

6. f_{aa} (weighed avg.) = $(24.0 \times 9 + 48.9 \times 20)/29 = 41.2$ psi.

   E_m (weighed avg.) = $(46.4 \times 9 + 116 \times 20)/29 = 94.4$ ksi.

7. $\Omega = 1.14(L/D)^{0.5} - 0.05[(L/D)^{0.5} - 1]\log_{10}(E_c/E_m) - 0.44 = 1.14 \times (29/6)^{0.5} - 0.05 \times [(29/6)^{0.5} - 1]\log_{10}(4000/94.4) - 0.44 = 1.97$.

   $\Gamma = 0.37(L/D)^{0.5} - 0.15[(L/D)^{0.5} - 1]\log_{10}(E_c/E_m) + 0.13 = 0.37 \times (29/6)^{0.5} - 0.15 \times [(29/6)^{0.5} - 1]\log_{10}(4000/94.4) + 0.13 = 0.65$.

8. $\Theta_f = E_m\Omega w_t/(\pi L\Gamma f_{aa}) = 94{,}400 \times 1.97/(3.14 \times 29 \times 0.65 \times 41.2) \times w_t = 76.3 \times w_t$, w_t is the displacement of the shaft measured from the top (in ft), here we use $w_t = 0.25$ in = 0.021 ft, then Θ_f (avg.)= $76.3 \times 0.021 = 1.60$.

9. Find n for each layer from Figure 5.4

   Layer 1: $q_u/\sigma_p = 556/14.7 = 37.8$, $Em/\sigma_n = 46{,}400/7.37 = 6{,}296$. n=0.

   Layer 2: $q_u/\sigma_p = 1{,}111/14.7 = 75.6$, $Em/\sigma_n = 116{,}000/11.04 = 10{,}507$. n=0.

10. n (weighed avg.) = $0 < \Theta_f$.

11. Then $K_f = n + [(\Theta_f - n)(1-n)]/(\Theta_f - 2n+1) = 0 + [(1.97 - 0) \times (1 - 0)]/(1.97 - 0 + 1) = 0.66$.

12. Unit side resistance $f = K_f f_{aa}$ (weighed avg.)= $0.66 \times 41.2 = 27.2$ psi = 1.96 tsf.

13. Calculate unit base resistance.

$$q_b = 0.0134 E_{m(base)} \frac{\frac{L}{D}}{\left(\frac{L}{D}+1\right)} \left\{ \frac{200\left[\left(\frac{L}{D}\right)^{0.5} - \Omega\right]\left[1+\frac{L}{D}\right]}{\pi L \Gamma} \right\}^{0.67} w_t^{0.67}$$

$= 0.0134 \times 116{,}000 \times [4.83/(4.83 + 1)][200 \times (4.83^{0.5} - 1.97) \times (1 + 4.83)/3.14/29/0.65]^{0.67} \times (0.021)^{0.67} = 266.3$ psi = 19.2 tsf.

14. Calculate nominal side base and total resistance

    $Q_s = \pi LDf = 3.14 \times 29 \times 6 \times 1.96 = 1{,}071$ ton

    $Q_b = [\pi D^2/4]q_b = [3.14 \times (6)^2/4] \times 19.2 = 543$ ton

15. Calculate the factored resistance using the resistance factor of 0.40 for side resistance and 0.15 for base resistance. $0.4Q_s + 0.15Q_b = 0.40 \times 1{,}071 + 0.15 \times 543 = 510$ ton > factored load (equals to 500 ton). The trial dimension of the drilled shaft is OK.

5.4.2 Shaft V5.0

Shaft V5.0 can calculate the ultimate side and base resistance of the drilled shaft (at "0.05D"), but there is no way to check the side and base resistance at the displacement of 0.25 inch. It is impossible to apply different resistance factors to the calculated side and base resistance. Therefore, the Service Limit State design can only be performed by hand calculation but not by Shaft V5.0 software.

5.5 Summary

Two design examples are provided in this chapter to illustrate load and resistance factor design of drilled shafts in weak rock based on the Strength Limit State design and the Service Limit State design. Resistance factors calibrated in the previous chapters are used. Design procedures using Shaft V5.0 are provided for the Strength Limit State design. Shaft V5.0 could not be used for the Service Limit State design because no output of side and base resistance at a specified displacement is available for this software. A spreadsheet can be developed to perform LRFD based on the Service Limit State.

