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Review and Refinement of the SDDOT's LRFD Deep Foundation Design Method

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16. Abstract
It is well-known by practicing engineers that uncertainty exists in design and that this uncertainty arises from numerous sources. Reliability-based design (RBD) methods address this by incorporating uncertainty in the design procedure and process. The FHWA mandated the use of the Load and Resistance Factored Design (LRFD) methodology for all federally funded transportation projects beginning in October 2007. The South Dakota Department of Transportation's (SDDOT) current deep foundation investigation procedures and design methodologies are unique and therefore require specific calibration for use in LRFD design. The first phase of this project reviewed the literature, conducted interviews with adjacent states, examined LRFD implementation, and reviewed the SDDOT's design method relative to FHWA requirements. The second phase of the project consisted of attempting to calibrate existing field data to generate resistance factors in assisting the SDDOT in transition to a full LRFD platform rather than using the calibration-to-fit method currently employed. Data was collected from available load tests within South Dakota which, when suitable, was the basis of the probabilistic analysis of the design methodology. This project identified the need for the SDDOT to collect field data to both verify the basis of their capacity predictions for deep foundation design as well as supplementing a sparse data set for LRFD resistance factor calibration.

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Acronym	Definition
AASHTO	American Association of State Highway and Transportation Officials
API	American Petroleum Institute
ASD	Allowable Stress Design
BOR	Beginning of Restrike
CAPWAP	CAse Pile Wave Analysis Program
COV	Coefficient of Variation
CPT	Cone Penetrometer Test
DL	Dead Load
DMT	Dilatometer Test
DOT	Department of Transportation
ENR	Engineering News Record
EOD	End of Driving
FHWA	Federal Highway Administration
FOS	Factor of Safety
GIS	Geographic Information System
GPR	Ground Penetrating Radar
GPS	Global Positioning System
IGM	Intermediate Geomaterials
LL	Live Load
LRFD	Load and Resistance Factor Design
MDT	Montana Department of Transportation
MENR	Michigan Engineering News Record (formula)
NCHRP	National Cooperative Highway Research Program
NDOR	Nebraska Department of Roads
PDA	Pile Driving Analyzer
PDF	Probability Density Function
RBD	Reliability Based Design
RTC	Reliability Theory Calibration
SBPMT	Self-Boring Pressuremeter Test
SDDOT	South Dakota Department of Transportation
SDENR	South Dakota Engineering News Record (formula)
SDSM&T	South Dakota School of Mines & Technology
SDSU	South Dakota State University
SFV	Skin Friction Value
SPT	Standard Penetration Test
UM	Ultimate Method
WEAP	Wave Equation Analysis of Piles
WSD	Working Stress Design

1 EXECUTIVE SUMMARY

It is well-known by practicing engineers that uncertainty exists in design and that this uncertainty arises from numerous sources. Typical sources of uncertainty are inherent variability, measurement errors, computation error, construction uncertainty and model error. In deep foundation engineering, nominal values for the pertinent soil properties and parameters are utilized in a prediction model to determine the ultimate capacity of the system. Design uncertainty has historically been managed using an allowable stress design (ASD) approach, where either a global factor of safety is applied to the calculated ultimate capacity of the foundation or nominal safety factors are assigned to the individual components that comprise the total design (i.e. soil friction angle or cohesion). Regardless of the method used, safety factors are typically assigned based on commonly accepted values or an individual design engineer's comfort level and without quantification of actual design uncertainty magnitudes. This often leads to designs that are extremely conservative, and thus very inefficient, or designs that are unsafe.

A reliability-based design (RBD) addresses these issues by accounting for uncertainty in the design procedure and ultimately in the design process for deep foundations. In an RBD procedure, the magnitude of design uncertainty can be rationally incorporated within the overall design process, thereby resulting in a safe, efficient, and consistent design approach. An RBD procedure based on the Load and Resistance Factor Design (LRFD) approach is becoming increasingly utilized in the design of geotechnical structures, such as deep foundations (AASHTO 2009). In fact, the Federal Highway Administration (FHWA) mandates the use of the LRFD methodology for all federally funded transportation infrastructure projects starting in October 2007. In the LRFD method, resistance factors are applied to the calculated ultimate resistance of the foundation, while load factors are applied to the calculated loads on the foundation. Load and resistance factors are determined using a probabilistic approach to ensure that design uncertainties are explicitly integrated and quantified in the design. In addition, resistance factors can be calibrated to produce designs that consistently achieve a desired level of reliability. The application of load and resistance factors is thus intended to replace the global factor of safety in the design process at both the strength and service limit states.

This research project examined the SDDOT's current design methodologies relative to deep foundation support of bridges and gathered data in an attempt to generate state specific resistance factors for LRFD deep foundation design. Since the SDDOT uses a unique method in determining the strength limit state of axial pile capacity, current resistance factors reported in the AASHTO specifications cannot be used in the design of pile foundations. The first phase of this research examined documentation available in the literature, as well as documentation provided by the SDDOT to determine the basis for current design methods relative to LRFD implementation. That documentation included the AASHTO Bridge Design Specifications, FHWA published design guidance, past research projects initiated by the SDDOT, and internal design guidance documentation. As part of this discovery phase, the researchers also contacted surrounding states to discern the geotechnical process by which they have implemented LRFD requirements. A standardized list of questions was generated and the information was gathered via telephone interviews.

The second phase of this project was designed around assisting the SDDOT in transitioning to a full LRFD design platform. The SDDOT currently uses a "calibration-to-fit" method of determining resistance factors for LRFD design. The disadvantage to this method is that it does not use state specific data in determining resistance factors that account for regional geotechnical conditions. Furthermore, the calibration-to-fit method is essentially the ASD method of determining pile capacity (i.e. strength limit state). As part of this research, the research team collected field test data from the SDDOT, FHWA, regional and local geotechnical consulting firms, and specialty contractors to construct a database for the purposes of determining regional resistance factors for LRFD design. Field test data consisted of full scale load test data, dynamic field test data, driving records, and site explorations.

The database was examined to assess the quality and character of the data relative to the ability to account for uncertainly in the variables that are used in deep foundation design. Once data was deemed suitable for inclusion in the database, a probabilistic analysis of the data was performed to calibrate resistance factors specific to South Dakota. Although a number of static load tests and dynamic capacity analyses for different types of deep foundation systems were collected, only a limited number of these load test data could be utilized in the resistance factor calibration process. For this project, resistance factors were calibrated for several different types of driven piles; however, a sufficient quantity of data was not available for a robust probabilistic analysis. In addition, the quality of the data could be improved by supplementing the database with new data that reflects current construction methodologies. Therefore, recommendations are made regarding advancing a load test and pile driveability database for calibration of resistance factors in the future.

The results of this study can be summarized in the following main conclusions:

1. The SDDOT uses unique field data collection and design methods in predicting the axial compressive capacity of bridge pile and drilled shaft foundations. Based on past research, the method has promise, but sufficient data does not exist to quantify its efficiency and reliability.
2. Documentation does not exist that quantifies the built-in factor of safety of the current design method. This could lead to cost inefficiencies due to overdesign or potential losses in the case where the predicted resistance is greater than the actual resistance.
3. The SDDOT's current method of incorporating LRFD into bridge design is currently adequate based on FHWA's communication. However, FHWA commented that the SDDOT should continue to develop the LRFD platform by continuing the process of determining local resistance factors.
4. Resistance factors were calibrated for the SDDOT drive test predictive resistance design method for several different types of driven pile systems. However, the quantity of the load test data used in those calibrations is relatively small and the quality of the data should ideally reflect current construction methods and conditions.

Based on the results of this study, the following recommendations are made:

1. A research project should be instituted with the specific purpose of verifying and improving the efficiency of the SDDOT's current drive test predictive resistance design method. This research project could not find documentation that completely supports the method or documents the basis of the process or parameters. This recommendation would require that extensive collection of load test data be instituted on full-scale driven piles and drilled shafts installed in the field. The data collection would include multiple test sites and would include static load testing and high strain dynamic pile testing using a Pile Driving Analyzer (PDA) and the CAse Pile Wave Analysis Program (CAPWAP). The static load testing would serve as a means to verify the results of the PDA and CAPWAP analyses, which would potentially lead to the widespread adoption of the PDA and CAPWAP technologies in future field verification and load testing programs. The load testing should incorporate instrumentation, such as strain gages or telltales, which can provide a distinct separation of the side and toe resistance components of driven piles and drilled shafts. The separation of the resistance components in the load testing may allow the SDDOT to refine the drive test predictive resistance method with respect to side and toe resistance since the design method includes both a drive component and pullout component. The load test data may also result in the adoption of alternative design methods, such as those reported in the AASHTO

LRFD Bridge Design Specifications or a soil-structure interaction method, such as the t-z model method, in order to further increase design efficiency and reliability.

2. The SDDOT should perform recalibration of the resistance factors that were developed for this project. Performing additional field load tests on driven piles and drilled shafts will allow for further development of the load test database initiated by this project and subsequent recalibration of geotechnical resistance factors for use in design. For resistance factor calibration, 15 to 20 field load tests should ideally be included in the database for each driven pile type and for drilled shafts in order to ensure adequate statistical representation of the SDDOT's drive test predictive resistance design approach. Since an initial load test database was developed for this project in order to calibrate resistance factors for driven piles, the SDDOT can easily add the additional load test results to the database and perform the recalibration exercise. As new load test data becomes available, it is recommended that the new data be assessed with respect to the old data and that the old data be replaced since the new load test data will reflect current construction methods and technologies.
3. The SDDOT should address other limit states with respect to LRFD design, especially the serviceability limit state. Design methodologies for the serviceability limit state can be adopted through the AASHTO LRFD Bridge Specifications. The serviceability limit state can be critical to the performance of bridge structures, especially with respect to maintenance and functionality. In addition, the AASHTO LRFD Bridge Design Specifications require the verification of all deep foundation designs at the serviceability limit state.
4. The SDDOT should formally address lateral pile performance in their designs per the design methodologies provided in the AASHTO LRFD Bridge Specifications or through the adoption of the LPILE software, which is widely utilized in the industry.
5. The SDDOT should use the rated steel strength for driven piles in their designs.
6. The SDDOT should formally address group effects in their designs per the design methodologies provided in the AASHTO LRFD Bridge Specifications.
7. The SDDOT should review their method for field sampling, especially when undisturbed soil samples are desired for laboratory testing such as for the consolidation test, unconfined compressive strength test, direct shear test, and triaxial shear test.
8. The SDDOT should update their driving formula for field verification of pile capacity. Current and past research has shown that better field verification formula exist that are more reliable than what is currently used by the SDDOT.
9. The SDDOT should perform driveability analysis using the GRLWEAP software as part of their designs in the future.

2 PROBLEM DESCRIPTION

It is well-known by practicing engineers that uncertainty exists in design and that this uncertainty arises from numerous sources. Uncertainty in geotechnical engineering can be formally grouped into aleatory and epistemic uncertainty (Lacasse et al. 1996). Typical sources of uncertainty are inherent variability, measurement errors, computation error, construction uncertainty and model error (Phoon et al. 1995). In deep foundation engineering, nominal values for the pertinent soil properties and parameters are utilized in a prediction model to determine the ultimate capacity of the system. Design uncertainty was historically managed using an allowable stress design (ASD) approach, where either a global factor of safety is applied to the calculated ultimate capacity of the foundation or nominal safety factors are assigned to the individual components that comprise the total design (i.e. soil friction angle or cohesion) (Phoon et al. 1995). Regardless of the method used, safety factors are typically assigned based on commonly accepted values or an individual design engineer's comfort level without quantification of actual design uncertainty magnitudes (Kulhawy and Phoon 2006). This often leads to designs that are extremely conservative, and thus very inefficient, or designs that are unsafe.

A reliability-based design (RBD) addresses these issues by accounting for uncertainty in the design procedure and ultimately in the entire design process for deep foundations. In an RBD procedure, the magnitude of design uncertainty can be rationally incorporated within the overall design process, thereby resulting in a safe, efficient, and consistent design approach. An RBD procedure based on the Load and Resistance Factor Design (LRFD) approach is becoming increasingly utilized in the design of geotechnical structures, such as deep foundations (AASHTO 2009). In fact, the Federal Highway Administration (FHWA) mandated the use of the LRFD methodology for all federally funded transportation infrastructure projects starting in October 2007. In the LRFD method, resistance factors are applied to the calculated ultimate resistance of the foundation, while load factors are applied to the calculated loads on the foundation. Load and resistance factors are determined using a probabilistic approach to ensure that design uncertainties are explicitly integrated and quantified in the design. In addition, resistance factors can be calibrated to produce designs that consistently achieve a desired level of reliability (Phoon et al, 1995). The application of load and resistance factors is thus intended to replace the global factor of safety in the design process at both the strength and service limit states.

In general, load factors have been extensively developed for structural design of bridges and other structures (Nowak 1995). Therefore, it is important to focus on the calibration of the resistance factor for the design of deep foundations. Currently, numerous approaches exist for resistance factor calibration, including the "calibration-to-fit" method and the reliability theory calibration (RTC) method (Withiam et al. 1998). Since the calibration-to-fit method does not involve explicit quantification of design uncertainty, the FHWA has focused on the use of the RTC technique for calibration of resistance factors. The basis of the RTC method is the examination and comparison of uncertainty in the foundation loads to the uncertainty in the foundation resistance while ensuring that a specific level of reliability is achieved in the overall design. This is accomplished by computing statistical parameters that define the bias and uncertainty within the static capacity prediction model used in the design, along with the magnitude of uncertainty from the previously identified sources. The calibration of resistance factors is necessary to satisfy a desired level of reliability. Since there are numerous combinations of deep foundation systems, geological conditions, and static capacity prediction models, the FHWA has permitted state DOTs to calibrate resistance factors based on conditions that are specific to that state (i.e. "local" resistance factors). To that end, this research project was focused towards gathering information that could lead to the calibration of resistance factors that are specific to the foundation systems and design methodologies utilized by the South Dakota Department of Transportation (SDDOT).

3 OBJECTIVES AND RESEARCH PLAN

3.1 OBJECTIVES

The study presented in this report was focused towards addressing the following three main research objectives:

1. *Assess the current status of FHWA sanctioned LRFD Specifications as they pertain to SDDOT foundation design procedures.*

This objective was accomplished by reviewing the FHWA requirements for implementation of the LRFD Specifications for deep foundation design. A comprehensive study of the SDDOT deep foundation design procedures was also completed to determine how the FHWA requirements can be instituted most effectively and efficiently within the LRFD framework at the SDDOT.

2. *Recommend refinements to SDDOT's current foundation design procedures consistent with LRFD Specifications.*

This objective was accomplished based on the results of Objective 1 and involved providing recommendations for refining SDDOT's current deep foundation design procedures to ensure compliance and consistency with the FHWA requirements for the utilization of the LRFD Specifications. The recommendations were based both on 1) a review of current SDDOT deep foundation design methods 2) analysis of resistance factors that have been calibrated specifically to South Dakota deep foundation systems and 3) conditions and resistance factors that are from an acceptable LRFD code, such as the AASHTO LRFD Bridge Specifications.

3. *Assist SDDOT with expanded use of LRFD Specifications in an effort to optimize reliability, consistency, and efficiency.*

This objective was accomplished by initial development of a load test database and the initial calibration of resistance factors for application in the design of driven piles using the methodologies that are specific to South Dakota. Since the quantity and quality of load test data that could be utilized in the resistance factor calibration process was limited, this objective was further accomplished by developing and outlining an implementation plan that focuses on an LRFD adoption process that will satisfy the published requirements for reliability while optimizing consistency and efficiency within the deep foundation design procedures.

3.2 RESEARCH PLAN

This research involved the study of current SDDOT methodologies for deep foundation design from a strength limit state standpoint with the goal of gathering information and data that would assist the SDDOT in transitioning to load factor calibration for implementing LRFD deep foundation design of bridges. This process was developed by identifying 14 research tasks to address the research objectives. This research is a collaborative effort between South Dakota State University (SDSU) and the South Dakota School of Mines & Technology (SDSM&T). The following section outlines each of the tasks that were performed for this research project.

Task 1: Meet with the project's technical panel to review the project scope and work plan.

This meeting set the framework for the project and occurred on December 16, 2008. At this meeting, the research team outlined the work plan for the first 6 to 12 months of the project and discussed assistance required from the SDDOT. Discussions revolved around documents that were needed from the SDDOT, the survey that was to be conducted in the neighboring states, and general LRFD discussions. Meeting minutes were provided in the quarterly report.

Task 2: Conduct interviews with key SDDOT stakeholders to review SDDOT deep foundation design methodology including: field testing procedures, lab testing procedures, sampling procedures, selection of pile materials, and pile driving formulas.

This task consisted of contacting key SDDOT stakeholders in the foundations section in an effort to systematically understand how deep foundation design is conducted at SDDOT. The stakeholders included the foundation section, bridge section, and research personnel. The effort in understanding the methodology(ies) included applicable field testing approaches and procedures, subsurface explorations procedures, soil and sample collection, laboratory testing approaches and methods, design methodologies, materials selection, and applicable aspects of field implementation (capacity verification). This task was assessed based on a series of documents provided by the SDDOT foundation section. The information provided is outlined in detail in the report. The information provided was evaluated relative to the Research Objectives outlined in Section 2.1. This task effort is outlined in Chapter 5 of this report and was conducted primarily by SDSU. In an effort to lay the framework, background information for deep foundation design is presented in Chapter 4 of this report.

Task 3: Provide detailed information on FHWA requirements for LRFD resistance values.

This task consisted of examining current FHWA LRFD requirements for bridge design and summarizing the requirements relative to current SDDOT procedures and practices discovered in Task 2, as well as the objectives of this research project presented in Section 3.1. This task was based on SDDOT communications with FHWA that pertained to current SDDOT LRFD practices. This task considered recommendations by FHWA relative to AASHTO Specifications and resistance factor calibration outlined in NCHRP (Paikowsky 2004) as it pertained to deep foundation design of driven H-pile sections, precast concrete, and timber piles. During the research study, it was discovered that the SDDOT rarely uses large diameter monolithic drilled shafts; therefore, little to no information on drilled shafts was available for evaluation. This task effort is outlined in Chapter 5 of this report and was conducted primarily by SDSU.

Task 4: Review and evaluate approaches used by neighboring states (MN, ND, WY, NE, MT, & IA) related to LRFD implementation and identify opportunities where their application may benefit SDDOT methodology.

The research team contacted the lead geotechnical personnel at DOTs in six surrounding states (Minnesota, North Dakota, Wyoming, Nebraska, Montana, and Iowa) to discern the geotechnical process by which they have (or will have) implemented LRFD requirements. A standardized list of questions was generated and information gathered via telephone interviews. Opportunities where methods utilized in other states could provide efficiencies to the SDDOT were highlighted. Jerry DiMaggio with the National Academy of Sciences (formerly with FHWA) and Samuel Paikowsky with the University of Massachusetts - Lowell were also interviewed. A summary of this survey is included in Chapter 6 and was primarily conducted by SDSU.

Task 5: Review SDDOT's current methods for determining resistance factors used in deep foundation design and assess their application to calibration of LRFD resistance factors.

The research team evaluated the information gathered in Tasks 2 through 4 and assessed their applicability to both LRFD design requirements and their application to determining resistance factors and calibration as appropriate. Based on this assessment, the research team provides recommendations in this report on proper determination of resistance factors and appropriate methods of calibration. This task does not include calibration efforts as that is provided under research Task 9. This task was primarily performed by SDSU and is included in Chapters 7 and 8.

Task 6: Meet with the Technical Panel to review and approve interim results and to confirm project direction.

The research team met with the Technical Panel after the completion of Tasks 1 through 5 to approve the results of the project and confirm the project direction. This meeting occurred on July 14, 2009 where the team summarized the results of the study to date and outlined the work plan for the remaining scope of work for the project. The research team confirmed that the research was on schedule and that significant progress has been made. The Technical Panel confirmed that the research should continue as planned at that time. The Technical Panel meeting minutes were provided in a quarterly report by the research team.

Task 7: Assemble a database of applicable field data, including load test or pile driveability analysis data, available from SDDOT, local contractors, consultants, and neighboring state transportation departments.

The key component of the resistance factor calibration process is the development of a load test database for various deep foundation systems. For this project, all pertinent load test data that was available from the SDDOT, specialty geotechnical contractors, geotechnical consultants, and neighboring state DOTs (assuming the deep foundation systems and geologic stratigraphy are similar to South Dakota) was assembled and compiled in tabular format. The load test database consists of load tests performed on various types of driven piles (H-piles, timber piles, pipe piles, and precast concrete piles) and drilled shafts. The load test data for the driven piles consists of both static load tests and tests conducted using a Pile Driving Analyzer (PDA) with CAPWAP. The load test data for the drilled shafts consists of both conventional static load tests and O-Cell tests. Since the nominal resistance of a deep foundation consists of both side friction and an end bearing component, load test data that distinctly separates these two components is highly beneficial in resistance factor calibration, although not entirely necessary. This task was performed primarily by SDSM&T and is detailed in Chapter 7.

Task 8: Analyze available load test data for resistance values and factors and compare to current SDDOT load capacity prediction methods.

Once all available load test data was collected and reviewed, the data was analyzed to determine the *actual* resistance of the deep foundation system. For most of the static load tests conducted on the driven piles and the drilled shafts, the determination of the actual resistance was accomplished using the Davisson's method, along with examination of the non-linear behavior of the load-settlement curve from the test. These resistance values were also provided in the documentation or report that accompanied the load test data. For the dynamic analyses conducted on the driven piles, the actual resistance was computed using signal matching with the CAPWAP software. For the O-Cell tests conducted on the drilled shafts, the actual resistance was provided in the accompanying test report and was generally determined from movements of the drilled shaft during testing. The SDDOT drive test predictive resistance formula approach was used to compute the *predicted* resistance for as many load tests as possible. However, for a large number of load tests, it was not possible to utilize the SDDOT drive test approach as explained in detail in Chapter 8. For some of these load tests, a resistance value computed using an alternative method has been provided in the database. The statistics of the ratio of the actual resistance to the predicted resistance were computed for the SDDOT drive test predictive resistance formula for different types of driven piles. This task was performed primarily by SDSM&T and is detailed in Chapter 8.

Task 9: Attempt probabilistic analysis of available project field data to calibrate resistance factors specific to South Dakota. This task will depend upon the amount and quality of the data obtained in Task 7 and Task 8. If the amount or quality of the data is deemed insufficient for a probabilistic analysis, recommendations will be made regarding advancing the load test and pile driveability database for calibration of resistance factors in the future.

The calibration of geotechnical resistance factors for the SDDOT drive test predictive resistance formula method was completed using the developed load test database. Resistance factors were developed for different driven pile types, including H-piles, timber piles, steel pipe piles, and precast concrete piles. In addition to resistance factors, efficiency factors and an equivalent factor of safety were computed, which provide additional information regarding the design of these pile systems using the SDDOT design methodology. However, the quantity of the data utilized in the calibration effort was limited. In addition, no usable load test data for drilled shafts was available and thus resistance factors for drilled shafts were not calibrated. Therefore, recommendations have been provided regarding how the load test database should be advanced by the SDDOT in order to recalibrate resistance factors for use in geotechnical design. This task was performed primarily by SDSM&T and is detailed in Chapter 8.

Task 10: Make recommendations for a specific approach for instituting LRFD and acquisition of data in support of the selected approach.

As discussed in Task 9, an initial calibration of resistance factors was conducted for driven piles, but could not be accomplished for drilled shafts. However, since the calibration was based on a limited quantity of load test data, the research team has provided recommendations for furthering the development of the load test database that will lead to recalibration of resistance factors for design of both driven pile and drilled shaft systems. Additional recommendations regarding potential revisions to current investigative and design methodologies have also been provided to fully integrate several additional design requirements within the LRFD approach per FHWA requirements. This task was performed primarily by SDSM&T with detail in Chapter 8 and summarized in Chapter 9.

Task 11: Meet with the technical panel to review and accept results of the investigation and proposed recommendations. Identify an appropriate project with sufficient parallel analysis to demonstrate application of LRFD principles.

The research team met with the Technical Panel after the completion of Tasks 7-10 to review the results of the resistance factor calibration process and proposed recommendations. This meeting occurred on August 11, 2010.

Task 12: Provide on-site technical support to SDDOT in applying LRFD resistance values and factors to deep foundation design, including a demonstration using parallel data from a suitable project to facilitate application of concepts on paper. If the probabilistic analysis for Task 9 is not possible, review of SDDOT foundation design procedures with respect to an acceptable LRFD approach will be conducted. Technical support and/or recommendations will be provided to SDDOT to implement the code specified resistance factors from an acceptable LRFD manual in lieu of resistance factors specific to South Dakota.

Since the calibration of resistance factors was based on a limited amount of load test data, the research team has assessed the SDDOT drive test predictive resistance design procedure and has provided a number of recommendations for implementation of the LRFD approach using this procedure. The recommendations include the addition of new load test data in order to supplement the database developed for this project and further the calibration of resistance factors for strength limit state design. The recommendations also address procedures that should be adopted by the SDDOT to address several additional requirements of the LRFD approach, including those for serviceability limit state design and assessment of pile driveability.