CHAPTER 6 - CONCLUSIONS AND FINDINGS

6.1 Overview

LRFD is a simplified form of reliability based design. By multiplying calibrated factors to load and resistance components, the designed structure will be maintained at a specific level of reliability (or probability of failure). By concept, the load and resistance factors should be calibrated by a large number of test data; however, they are often unavailable in geotechnical engineering. Significant efforts are needed to calibrate load and resistance factors based on test data of good quality. In this study, O-Cell test data was collected from Kansas, Colorado, Missouri, Ohio, and Illinois and was analyzed and used to calibrate side and base resistance factors for drilled shafts in weak rocks.

6.2 Statistical Analyses on O-Cell Test Data

Twenty-six O-Cell test data were collected in this study for drilled shafts in weak rocks. Seven methods available in the literature were selected to estimate the load capacities of 25 out of 26 drilled shafts. Calculated load capacities from five methods (FHWA 0.05D, Davisson's, Brinch-Hansen's 80%, Butler and Hoy's, and Fuller and Hoy's methods) were used for statistical analyses. The comparison showed that Butler and Hoy's method is most reliable but the interpreted capacity by this method is to some extent overestimated. The "FHWA 0.05D" method was found to yield the closest and conservative predictions of the ultimate resistances to the representative values. Therefore, the resistance corresponding to a displacement of 5% shaft diameter is recommended as the ultimate resistance of drilled shafts. This method was adopted in this study when the resistance factors were calibrated for the Strength Limit State design.

6.3 Resistance Factors for Drilled Shafts in Weak Rock

Side and base resistance factors were calibrated based on the O-Cell test data. Strength I Limit State and Service I Limit State were considered. Resistance factors were calibrated at two different target reliability indices: 2.3 (P_f≈1/100) for group of five or more shafts and 3.0 (P_f≈1/1000) for shafts with less redundancy. Side resistance factors were calibrated from two different sources of measured resistance: total side resistance and layered unit side resistance. Since the layered unit side resistance had more datasets than the total side resistance, the resistance calibrated from the layered measured unit side resistance was considered more reliable, thus recommended for designers to use. The recommended resistance factors are listed in Table 6.1. Some calibrated resistance factors are considerably less than those in the AASHTO specifications. The main reason for the lower resistance factors in this study is the low efficiency of the FHWA design method, as indicated by the wide distributed biases especially for base resistance. Another possible reason is the limited size of the available test database. The result may be further improved by increasing the size and improving the quality of the database in the future. At present, as an alternative to the use of lower resistance factors, field load tests on the drilled shafts are also recommended. In that case, the AASHTO allows the use of a resistance factor of no more than 0.70.

Table 6.1 Recommended resistance factors from this study

Situation		φ (β_T=3.0)	φ (β_T=2.3)
Side resistance	Strength Limit State	0.70	1.00
	Service Limit State	0.40	0.65
Base resistance	Strength Limit State	0.25	0.45
	Service Limit State	0.15	0.35

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APPENDIX A - O-CELL TEST DATABASE

Test No	Location	Field Condition	Avg. q_u of rock (MPa)	Installation Method	Drilling/Clean Method	Grooving	Failure Type
1	Topeka, KS	Sandy shale, cross-bedded, sandstone lenses	14.5	Dry excavated, temporary surface casing	Auger/Hand cleaned	None	Cell
2	Scandia-1, KS	Graneros shale, some sand, limey seams finely laminated	0.9	Dry excavated, temporary surface casing	Auger/Cleanout bucket	None	Side
3	Scandia-2, KS	Graneros shale, numerous thin limey lenses	1.1	Dry excavated, temporary surface casing	Auger/Cleanout bucket	None	Side
4	Republican River, KS	Wellington shale, gypsum, anhydrite	2.7	Wet excavated, , 2-layer temporary casing, inner casing to 9.34m	Auger/Mud bucket	None	Tip
5	El Dorado, KS	Paiddock shale, thin beds of Gypsum	18.3	Dry excavated, 2-layer temporary casing, inner casing to 7.77m	Auger, rock auger/Cleanout bucket	None	Cell
6	Ellsworth, KS	Dakota sandstone, very poorly to moderately cemented, thin clayey shale beds	1.5	Wet excavated, 2-layer temporary casing, inner casing to 16.57m, both left in place during test	Auger, core barrel/Cleanout bucket	None	Tip
7	Osborne, KS	Fairport chalk member hard shale, ammonite and Ostrea fossils	2.7	Wet excavated, , temporary casing to 16.57m	Not specified/ Not specified	N/A	Side
8	Coffey Co., KS	Snyderville shale, soft, clayey	15.3	Wet excavated, , 2-layer temporary casing, outer to 10.52m and inner to 10.97m, both left in place during test	Auger/ Cleanout bucket	None	Tip
9	I225, CO	Sandy claystone, clayey sandstone	0.6	Dry excavated, roughed to some extent	Auger, roughening tooth/ Mud bucket	To some extent at depth 6.28-8.72m	Tip