Task 13: Prepare a final report and executive summary of project results, findings, conclusions, and recommendations including: opportunities for optimization of current practices; potential impacts to existing field, lab and design procedures; qualifying perceived efficiencies; and detailed proposal to assist SDDOT with application of advanced LRFD principles to deep foundation design.

This task has been addressed with this report.

Task 14: Make an executive presentation to the SDDOT Research Review Board at the conclusion of the project.

An executive presentation was given on August 16, 2010.

4 BACKGROUND

4.1 INTRODUCTION

The geotechnical profession has recognized the shortcomings in design methodologies that address design uncertainly based on generally accepted safety factors or an individual design engineer's comfort level and without quantification of actual design uncertainty magnitudes. To that end, the need to systematically address uncertainty has recently been promulgated by the FHWA by implementing a Load and Resistance Factor Design (LRFD) approach and specified process. The SDDOT has implemented calibration-to-fit resistance factors in the process of transitioning to LRFD reliability-based resistance factors for implementing the recommendations of a FHWA Resource Center courtesy review.

This research study is the next step in implementing the FHWA's recommendations by investigating current SDDOT design procedures and determining if calibrated resistance factors can be developed based on current available information. This chapter introduces the fundamentals of deep foundation design primarily focusing on the strength limit state of axial pile capacity. The majority of the information presented herein is derived from the recommended methods of FHWA Pile Foundation Manual (Hannigan 2005) and AASHTO (2007, 2009). Given that AASHTO is specifically for design implementations, we have supplemented that information with the cited references to provide an adequate background basis. These include the works by Fellenius and Hannigan which are heavily cited in AASHTO. Special attention has been paid to recommendations and methods outlined by FHWA and AASHTO including deep foundation design as well as subsurface investigation, laboratory testing, field verification, and wave equation analysis.

NCHRP Report 507 (Paikowsky et al. 2004) discusses the most common static analysis methods for vertical pile capacity relative to AASHTO (2009). Paikowsky et al. (2004) found through a national survey that the three predominant design methods for driven piles were the α-method, β-method and Nordlund's method with 59%, 25%, and 75% of respondents using each, respectively.

The LRFD platform and the background to its methodology are introduced in this Chapter as well. Probability theory is required to appropriately understand the platform and therefore must be discussed. The use of probability theory allows for multiple calibration methods to be used for reliability based designs which are covered here as well.

4.2 WORKING STRESS DESIGN

Working Stress Design (WSD), or Allowable Stress Design (ASD), accounts for uncertainties with the use of a factor of safety (FS). The value of the FS in WSD is based on the design engineer's judgment and the collective experience of the practitioner. When inadequate foundation performance or failures were observed, the FS was increased in an attempt to provide a desired level of safety.

WSD does not separate the differences in variability between the loads and resistance; as such, the FS is applied globally. Loads are generally based on conservative estimates or past experience, while resistances are calculated and reduced by the FS. Though the use of a FS is convenient, it does not account for the various sources of uncertainty and therefore the level of safety is not typically quantifiable. The WSD design approach does not and cannot systematically account for variability in loading, material strength, plasticity, construction quality, etc.

While WSD does not provide a quantifiable level of safety or systematically account for load and resistance variability, is had been used successfully on the vast majority of projects historically. The primary disadvantages of the WSD approach are inefficiencies in how uncertainty is addressed.

4.3 LIMIT STATE DESIGN

In general, a limit state design method utilizes a combination of load factors and resistance factors to account for various uncertainties within the applied foundation loads and the computed foundation resistance, respectively. The load and resistance factors are ideally computed using a probabilistic approach that can explicitly incorporate the magnitude of uncertainty within the design that arises from a number of different sources. Limit state design also requires that the foundation element be designed to satisfy a number of limit states, such as those for strength, serviceability, and extreme loading conditions. A common limit state design method employed in the United States is the Load and Resistance Factor Design (LRFD) approach.

Load factors are generally greater than or equal to unity. Different load factors are applied to the various components that make up the total load. For example, one load factor is applied to the dead load, while another load factor is applied to the live load (AASHTO 2009). The magnitudes of the load factors are often different for each load component and between different limit states as the magnitude of uncertainty within the load components is not equal. In general, however, the loads on the system, along with their uncertainty, are well known and thus the magnitude of the load factors within most limit state design approaches have been well developed by the structural engineering community for use in bridge design.

Resistance factors are generally less than or equal to unity. For resistance factors, however, a single resistance factor can be applied to the total computed nominal resistance of the foundation system or a different resistance factor can be applied to the various resistance components that make up the total resistance (Kulhawy and Phoon 2006). Unlike the load factors, the magnitude of the uncertainty within the resistance of a foundation system is often not well known and thus determination of resistance factors for use in design must involve a reliability theory calibration approach using a well-developed load test database for a specific deep foundation system. The reliability theory calibration approach must be applied for each limit state as the magnitude of uncertainty within the resistance can vary based on the limit state performance objectives.

4.4 DEEP FOUNDATION DESIGN

The process for deep foundation design is not limited to the analysis method. The process also includes site investigation methods, other field data considered in the analysis, and laboratory testing. In addition, other foundation performance aspects, such as how capacity is derived in differing conditions, are important as well. Integral to determining the strength limit state (capacity) is the serviceability limit state outlined by AASHTO (2009), as well as driveability analysis of piles and other design checks. Due to the variability of soils and construction methods, field verification is also an integral part of the design (AASHTO 2009). The next few sections outline the methods based on the previously discussed motivation.

4.4.1 SITE INVESTIGATION

Designing deep foundations utilizing the AASHTO (2009) specifications require site specific subsurface investigation. The investigation is performed in an attempt to determine site characteristics, parameters and properties needed for the design. According to AASHTO (2009), the extent of the investigation "shall be based on variability in the subsurface conditions, structure type, and any project requirements that may affect the foundation design or construction," and should be extensive enough to reveal the geologic and geotechnical conditions present which may impact any phase of the design, construction or structure.

AASHTO (2009) requires that the minimum level of investigation consist of "information adequate to analyze foundation stability and settlement with respect to," the geological formations present, soil profile, engineering properties of the soils, groundwater conditions, ground surface topography and local considerations. The AASHTO LRFD Bridge Design Specifications 4th Edition (AASHTO 2009) provides Table 10.4.2-1, Minimum Number of Exploration Points and Depth of Exploration, as a basis for a minimum subsurface investigation based on the application.

Table 4-1: Summary of information needs and testing considerations for deep foundations (Modified after Sabatini et al. 2002).

Geotechnical Issues	Engineering Evaluations	Required Information for Analyses	Field Testing	Laboratory Testing
Driven Pile Foundations	• Pile end-bearing • Pile skin friction • Settlement • Down-drag • Lateral earth pressures • Chemical compatibility of soil and pile • Driveability • Presence of boulders/ very hard layers • Scour (for water crossings) • Vibrations/heave damage to nearby structures • Extreme loading	• Subsurface profile (soil, ground water, rock) • Shear strength parameters • Horizontal earth pressure coefficients • Interface friction parameters (soil and pile) • Compressibility parameters • Chemical composition of soil/rock • Unit weights • Presence of shrink/swell soils (limits skin friction) • Geologic mapping including orientation and characteristics of rock discontinuities	• SPT (granular soils) • Pile load test • CPT • Vane shear test • Dilatometer • Piezometers • Rock coring (RQD) • Geophysical testing	• Triaxial tests • Interface friction tests • Grain size distribution • 1-D Oedometer tests • pH, resistivity tests • Atterberg Limits • Organic content • Moisture content • Unit weight • Collapse/swells potential tests • Slake durability • Rock uniaxial compression test and intact rock modulus • Point load strength test
Drilled Shaft Foundations	• Shaft end bearing • Shaft skin friction • Constructability • Down-drag on shaft • Quality of rock socket • Lateral earth pressures • Settlement (magnitude & rate) • Groundwater seepage/ dewatering • Presence of boulders/ very hard layers • Scour (for water crossings) • Extreme loading	• Subsurface profile (soil, ground water, rock) • Shear strength parameters • Interface shear strength friction parameters (soil and shaft) • Compressibility parameters • Horizontal earth pressure coefficients • Chemical composition of soil/rock • Unit weights • Permeability of water-bearing soils • Presence of artesian conditions • Presence of shrink/swell soils (limits skin friction) • Geologic mapping including orientation and characteristics of rock discontinuities • Degradation of soft rock in presence of water and/or air (e.g., rock sockets in shales)	• Technique shaft • Shaft load test • Vane shear test • CPT • SPT (granular soils) • Dilatometer • Piezometers • Rock coring (RQD) • Geophysical testing	• 1-D Oedometer • Triaxial tests • Grain size distribution • Interface friction tests • pH, resistivity tests • Permeability tests • Atterberg Limits • Moisture content • Unit weight • Organic content • Collapse/swell potential tests • Rock uniaxial compression test and intact rock modulus • Point load strength test • Slake durability

The extent of and methods for investigation for deep foundations are typically determined by the type of foundation analysis to be performed. In other words, the AASHTO Specifications suggests that there is no one exploration method for all types of deep foundation design. Soil sampling and testing is part of the investigation approach used in the design process and includes undisturbed sampling, disturbed sampling, non-intrusive sampling, and in-situ soil and rock testing (Sabatini et al. 2002). Undisturbed sampling is used for laboratory testing to reflect the in-situ engineering properties of the soil. A summary of information needs and testing considerations suggested by AASHTO are shown in Table 4-1 as prepared by Sabatini et al. (2002) which provides an in-depth review of subsurface investigations and the determination of soil properties.

4.4.1.1 Field Methods

Field methods are often used to determine the in-situ properties of the soil stratum. In-situ methods discussed by Sabatini et al. (2002) include, but are not limited to, the cone penetrometer test (CPT), standard penetration test (SPT), and vane shear test. These methods have been extensively developed to obtain representations of the in-situ soil conditions at the location tested in a manner which is often less expensive than laboratory methods.

AASHTO (2009) recommends that field methods for obtaining subsurface data for design be supplemented with use of correlations to assist in selecting the engineering properties of the soil. Sabatini et al. (2002) states that "correlations in general should never be used as a substitute for an adequate subsurface investigation program, but rather to complement and verify specific project-related information." The correlations should be applied carefully by selecting the ones that are appropriate for the conditions under which the correlations were developed. In some cases, the correlations provide an approximate value and should be further calibrated to local soil conditions (Sabatini et al. 2002). Use of these indirect correlations requires further use of the appropriate corrections for in-situ tests, especially the SPT and CPT (stress level, variations in equipment, etc.).

The SPT and CPT methods must use indirect correlations to determine the state of stresses in soils, which is discussed above. Despite the limitations of the aforementioned tests, the use of in-situ methods to directly correlate the state of stresses in soils has shown promise and may be capable of directly measuring the in-situ stress. The self-boring pressuremeter test (SBPMT) is "one of the few devices capable of providing a direct measurement of the in-situ horizontal stress," (Kulhawy and Mayne 1990). Similarly, the dilatometer test (DMT) provides direct measurement of the horizontal stress state and can be correlated to K_o, a property that is widely used for determining the strength limit state of pile foundations. While these tests provide direct measurement of the horizontal stress state of in-situ soil, they are not without their disadvantages. With respect to the DMT, for example, "local calibration of the DMT should be made relative to K_o measurements obtained with the SBPMT or push-in spade cells" (Kulhawy and Mayne 1990).

4.4.1.2 Laboratory Analysis

Laboratory analyses are grouped broadly into two classes: classification tests and quantitative tests (AASHTO 1988). Quantitative tests include hydraulic conductivity, compressibility and shear strength. Common laboratory tests for strength are the direct shear test, unconfined compression, and triaxial tests. Test results are limited by the quality of the samples obtained; therefore great care must be taken in field sampling of soils for laboratory testing (AASHTO 1988). Since the method and sampling quality have such a great effect on these tests, it is advantageous to collect the most undisturbed sample at an appropriate economical level.

Ideally, sampling would obtain truly undisturbed samples, but this is not economically or physically feasible. Therefore, cost verses sample quality must be balanced and evaluated in the foundation design process. Uncertainty in sample quality can be systematically incorporated into the design through the LRFD calibration process. In any event, it is important that a minimum quality of sampling be obtained with undisturbed sampling methods. Samples for laboratory testing may be considered undisturbed if the following requirements are met:

- No disturbance of the soil structure;
- No change in water content or void ratio; and
- No change in constituents or chemical composition.

AASHTO (2009) recognizes it is very difficult to evaluate these requirements; therefore, the AASHTO Manual on Subsurface Investigations (1988) provides requirements to meet a specified minimum level of quality. According to Sabatini et al. (2002), undisturbed soils samples are required for performing quantitative tests on cohesive soils ranging from soft to stiff consistency, with the thin-walled Shelby tube being the most common method of sampling. Specialty samplers are required for the sampling of soil and soft rock. For soils which are very stiff, brittle, partially cemented, or soils with coarse gravels or stones, the best method of sampling is by block sampling (Sabatini et al. 2002).

Common types of samplers for obtaining disturbed soil samples are: split barrel, retractable plug, augers, and diamond core barrels. Common samplers for undisturbed samples include: Shelby tube, stationary piston, hydraulic piston, Denison, and pitcher (NAVFAC 1986). The California Modified Sampler can be used for various soil property tests; however, they are considered disturbed samples. Samples taken from the California Modified Sampler with a tube extension in fine-grained soils are relatively undisturbed as it acts similar to a Shelby Tube sampler (NDOT 2005). Even more stringent in their requirements, since the California Sampler is a thick-walled sampler, the Army Corp of Engineers (US Army Corps of Engineers 2001) discusses that all samples collected are considered disturbed. Sabatini et al. (2002) considers the California Sampler a Large Penetration Test (LPT). LPT's provide an "intact but very disturbed" sample and can record resistance, but it is "not equivalent to the SPT N-value and is more variable due to no standard equipment methods" (Sabatini et al. 2002). Samplers are classified as thin-walled tube samplers based on their inside clearance ratio (ICR) and the area ratio (AR), so as to determine their potential for soil disturbance. In order for a sampler to be capable of collecting undisturbed samples, "the ICR should be approximately 1% and the AR should be 10% or less" (Sabatini et al. 2002). Furthermore, California samplers have no cutting edge which is needed to obtain undisturbed soil samples. SDDOT refers to this type of sampler as a "California Retractable Plug Tube Sampler".

The disturbance of samples is not only caused by the method of sampling but transportation and handling; therefore, various measures of quality control need to be adopted. Samples taken to the laboratory from the field must be preserved, stored, and shipped under conditions that minimize chances of disturbance (AASHTO 1988). While any method which protects a sample from shock, detrimental temperature changes, and moisture loss may be used, AASHTO (1988) provides specific methods. Samples should be properly cataloged and stored when received at the laboratory and should be tested as soon as possible after arrival. Cohesive soils should be sealed with a non-shrinking flexible micro-crystalline wax installed in several layers in a room at or near 100% humidity, if possible (AASHTO 1988).

Even though undisturbed samples are preferred, in all cases sample preparation for the laboratory strength tests depends on the type of soil obtained. The allowable sample types for each test are outlined in Table 4-2.

Table 4-2: Test methods and requirements (modified after AASHTO 2004).

	Sample Type			AASHTO Standard Test
Test Method	Undisturbed	Remolded	Compacted	
Direct Shear Test of Soils Under Consolidated Drained Conditions	Yes	No	Yes	T 236-4
Unconsolidated, Undrained Compressive Strength of Cohesive Soils In Triaxial Compression	Yes	Yes	Yes	T 296-5
Consolidated, Undrained Triaxial Compression Test on Cohesive Soils	Yes	No	Yes	T 297-6
Unconfined Compressive Strength of Cohesive Soil	Yes	Yes	Yes—if less than 90% saturation	T 208-4

4.4.2 *<u>FUNDAMENTALS</u>*

The most basic level of deep foundation design to support axial loads is how the element derives axial capacity from the soil. The understanding of the derivation of axial capacity allows the use of multiple methods to estimate the static capacity of the foundation. Further, this understanding also allows for the development of settlement prediction methods to account for the service limit state.

The installation of a pile foundation is also a critical segment of design process as an appropriate hammer system must be selected so as to efficiently drive the pile while not overstressing the pile. The use of a dynamic wave equation is used to model this system and predict the behavior of the pile during installation.

Driving formulas are used to estimate the pile capacity during installation (for verification purposes) but are generally considered less than adequate at doing so. Large strain dynamic testing allow for the monitoring and measurement of the pile during driving to determine the axial capacity of the pile at the end of driving. Large strain testing also has the capability of assessing the capacity gain with time by assessing restrike measurements at a much greater accuracy than the driving formula. Load tests are the only method to determine the actual capacity of a pile and the load-deflection curve but are expensive and can be time consuming.

4.4.2.1 Axial Capacity

The capacity of piles and drilled shafts consists of the side resistance over the length of the foundation element and the end area. The general formula, expanded for understanding, for the ultimate geotechnical capacity of both piles and shafts as described by Reese (2006) is:

$$R_n = R_s + R_p = (q_s)A_s + (q_p)A_p$$

Equation 4-1

where,

R_n = nominal total pile resistance;

R_s = pile side resistance;

R_p = pile tip resistance;

q_s = unit load transfer in shaft resistance;

q_p= unit load transfer in tip resistance

A_p = are of pile tip; and

A_s = side surface area of the foundation.

The stresses and forces placed upon a pile in cohesionless soil are modeled in Figure 4-1.

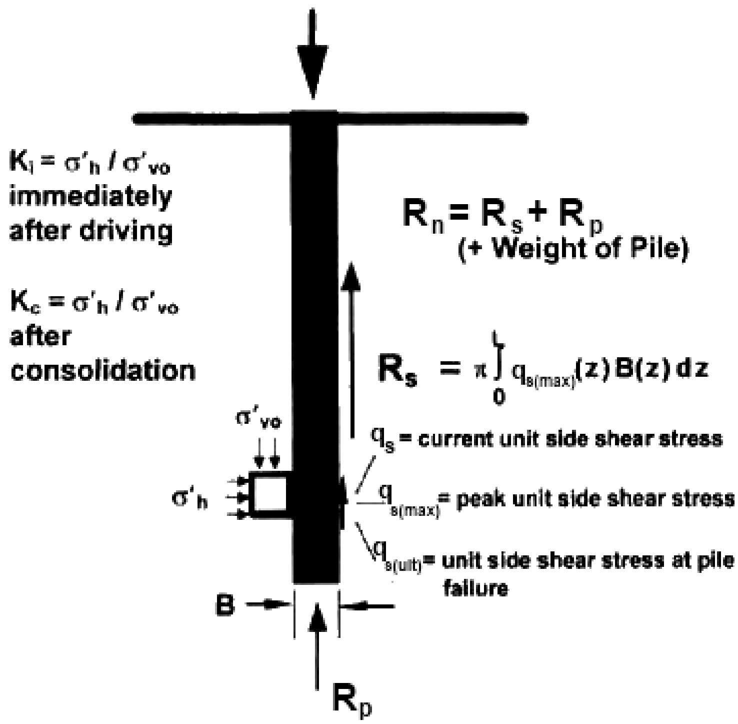

Figure 4-1: Stresses and forces acting upon axially loaded piles (after O'Neill 2001). ***R_n*** **represents total pile capacity,** ***R_p*** **represents the tip resistance and** ***R_s*** **represents the side resistance.**

The stresses acting upon the shaft, or side, of the pile compose the side resistance and are equal to the sum of unit side shear resistance between the pile and soil and their respective unit areas. The side shear resistance is a function of the internal angle of friction of the soil and adhesion between pile and soil material, and the normal stress acting upon the pile, which is generally the horizontal effective stress. The horizontal effective stress acting on the pile is often related to the vertical effective stress by the coefficient of lateral pressure, K. In situations where the loading creates an undrained condition in the soil, namely in the relatively short term loading of cohesive material, the undrained shear strength is often considered the primary indicator of capacity. It should be noted that side resistance is not constant along the pile and varies due to several factors including geomaterial type, installation method, pore-water pressures, shaft material type, construction methods and many other conditions that are not fully understood. In practice, the first consideration is often the geomaterial type. Mohr-Coulomb soils, intermediate geomaterials (IGM), and rock all have different load-settlement curves, design procedures and strengths, and thus each must be considered appropriately (O'Neill and Reese 1999). A hypothetical load-settlement curve of a drilled shaft in different geomaterial types, along with common factors which influence the shaft behavior, is shown in Figure 4-2. The load-settlement curve demonstrates that in order to gain resistance there must be some degree of movement of the pile relative to the soil.

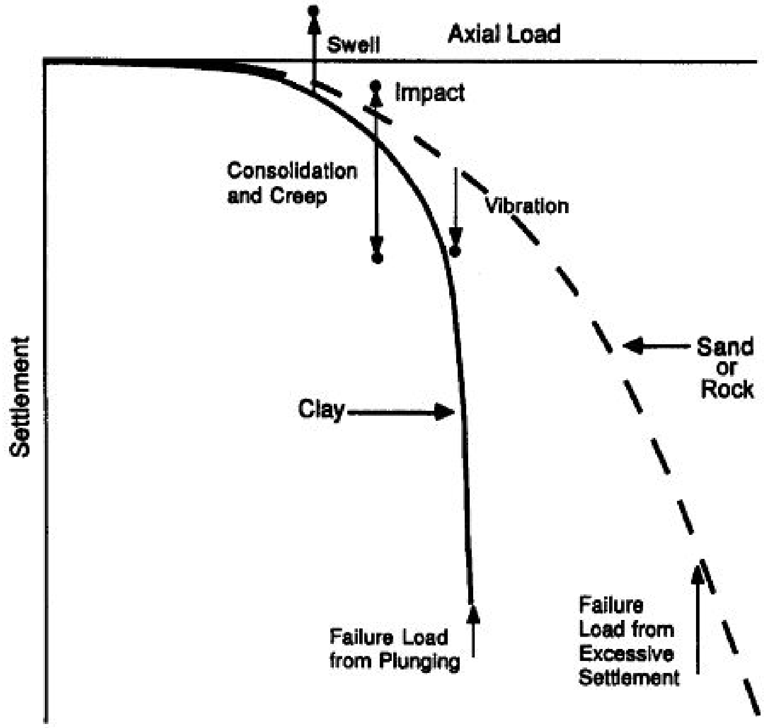

Figure 4-2: Hypothetical load-settlement relations for drilled shafts, indicating factors that influence shaft behavior under axial load (O'Neil and Reese 1999).

The stresses acting along the base of the foundation account for the end bearing. These stresses are dependent on the shear strength properties of the soil below the foundation element. The drainage condition of the soil beneath the foundation determines the method of design. End bearing is affected by not only the geomaterial type, but various other factors including the diameter of the tip and the installation method. Again, in order to gain resistance, there must be some degree of movement of the pile to mobilize the stresses.

It should be noted that the settlement required to reach full mobilization of the soil resistance differs between the side resistance and end bearing. In general, it is accepted that the end bearing resistance is fully mobilized at a settlement of about 5% of the pile's toe diameter (O'Neill 2001). However, movement to fully mobilize side resistance is more difficult to quantify due to the complex stress regime that exists around a pile after driving or drilling (O'Neill 2001). However, it is generally accepted that the magnitude of displacement for side resistance is much lower than that for end bearing. In regards to mobilization of end bearing, a linear increase in resistance with displacement has been shown and no failure value with settlement has been shown in full scale testing (Fellenius 2001, Fellenius 2006). Therefore, the term "fully mobilized" end bearing typically refers to a corresponding pressure at a limited displacement. When end bearing is fully mobilized, the side resistance may have actually decreased from its peak value to a residual value due to the greater magnitude of movement required to mobilize end bearing and the possibility of strain softening of the soils providing by side resistance (Hannigan et al. 2005). Designers should also be aware that while settlement at the head of the pile may be observed, elastic shortening of the pile also occurs, thus the penetration at the pile-head is not equal to the penetration of the pile toe.

Uplift resistance of a pile or shaft is limited to the side resistance, unless the shaft is under-reamed (belled) during installation (O'Neill and Reese 1999). Multiple researchers have noted that the uplift resistance in high permeability soils is less than that of the side resistance, partially due to a "decrease of vertical effective stress during pull" and that under tension, the width of the pile decreases, while it increases in compression (Fellenius 2002). Even though Fellenius (2002) reports some reservation to these findings, the majority of researchers have found them appropriate (Elhakim and Mayne 2002, O'Neill 2002). Analyses of databases of large scale test data by Elhakim and Mayne (2002), and O'Neill (2001), as well as the laboratory test data performed by De Nicola (1996) suggest a measurably smaller side resistance in uplift than compression. O'Neill (2001) recommends a value of 74% of the compressive resistance, whereas Elhakim and Mayne (2002) recommend a value of 71%. The LRFD resistance factor for uplift provided by AASHTO (2009) is reduced to 80% of the static compressive shaft resistance factor.

4.4.3 STATIC CAPACITY PREDICTION METHODS

Static capacity prediction methods suggested by FHWA are used to estimate pile penetration lengths, bid quantities, and long term and short term capacity of the foundation (Hannigan et al. 2005). This section briefly outlines the recommended methods relative to FHWA (Hannigan et al. 2005) and AASHTO (2009) and is cited to provide the original author(s) credit where possible. Methods for drilled shafts typically parallel those for driven piles but are more variable and thus the work of O'Neill and Reese (1999) is suggested.