Test No	Location	Field Condition	Avg. q_u of rock (MPa)	Installation Method	Drilling/Clean Method	Grooving	Failure Type
10	County Line, CO	Stiff Clay, claystone, very weak sandstone	0.4	Dry excavated, roughed to some extent	Auger, roughening tooth/ Mud bucket	To some extent at depth 6.28-8.72m	Tip
11	Franklin, CO	Sandy claystone, thin clayey sandstone on top	3.1	Wet excavated	Auger/ Mud bucket	None	Tip
12	Broadway, CO	Silty-clayey sandstone, slightly sandy siltstone	7.9	Dry excavated	Auger/ Mud bucket	None	Tip
13	Trinidad-1, CO	Pierre shale, with sand, gravel on top	16.7	Dry excavated, temporary surface casing	Auger/Cleanout bucket	None	Side
14	Trinidad-2, CO	Pierre shale w/ sandy clay, gravel on top	20.2	Dry excavated, temporary casing to 9.14m	Auger/Cleanout bucket	None	Tip
15	Lexington-1, MO	Micaceous silt shale, clay shale, limestone, fleming foundation	1.5	Wet excavated, permanent casing in overburden	Bullet tooth rock auger/Not specified	None	Cell
16	Lexington-2, MO	Micaceous silt shale, clay shale, limestone, fleming foundation	3.6	Wet excavated, permanent casing in overburden	Bullet tooth rock auger/Not specified	None	Tip
17	Grandview, MO	Weathered shale, some unweathered shale, limestone	16.9	Dry excavated, temporary surface casing	Bullet tooth rock auger, rock auger, core barrel/Not specified	None	Side
18	Waverly, MO	Clay shale, carbonaceous shale, coarse grained sandstone, fossiliferous limestone	1.0	Dry excavated, temporary surface casing to top of rock	Bullet tooth rock auger, core barrel/Not specified	None	Cell

Continued

Test No	Location	Field Condition	Avg. q_u of rock (MPa)	Installation Method	Drilling/Clean Method	Grooving	Failure Type
19*	Atchison, KS	Silty shale, hard shale, shale limestone	9.5	Pile was flooded after installation, and cleanout before test, 2-layer casing	Not specified/ Not specified	None	none
20	Dearborn, MO	Soft calcareous rock, gray thinly laminated silt shale	12.4	Steel casing	Not specified/ Not specified	None	Side
21*	Clarksville-1, MO	Hard shale covered by 24ft clay and organic fill	n/a	n/a	n/a	n/a	Cell
22*	Clarksville-2, MO	Hard shale covered by 27.5ft clay and organic fill	n/a	n/a	n/a	n/a	Cell
23*	Toledo-1, OH	Stiff to very stiff till, hardpan, dolomite	n/a	Wet method, temporary casing	Auger/ Cleanout bucket	None	Side
24*	Toledo-2, OH	Stiff to very stiff till, hardpan, dolomite	n/a	Wet method, temporary casing	Auger/ One-Eye cleanout bucket	None	Side
25*	St. Louis-1, IL	n/a	n/a	n/a	n/a	n/a	Base
26*	St. Louis-2, IL	n/a	n/a	n/a	n/a	n/a	Side

* exclude from calibration, mostly due to lack of enough information.

K - TRAN

KANSAS TRANSPORTATION RESEARCH
AND
NEW - DEVELOPMENTS PROGRAM

A COOPERATIVE TRANSPORTATION RESEARCH PROGRAM BETWEEN:

KANSAS DEPARTMENT OF TRANSPORTATION

THE UNIVERSITY OF KANSAS

KANSAS STATE UNIVERSITY