4.4.3.1 Effective Stress Analysis, β-method

The effective stress analysis method is commonly referred to as the β-method in reference to the dimensionless factor which relates vertical effective stress to the horizontal effective stress upon the foundation element. It is extensively used in sands, but is also appropriate in clays depending on the loading and pore-water pressure conditions. The method is used by approximately 25% of state highway professionals for driven piles and 41% for drilled shafts (Paikowsky et al. 2004).

4.4.3.1.1 Side Resistance

When designing based on effective stress, the soil type governs the basic formula for side resistance. The original method was based on the now disproved concept of a critical depth (Fellenius and Altaee 1995); however, the current method does not consider a limiting value of unit side resistance. Meyerhof (1976) provided the governing equation for side resistance in sands as:

$$q_s = K_s \sigma_v' \tan \delta$$

Equation 4-2

where,

K_s = the coefficient of earth pressure along the shaft;

σ_v'= the effective overburden pressure along the shaft; and

δ = angle of skin friction between the pile and soil.

The value of the coefficient of lateral earth pressure is difficult to obtain and differs before and after construction; therefore, the Bjerrum-Burland coefficient, or β, was derived by Burland (1973) which is equal to:

$$\beta = K_s \tan \delta$$

Equation 4-3

therefore,

$$q_s = \beta \sigma_v'$$

Equation 4-4

There are multiple recommendations for values of the coefficient for sands and gravels which are specific to pile types and or conditions, but the value can vary from 0.3-0.9 for sands and 0.35-0.8 for gravels (Fellenius 2009).

When considering the case of silts and clays, Meyerhof (1976) proposed the following equation, which provides the model for unit side resistance:

$$q_s = c + K_s \sigma_v' \tan\phi \le c_u$$

Equation 4-5

Based on the fact that excess pore pressures dissipate relatively quickly in terms of loading and construction, it "can be concluded that the ultimate skin friction of piles in saturated clay can be estimated from the drained shear strength of remolded soil for which the cohesion may usually be taken as zero" (Meyerhof 1976). Given this, the governing equation for side resistance once again becomes Equation 4-2 with a limiting value of the undrained shear strength c_u of the undisturbed clay (Meyerhof 1976).

The values for β in silts and clays, which are normally consolidated, are relatively well-defined and predicted. Therefore, the values of β for silts and clays are relatively constant and usually within the range of 0.25-0.50 and 0.15-0.35, respectively (Fellenius 2009). Values recommended by Hannigan et al. (2005) are provided in

Figure 4-3. AASHTO (2009) recommends the use of data provided in Figure 4-4 for overconsolidated clays with a limiting β-value of 2.0 for heavily overconsolidated clays as the presented relation tends to overestimate the shaft resistance in these situations. The value of β in overconsolidated clays is less well known and can vary greatly; therefore, caution is recommended.

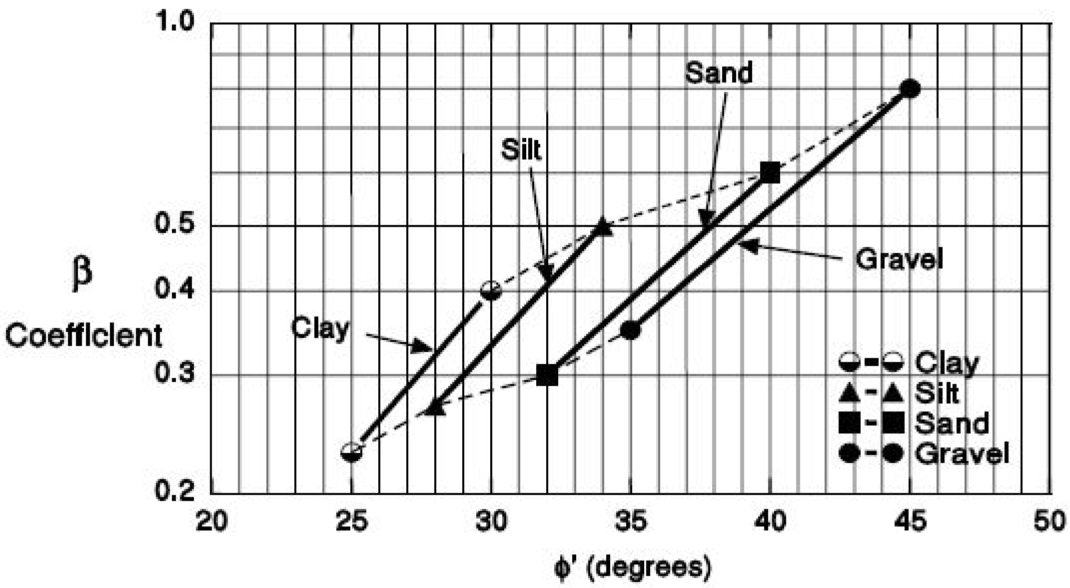

Figure 4-3: Estimation of β Coefficient Versus Φ (Hannigan et al. 2005)

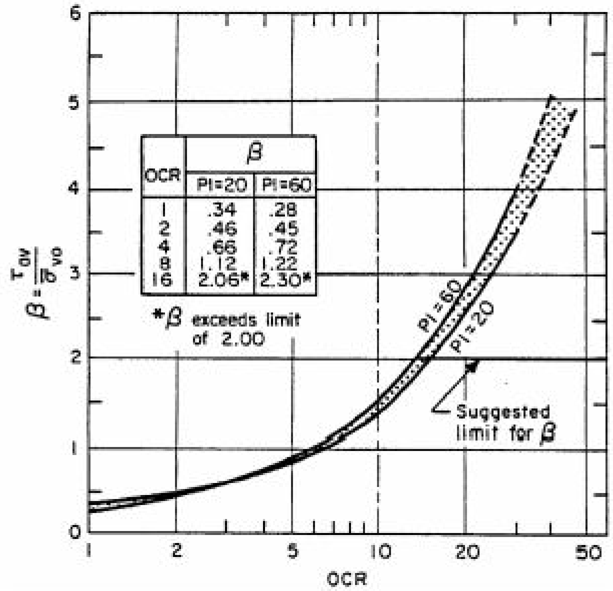

Figure 4-4: β Versus OCR for Displacement Piles (AASHTO 2009)

4.4.3.1.2 <u>Tip Resistance</u>

The unit tip resistance for the effective stress condition as recommend by the Hannigan et al. (2005) is shown in Equation 4-6.

$$q_p = N_t \sigma_t$$

Equation 4-6

where,

q_p = unit tip resistance;

N_t = Toe bearing capacity coefficient; and

σ_t = Effective overburden pressure at the pile toe.

AASHTO (2009) recommends the N_t values provided by Fellenius (1999, 2009) for different types of soils as shown in Table 4-3. As previously mentioned, Fellenius (1999) states that the toe resistance exhibits no true failure value even under movements of 50 mm, therefore, the N_t coefficients provided correspond to movements of the pile toe of no more than 5 to 12 mm. If there is an absence of local test data, Figure 4-5 may be used to estimate N_t. AASHTO (2009) does not comment on the use of N_t values, and therefore, recommends the use of the FHWA-Nordlund method of determining the tip resistance of piles in sands and non-plastic silts.

Table 4-3: Ranges of N_t Coefficients (Fellenius 1991; Fellenius 2009)

Soil Type	Phi	N_t
Clay	25 – 30	3 – 30
Silt	28 – 34	20 – 40
Sand	32 – 40	30 – 150
Gravel	35 – 45	60 – 300

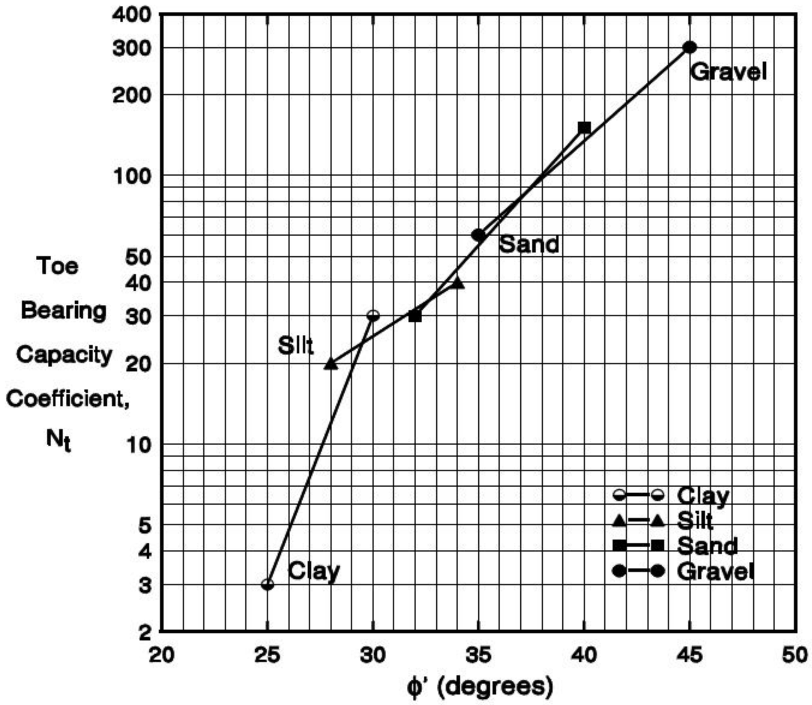

Figure 4-5: Chart for estimating N_t versus Φ' (Fellenius 2009, Hannigan et al. 2005).

4.4.3.2 Total Stress Analysis

The load carrying capability of a single pile in a cohesive soil is often determined by performing a total stress analysis because the undrained shear strength of the soil will govern. Therefore, the total load carrying capability of the pile will be the sum of the end bearing and side resistance components.

4.4.3.2.1 Side Resistance – α-method

The α-method is a total stress analysis method which uses the empirical adhesion factor, α, to relate the undrained shear strength of soil to the unit shaft resistance of a pile or shaft in clay. Each form of the α-method is somewhat different and includes the American Petroleum Institute (API) method, the United States Army Corps of Engineers (1991), and the recommendations by Tomlinson (1980) which is the method recommended by AASHTO (2009).

The α-method is commonly used by a large number of engineers in practice. It has been shown to provide "reasonable results for both displacement and non-displacement piles in clay" (AASHTO 2009). However, O'Neill (2001) suggests its use be limited because the undrained shear strength, s_u, is not a unique soil property, but rather an "artifact of the way the soil is tested" and because the design should be an effective stress solution. Still, the method deserves discussion and thus the side resistance of the foundation can be determined from Equation 4-7.

$$q_{s(\max)} = \alpha s_u$$

Equation 4-7

where

$q_{s(max)}$ = peak unit side shear stress;

α = adhesion factor; and

s_u = undrained shear strength along the pile.

The adhesion factor, as previously mentioned, is empirical and therefore is a function of the methods used to produce that database. Functions for the adhesion factor have been published by multiple sources, including the US Army Corps of Engineers (1991) as shown in Figure 4-6. Further correlations are presented by the FHWA (Hannigan et al. 2005), including values for the adhesion factor as recommended by the American Petroleum Institute (API), as well as Tomlinson's recommendations. AASHTO (2009) similarly describes the method using Tomlinson's values, as presented in Figure 4-7. Both sources recommend the boxed area of H-piles for the calculation of the side resistance rather than the surface area of the pile.

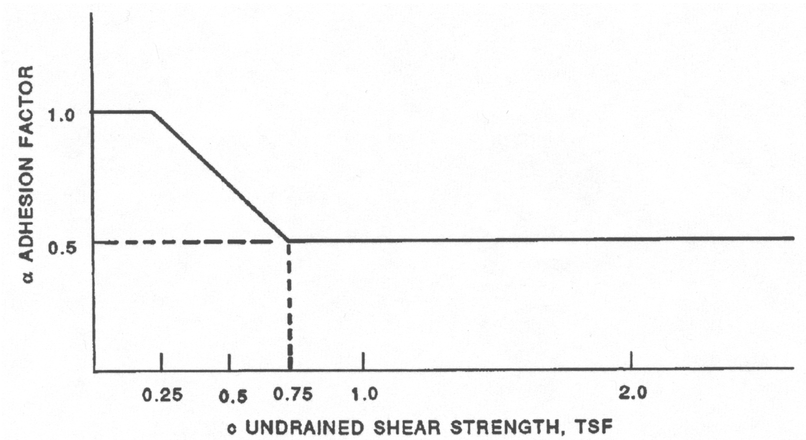

Figure 4-6: Values of alpha versus shear strength (US Army Corp of Engineers 1991).

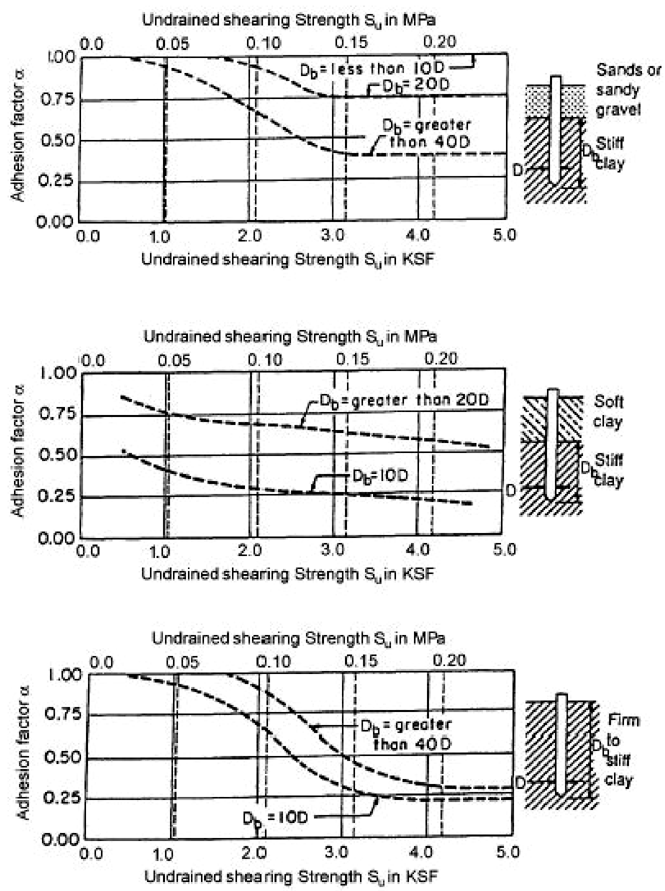

Figure 4-7: Design Curves for Adhesion Factors for Piles Driven into Clay Soil (after Tomlinson 1980).

4.4.3.2.2 *Tip Resistance*

When utilizing a total stress analysis method, the FHWA (Hannigan et al. 2005) determines the unit toe resistance using Equation 4-8. Hannigan et al. (2005) notes that since the required movement to mobilize tip resistance is much greater than that of side resistance, the toe resistance is sometimes ignored, except in hard cohesive deposits.

$$q_p = S_u N_c$$

Equation 4-8

where,

q_p = unit end-bearing capacity;

S_u = undrained shear strength at the tip of the pile, usually taken as the average over a distance of two diameters below the tip of the pile; and

N_c= dimensionless bearing capacity factor dependent upon the pile diameter and embedment, usually taken as 9.

4.4.3.3 FHWA-Nordlund Method

The Nordlund method suggests that the shaft resistance is a function of the volume of soil the pile displaces during installation, friction angle of the soil, friction angle on the sliding surface, the taper of the pile, the effective unit weight of the soil, pile length, and minimum pile perimeter (Hannigan et al. 2005). The Nordlund method equation is presented in Equation 4-9. The method is semi-empirical, based on the analysis of load test data and the field observations of Nordlund. The method is intended for cohesionless soils.

$$R_n = \alpha_t N_q' A_t p_t + \sum_{d=0}^{d=D} K_s C_f p_d \frac{\sin(\delta + \omega)}{\cos \omega} C_d \Delta d$$

Equation 4-9

where,

d = depth;

D = Embedded pile length;

K_s= Coefficient of lateral earth pressure at depth d;

C_f = Correction factor for $K\delta$ when $\delta \neq$ □;

p_d = Effective overburden pressure at the center of depth increment d;

δ = Friction angle between pile and soil;

ω = Angle of pile taper from vertical;

ϕ= Soil friction angle;

C_d = Pile perimeter at depth d;

d = Length of pile segment;

α_t = Dimensionless factors (dependent on pile depth-width relationship);

N'_q = Bearing capacity factor;

A_t = Pile tip area; and

p_t = Effective overburden pressure at the pile tip.

Figures producing the various correction factors are located in Hannigan et al. (2005) as well as the original documents by Nordlund (1963, 1979).

4.4.4 SETTLEMENT PREDICTION

According to AASHTO (2009), settlement is a portion of the service limit state which includes vertical and horizontal movements, overall stability, and scour. Rideability, economy, and structural tolerances to movement are factors which need consideration when designing for the service limit state. AASHTO (2009) discusses tolerable movements for the foundation relation to applicable load combinations and methods to determine tolerable movements. Tolerable movements are determined by empirical procedures or structural analysis. Deep foundations must also consider the possibility of downdrag loads in the service limit design.

AASHTO (2009) limits the discussion of settlement methods for deep foundations to the use of the equivalent footing method for pile groups as presented by Terzaghi and Peck (1967) and references Hannigan et al. (2005) for further methods. Hannigan et al. (2005) presents a more in-depth discussion of pile settlements, including methods based on SPT and CPT data, the Janbu tangent modulus approach and the neutral plane method.

4.4.5 DYNAMIC ANALYSIS

Dynamic analysis provides a realistic analysis of the pile driving process. The analysis assists the designer with the following:

- Developing curves of capacity verses pile blow count for various lengths;
- Selecting the most efficient combination of driving equipment to ensure that the piles can be driven to the required depth;
- Selecting the minimum pile section for design; and
- Selecting the proper pile type to minimize the possibilities of overstressing the pile.

Most dynamic analyses are performed using the wave equation, a term applied to several computer programs. According to Hannigan et al. (2005), when the hammer strikes the pile system, a compression wave is transferred down the pile at a speed which is a function of the elastic modulus and mass density of the pile until it reaches the embedded portion of the pile. At that moment, the wave's amplitude decreases due to the soil forces on the pile. The wave may reflect off the pile toe and cause both permanent pile displacement as well as tensile forces as the wave travels toward the pile head. The Smith-type wave equation analysis (Smith 1960) models the driving system and pile as a series of segments. Each segment is composed of a mass and spring. Coefficients of restitution are used to model energy losses with values ranging from zero for perfectly elastic conditions to one for perfectly plastic conditions. Soils are modeled using elasto-plastic springs for the static resistances and dashpots for the dynamic resistances. The model is shown in Figure 4-8. The displacement shown in Figure 4-8 at which the ultimate static resistance is achieved is called a quake, after which the dynamic resistance of the soil is activated and modeled as a dashpot.

The analysis consists of a series of time steps using the model as well as the ram velocity. The analysis concludes when the pile toe begins to rebound. Pile stresses are capable of being calculated and therefore driveability can be determined as required by AASHTO (2009).

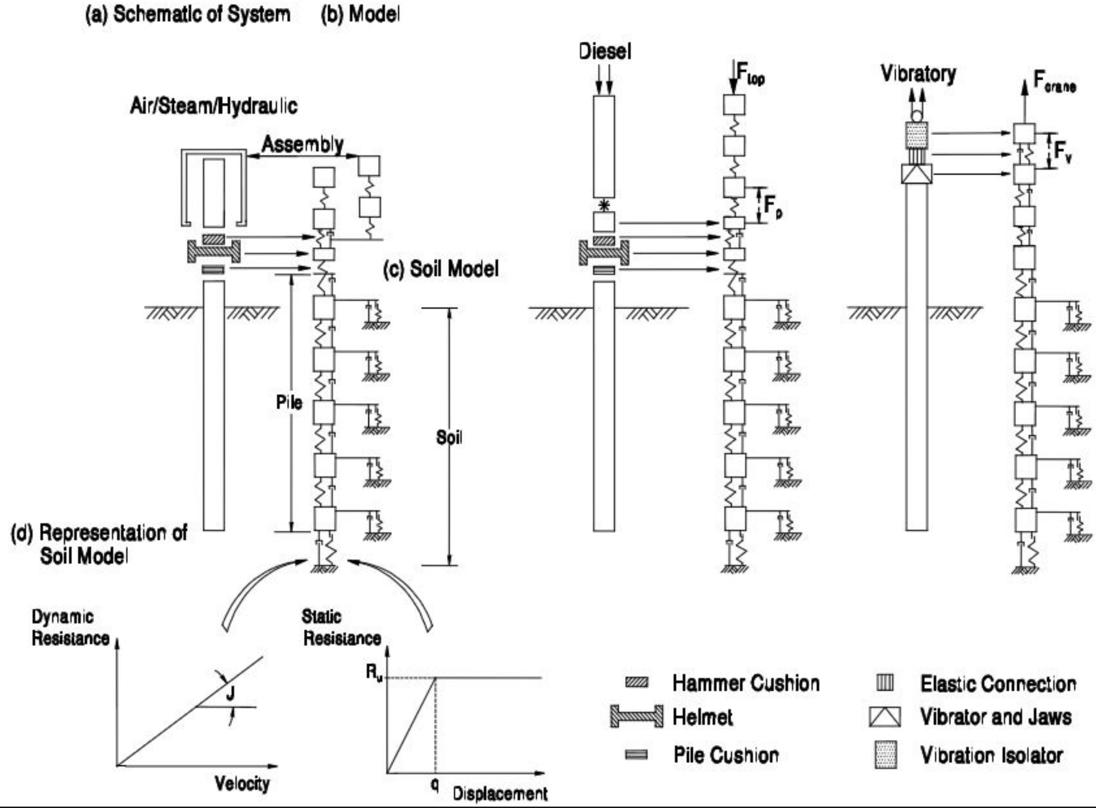

Figure 4-8: Typical Model for Wave Equation Analysis (Hannigan et al. 2005).

4.4.5.1 WEAP

Wave Equation Analysis of Piles (WEAP) is the most commonly used computer program to perform a dynamic analysis. It was originally created to model the approach developed by Smith (1960) and developed in parallel with a similar program created by Texas Transportation Institute, both funded by the FHWA. WEAP was eventually transitioned to a proprietary program now called GRLWEAP and developed by Pile Dynamics, Inc. which has various improvements over the original program. Wave equation programs, such as GRLWEAP, are capable of providing pile stresses, permanent set, and blows per foot. The analysis of the pile in relation to the blows per foot for various ultimate capacities creates a "bearing graph" which relates ultimate capacity to penetration resistance (Hannigan et al. 2005).

Hannigan et al. (2005) states that the bearing graphs from wave equation analysis provides the designer with a relationship between ultimate capacity and penetration resistance, as well as the driving stresses in the pile for penetration resistances. However, AASHTO (2009) cautioned against the use of bearing graphs during driving to predict pile bearing resistance as many of the key input values are highly variable and should have at least some level of verification. When wave equation analyses are calibrated by the use of high strain measurements or load test data, discussed subsequently, higher reliability arises and has a key impact on the resistance factor used for design (AASHTO 2009).

4.4.6 FIELD VERIFICATION

The static analysis methods previously discussed are all estimates of the pile-soil system and may over or underestimate the capacity at calculated depths. Soil layer variability and the real world soil-pile interaction often results in different actual capacity corresponding to the same depth of the predicted capacity by the static methods. This variability in actual and predicted capacity requires the use of field verification of the capacity of the pile. When actual capacity is less than predicted, additional pile length may be needed. When more actual capacity is achieved than what was predicted, pile lengths may be reduced.

Multiple methods of field verification are used in foundation engineering. Driving equations are the easiest to use but also the least consistent or accurate. Static load tests provide the actual load-displacement curve of the pile and therefore are the most accurate but are also the most expensive. Other methods have been developed and provide a spectrum of accuracy between driving equations and static load tests. Those methods are discussed subsequently.

4.4.6.1 Driving Equations

Driving equations, also called dynamic formulas, attempt to rationally determine the pile's capacity based on the penetrations of the pile and kinematics of the hammer energy. Kinematic based driving equations, such as the Engineering News Record (ENR) formula have been used for years, but have proven inadequate in their ability to estimate pile capacity (Fragaszy et al. 1985). Driving equations, which attempt to model other factors such as energy losses, also have shown poor correlation to capacity (Hannigan et al. 2005).

According to the FHWA (Hannigan et al. 2005) driving equations are fundamentally incorrect beyond their poor predictive capabilities. Driving equations poorly represent the pile driving system and components as well as assume a rigid pile. Soils are also modeled incorrectly in that driving formulas assume that the "soil resistance is constant and instantaneous to the impact force" (Hannigan et al. 2005). Considering their poor performance and incorrect modeling of the actual pile system and soils, the FHWA strongly suggests driving formulas not be used unless very well supported empirical correlations to local conditions and hammer systems exist from static load tests (Hannigan et al. 2005).

4.4.6.2 High Strain Dynamic Testing of Piles

High strain dynamic testing of piles, commonly referred to as dynamic load testing, is a fast, reliable and cost effective method of assessing pile capacity. The method uses strain and acceleration measurements near the pile head during driving to evaluate aspects of the pile system including the pile integrity and to estimate the static pile capacity (Hannigan et al. 2005). The most commonly used dynamic load testing is the Pile Driving Analyzer (PDA), developed by Pile Dynamics, Inc. The PDA performs real time computations to analyze and record the transferred energy, driving stresses, structural integrity and pile capacity.

The PDA obtains the best results when the soil damping constants are calibrated from static load tests. However, incorrect results can be obtained when calibration is performed against special wave equation programs designed to infer capacity from pile-head measurements. As a result, the PDA is very specialized and requires an experienced operator. To account for the time dependency of capacity, the pile should be restruck after a specified time (e.g. 7 to 14 days) to evaluate the sensitivity.

The PDA uses the Case Method to estimate pile capacity from the measured force and velocity. The measurements are applied to the Case Method for each blow in real time. Hannigan et al. (2005) provides an extensive discussion on the background and implementation of the Case Method. AASHTO (2009) states that the use of signal matching with the dynamic analysis, or PDA, should always be used to determine the static capacity if no static load test is performed.

4.4.6.3 Signal Matching with High Strain Dynamic Testing

Measurements from the PDA can be coupled with a signal matching technique to refine the predicted estimate of capacity. The complimentary computer program CAse Pile Wave Analysis Program (CAPWAP) is the most commonly used for that purpose. According to the FHWA (Hannigan et al. 2005), CAPWAP uses the measured force and velocity records from the PDA from one hammer blow and analyzes that data with the wave equation model. The use of CAPWAP allows the calculation of the "ultimate static pile capacity, the relative soil resistance distribution, the dynamic soil properties of quake and damping, and the driving stresses throughout the pile" (Hannigan et al. 2005). While not equivalent to a static load test for capacity verification, the use of CAPWAP is considered more accurate than the PDA using the Case Method alone.

CAPWAP matching is performed by an initial estimation of the soil parameters and resistance distribution. The model is then applied using the measured acceleration and compares the measured force to the calculated force at the pile head. If the results do not agree, the soil model is adjusted and the program analyzes the measured and computed wave and quantifies the quality of the match between the two. The program iteratively solves the process until the best match available is used, at which point the best estimate of static pile capacity, resistance distribution and soil model characteristics are calculated.

While dynamic testing is accurate, it does have disadvantages. The use of these methods requires full mobilization of the soil resistance and therefore requires large toe movements. FHWA (2005) notes that blow counts approaching 10 blows per inch are unlikely to produce large enough toe movements to fully mobilize the soil resistance near the toe and therefore tends to under predict the capacity of the pile. It is also noted that large open ended pipe piles and H-piles, which do not fully plug during driving, may develop a plug under static load tests, which also provides a lower capacity value than the static load test. AASHTO (2009) similarly notes that large blow count values hinder the accuracy of signal matching. Likins (2005) has documented limitations in CAPWAP in that the results tend to over predict capacity in shales. *This could have an impact on CAPWAP use in South Dakota*.

Table 4-4 presents a summary of the major available dynamic methods for evaluating pile capacity. The methods are subdivided by design vs. construction and the need for data obtained through dynamic measurements.

Table 4-4: Comparison of methods for evaluating pile capacity: advantages, disadvantages, and comments (after Paikowsky et al. 2004). ENR = Engineering News Record; FS = Factor of Safety; BOR = Beginning of Restrike; EOD = End of Driving

Category	Method	Advantages	Disadvantages	Comment
Design Stage	WEAP (Smith 1960, Goble et al. 1976)	• Equipment Match • Driveability 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 Equation. (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	• 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?

The statistical results of research funded by the FHWA (Rausche et al. 1996) compares predictive methods to verification methods (Hannigan et al. 2005) is shown in Table 4-5.

Table 4-5: Mean Values and Coefficients of Variation for Capacity Prediction Verses other Verification Methods (Hannigan et al. 2005). EOD is end of driving and BOR is beginning of restrike.

Prediction Method	Status	Mean	COV	# Piles
Static Analysis using penetration tests	-	1.3	0.68	89
Standard WEAP	BOR	1.22	0.35	99
Hammer Performance Adjusted WEAP	BOR	1.16	0.35	99
CAPWAP	BOR	0.92	0.22	99
Engineering News Formula	EOD	1.22	0.74	139
Engineering News Formula	BOR	1.89	0.46	122
Gates Formula	EOD	0.96	0.41	139
Gates Formula	BOR	1.33	0.48	122

4.4.6.4 Static Load Testing

Static load testing is considered the most accurate method of determining load capacity. Load tests are also used to confirm the capability of the pile-soil system to carry the design load, determine settlements under loading, LRFD calibration, and implementation of new analysis methods (Hannigan et al. 2005). Hannigan et al. (2005) also states that, in concert with the load test, a detailed subsurface exploration, knowledge of the subsurface conditions and a static analysis should exist for the test site. The advantages of load testing and when to perform load tests are described in Hannigan et al. (2005) and Kyfor et al. (1992).

AASHTO (2009) recommends the use of ASTM D 1143 and the "Quick Load Test Method" to minimize the possibility of problems arising from longer load tests. The use of a modified Davisson's Method is recommended by Cheny and Chassie (2000) as the Davisson Method under predicts the axial resistance of large diameter piles. The soil resistance distribution along the pile shaft is determined with the use of telltales or strain gages. The method to evaluate the soil distribution between two measuring points, assuming no residual stresses, is outlined by Kyfor et al. (1992). The presence of residual loads, which are locked into the pile, is determined by the lack of complete rebound after a hammer blow. The presence of residual stresses can result in an inaccurate load transfer curve if not accounted for in analyzing the load test results. This is shown in Figure 4-9.

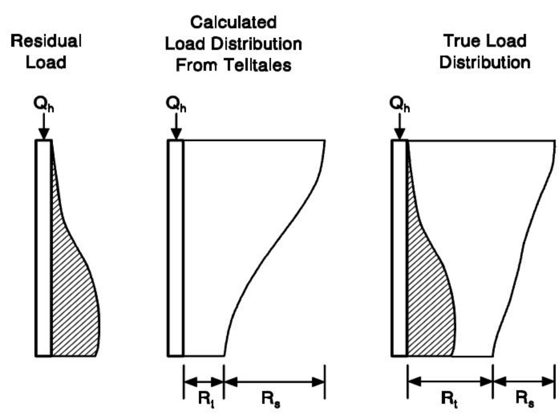

Figure 4-9: Example of Residual Load Effects on Load Transfer Evaluation (Hannigan et al. 2005).

The use of other load test methods is outlined in Hannigan et al. (2005), including the use of the Osterberg Cell (O-cell) for drilled shafts and the Statnamic method, which is a dynamic load test method. The O-cell is a sacrificial loading device attached along the shaft or near the shaft toe prior to installation.

The O-cell does not require a reaction frame as it expands internally, thus the pile shaft and toe resistances counteract each other. The Statnamic method uses a reaction mass placed on top the pile. The reaction mass is placed above a propulsion ignition source and is instrumented to measure acceleration. The ignition source causes acceleration in the reaction mass from which the force applied to the pile is determined. There are multiple methods of analysis for Statnamic tests which account for dynamic rate effects and pile resistances differently.

4.5 LOAD AND RESISTANCE FACTOR DESIGN (LRFD)

The following sections provide a summary of important concepts in applying LRFD. This includes an introduction to probability theory and LRFD fundamentals, followed by resistance factor calibration methods.

4.5.1 PROBABILISTIC THEORY AND LRFD FUNDAMENTALS

It is well known and accepted by practicing engineers that uncertainty exists in any design. It is also known that those uncertainties arise from many sources. Typical sources of uncertainty are inherent variability, measurement errors, transformation uncertainty, construction processes, and model error (Phoon and Kulhawy 1999, Misra and Roberts 2006). Inherent variability results from the natural process of soil deposition, consolidation, erosion, etc, which leads to vertical and horizontal fluctuations within soil properties over a given site. For example, it has been reported that within any site, the vertical scale of fluctuation can be between 3 to 18 feet, while the horizontal scale of fluctuation can be between 100 to 150 feet (Phoon et al. 1995). A very thorough subsurface investigation of a site, which includes frequent vertical and horizontal sampling, will result in a better defined subsurface profile and, consequently, an improved understanding of the magnitude of inherent variability. Measurement errors result predominately from the field investigation and laboratory testing of soils and rock. However, this type of uncertainty can also arise during load testing of deep foundation systems where it is not possible to obtain exact measurements in order to assess capacity. Transformation uncertainties arise from the attempt to correlate the field, laboratory, or load testing data into design properties. One example of this is the development of the numerous correlations between the Standard Penetration Test (SPT) and soil strength properties, which often leads to values that under or over predict the actual magnitude. Uncertainties from the construction or installation process for deep foundations are possible due to the variability in the construction materials, foundation geometry, installation technique, and magnitude of soil disturbance. Finally, the selection of a model, along with the assumptions made in using that model, will result in uncertainty in the design. For example, the various models reported in the literature to predict deep foundation capacity can contain a built-in factor of safety or were developed using assumptions that are unknown to the designer which can lead to significant uncertainty in the design.

Typically, design uncertainty is handled by assigning a global factor of safety to the calculated ultimate capacity or assigning nominal safety factors to the individual components that make up the total design. Regardless of the method used, safety factors are typically assigned based on an individual design engineer's comfort level and without quantification of actual design uncertainty magnitudes (Kulhawy and Phoon 2006). This can often lead to designs that are very conservative, and thus inefficient, or potentially to designs that may be unsafe. In order to address these issues, a reliability- based design (RBD) method, such as the Load and Resistance Factor Design (LRFD) approach, needs to be implemented in order to rationally identify and quantify the uncertainties in the design process. In an RBD approach, the magnitude of design uncertainty can be explicitly incorporated into the overall design, therefore leading to a safe, consistent, and efficient design (Roberts et al. 2008).

In a general RBD approach for the design of deep foundations, the soil/IGM/rock properties that comprise the capacity, or resistance, of the deep foundation are generally characterized using a probability distribution function (PDF). The PDF, or "Bell Curve", is described with a mean (i.e. average) value of the soil property, along with a value that describes the magnitude of variability within that soil property. This magnitude of variability is the direct result of the aforementioned sources of uncertainty. Generally, the variability is described using the coefficient of variation, or COV, which is the ratio of the standard deviation of the soil property to the average. Table 4-6 provides typical values of the COV for numerous soil properties commonly used in the design of deep foundation systems. Due to the various sources of uncertainty that comprise the magnitude of a soil property for use in design, it is observed in Table 4-6 that the COV can vary quite significantly between the various soil properties.

Table 4-6: Typical COV values of soil properties (adapted from Duncan 2000).

Measured or Interpreted Parameter Value	Coefficient of Variation, COV (%)
Unit weight, γ	3 to 7%
Buoyant or effective unit weight, γb	0 to 10%
Peak friction angle, φ'p	2 to 13%
Critical state friction angle, φ'cs	0 to 3%
Undrained shear strength, su	13 to 40%
Compression index, Cc	10 to 37%
Preconsolidation stress, σ'zc	10 to 35%
Standard penetration blowcount, N	15 to 45%
Electric cone penetration test, qc	5 to 15%
Vane shear test undrained strength, suVST	10 to 20%

Ultimately, the required soil properties are utilized within a prediction model in order to compute an ultimate capacity, or resistance, for the deep foundation system. Since the soil properties possess a magnitude of variability and are described using a PDF, so will the foundation resistance. Although not previously discussed, the loads that are applied to the system will also possess some uncertainty within their magnitude and direction (Budhu 2008) and are thus also described using a probability distribution function. Unlike the foundation resistance, however, the mean values of the loads on the system are generally well-known and there is significantly less variability within the loads when compared to the resistance. In fact, the structural engineering community has extensively researched loads on bridges and other structures and has assigned load factors for various cases to account for the uncertainty. To that end, for a given site, the PDF of the load is superimposed with the PDF of the foundation resistance as shown in Figure 4-10. In Figure 4-10, μ_Q and μ_R are the mean values of the load and foundation resistance, respectively. The shaded area where the PDFs overlap in Figure 4-10 is computed to be the probability of failure, p_f, as it represents the possibility of the load exceeding the foundation resistance given the overall uncertainties within the design.

In order to simplify the generalized RBD approach shown in Figure 4-10, the PDF of the resistance can be subtracted from the PDF of the load. This will result in a PDF that represents the safety margin as shown in Figure 4-11. In Figure 4-11, μ_s is the mean of the safety margin and σ_s is the standard deviation. The shaded area denotes the probability of failure, p_f, of the deep foundation system and corresponds to where the safety margin is less than unity. Since the magnitude of the probability of failure is generally small and thus difficult to assess, the probability of failure can be alternatively described using a target reliability index, β_T.

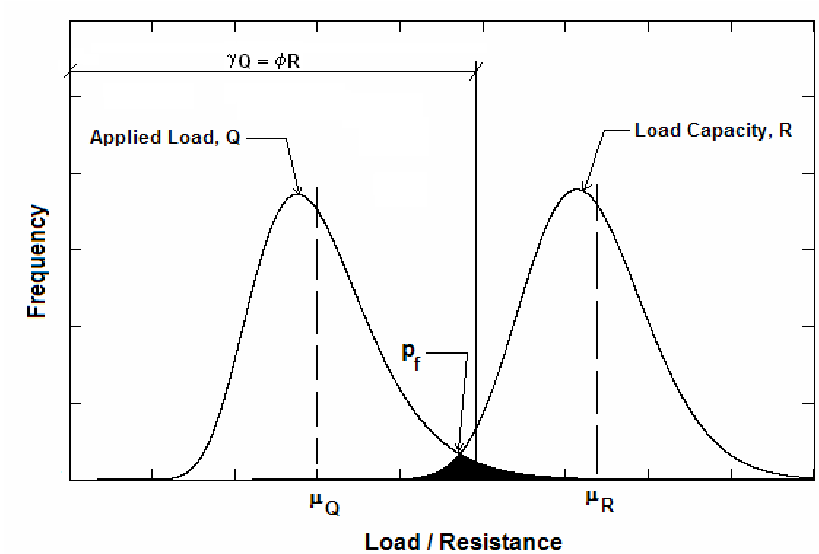

Figure 4-10: Generalized reliability-based design approach using a PDF for load and resistance.

As observed from Figure 4-11, β_T is essentially a factor that represents the number of standard deviations the value of unity is from the mean within the safety margin PDF. To that end, Table 4-7 provides the relationship between β_T and p_f, along with the expected performance level of the system for different β_T magnitudes. Typically, to ensure both a safe and efficient design, the value of β_T selected for design in geotechnical engineering ranges from 2.0 to 3.5 (Kulhawy and Phoon 2006). Therefore, a β_T value of 3.0, for example, would correspond to a probability of failure of 0.1% suggesting that 1 out of every 1000 foundations designed would fail under the given combination of design loads and resistance. The safety margin PDF, along with the selected value of the target reliability index, β_T, will form the basis of the LRFD approach and subsequently, the resistance factor calibration process.

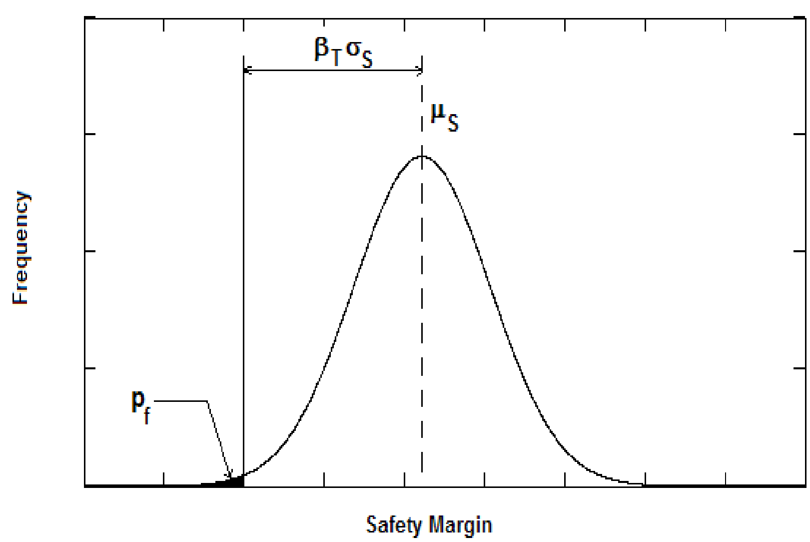

Figure 4-11: Safety margin PDF.

Table 4-7: Relationship between β_T and p_f along with expected performance (from USACE 1997).

β_T	p_f	Expected Performance
0	0.5	-
0.5	0.309	-
1.0	0.159	Hazardous
1.5	0.067	Unsatisfactory
2.0	0.023	Poor
2.5	0.006	Below average
3.0	0.001	Above average
3.5	0.0002	-
4.0	0.00003	Good
4.5	0.000003	-
5.0	0.0000003	High

Until recently, the design of geotechnical elements, including deep foundations, had been excluded from the LRFD requirements. Historically, the design of such elements had been conducted using the allowable stress design (ASD) method. The allowable stress design (ASD) method assigns a factor of safety (FS) to account for the uncertainties in the design. The ASD design equation is as follows:

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

Equation 4-10

where,

Q = the working (unfactored) design load,

Q_{all} = the allowable design load, and

Q_{ult} = the ultimate geotechnical capacity.

As stated before, when using the ASD approach, the magnitude of uncertainty is not explicitly quantified in the design and thus the factor of safety is an arbitrary value chosen by the design engineer. In general, factors of safety between 2 and 3 are considered to be sufficient in foundation design (Focht and O'Neill 1985), although values as large as 6 are reported in the literature for various design methods (Reese et al. 2006, Das 2007).

In comparison, the LRFD approach utilizes a resistance factor, with a value less than or equal to 1.0, that is applied to the calculated nominal resistance of the deep foundation, while load factors, with values greater than or equal to 1.0, are applied to the calculated loads. To that end, the following inequality must be satisfied when using the LRFD method:

$$\sum \gamma_i Q_i \le \phi_R R_n$$

Equation 4-11

where,

γ_i = a load factor,

Q_i = a nominal load applied to the system,

ϕ_R = the resistance factor, and

R_n = the nominal foundation resistance.

In general, the resistance factor and the load factors are computed using a probabilistic method that can explicitly consider the various uncertainties known to exist in the design. As stated previously, the load factors for the LRFD approach have been extensively developed by the structural engineering community for design of bridges and other structures. Therefore, it is most important to focus on the calibration of the resistance factor for the design of deep foundations. Using the safety margin PDF developed previously in Figure 4-11 and following the First Order, Second Moment (FOSM) method, an equation for ϕ_R can be developed as follows (Baecher and Christian 2003):

$$\phi_R = \frac{\lambda_R\left(\frac{\gamma_D E(Q_D)}{E(Q_L)}+\gamma_L\right)\sqrt{\frac{1+\Omega_{QD}^2+\Omega_{QL}^2}{1+\Omega_R^2}}}{\left(\lambda_{QD}\frac{E(Q_D)}{E(Q_L)}+\lambda_{QL}\right)e^{\beta_T\sqrt{ln\left[\left(1+\Omega_R^2\right)\left(1+\Omega_{QD}^2+\Omega_{QL}^2\right)\right]}}}$$

Equation 4-12

where,

λ_R = the bias of the resistance,

λ_{QD} and λ_{QL} = the bias of the dead load and live load, respectively,

γ_D and γ_L = the load factors for the dead load and live load, respectively,

Ω_{QD}, Ω_{QL}, Ω_R, = the coefficient of variation (COV) for the dead load, live load and resistance, respectively, and

$E(Q_D)$ and $E(Q_L)$ = the expected values of the dead and live load, respectively.

The COV and bias of the dead load and live load, along with the dead load and live load factors, have been extensively investigated, with typical values given in Baecher and Christian (2003) and Paikowsky et al. (2004). The ratio of the expected value of the dead load to the expected value of the live load does not significantly affect the value of the resistance factor and can be assigned a magnitude based on the characteristics of the structure, with a value typically given between 2 and 4 (Baecher and Christian 2003). The value of the target reliability index, β_T, is based on the desired performance of the system and appropriate values have been provided previously. Therefore, after assigning magnitudes to the known variables in Equation 4-12, it is observed that the only unknown parameters are the COV of the resistance, Ω_R, and the bias of the resistance, λ_R. To that end, the probabilistic resistance factor calibration methods presented in the following section will focus on determining the magnitude of λ_R and Ω_R, which ultimately form the basis of the Load and Resistance Factor Design approach.

4.5.2 RESISTANCE FACTOR CALIBRATION METHODS

The calibration of resistance factors is critical to the transition and adoption of the LRFD method from the ASD approach. The following sections will outline the basic principles of the calibration process and will discuss the methods that can be employed in calibration. In addition, these sections will provide a reference for Chapter 8 which will outline the resistance factor calibration procedure using one of the methods below in conjunction with Equation 4-12.

4.5.2.1 Principles

The design of deep foundations using the LRFD approach requires that the design is satisfied for three different limit states: (1) the Strength Limit State; (2) the Service Limit State; and (3) the Extreme Limit State. The Strength Limit State defines the strength of a system that should not be exceeded by any combination of loading during its design life. At the Strength Limit State, the system is likely to become unstable, resulting in structural damage or complete failure (i.e. collapse). The Service Limit State defines a limiting tolerable deformation of a system that, if exceeded, will impair its function. At the Service Limit State, the system will not collapse, but the system may become unusable or develop aesthetic or basic functionality problems (Roberts and Misra 2010, Budhu 2008). An example of this is the differential settlement in a bridge, which does not cause a collapse in the system, but creates cracking in the bridge deck that may lead to functionality or maintenance problems. Last, the Extreme Limit State ensures the structural survival of a system during an extreme event, such as an earthquake, flood, or impact loading from a vehicle or vessel. In many cases, the return period for extreme events may be significantly greater than the design life of the structure.

As discussed previously, the advantage of the LRFD approach over the traditional ASD method is that the load and resistance factors are determined using a probabilistic approach to ensure that design uncertainties are explicitly integrated and quantified in the design (Roberts 2008). In addition, resistance factors are calibrated to produce designs that consistently achieve a desired level of reliability (Phoon et al. 1995) based on the selected target reliability index used in Equation 4-12. The following section will discuss the various methods that are currently utilized to calibrate resistance factors for use in the LRFD approach and will highlight the common assumptions necessary to employ each of the methods.

4.5.2.2 Methods

There are a number of methods that can be employed in order to calibrate resistance factors for use in the LRFD method. The following sections will introduce three of these methods and will discuss the advantages and disadvantages of each. The first two methods can be utilized for calibration of resistance factors at the Strength Limit State only, while the third method can be utilized for calibration of resistance factors at both the Strength and Service Limit States (Roberts et al. 2008).

4.5.2.2.1 Calibration-to-Fit

The allowable stress design (ASD) "calibration-to-fit" method is a relatively simple method for computing resistance factors. The premise of the method is to calibrate resistance factors so that they are relatively consistent with previous design practice under the ASD approach (Allen et al. 2005). Since the traditional factor of safety is essentially "fit" to the resistance factor framework, the approach is often referred to as a "fitting" method. The following equation is used to accomplish this approach when two load sources are assumed (Withiam et al. 1998):

$$\phi R = \frac{\gamma_D \dfrac{DL}{LL} + \gamma_L}{FS\left(\dfrac{DL}{LL} + 1\right)}$$

Equation 4-13

where,

γ_D and γ_L = the load factors for the dead load and live load, respectively,

(DL/LL) = the dead load to live load ratio for the structure, and

FS = the historic global factor of safety.

Table 4-8 provides the value of resistance factors determined using the calibration-to-fit formula given in Equation 4-13 for various combinations of DL/LL ratios and global factors of safety. The dead load and live load factor were assumed as 1.25 and 1.75, respectively, in the computations.

Table 4-8: Resistance factors determined using the calibration-to-fit method for variable dead load to live load ratios and global factors of safety.

Global Factor of Safety	Resistance Factor, φR			
	DL/LL 1	DL/LL 2	DL/LL 3	DL/LL 4
1.0	1.50	1.42	1.38	1.35
1.5	1.00	0.94	0.92	0.90
2.0	0.75	0.71	0.69	0.68
2.5	0.60	0.57	0.55	0.54
3.0	0.50	0.47	0.46	0.45
4.0	0.38	0.35	0.34	0.34

Since the calibration-to-fit method utilizes the historic global factor of safety to calibrate the resistance factor, the method does not explicitly quantify the design uncertainty in the process (Roberts 2008). The method will result in either a global or project specific resistance factor based on the variance in the *FS* that is employed for each design. However, this approach has generally only been applied for calibration of resistance factors at the Strength Limit State and may not be appropriate for calibration at the Service Limit State.

4.5.2.2.2 Global Resistance Factors using Reliability Theory Calibration Method

Previously, the general equation utilized to compute a resistance factor based on the FOSM method was provided. Recall that Equation 4-12 incorporates the statistical principles represented by the safety margin PDF shown in Figure 4-11. The safety margin PDF explicitly allows for the inclusion of design uncertainties associated with the loads and the foundation resistance (i.e. capacity). The unknown parameters in Equation 4-12 are the COV of the resistance, Ω_R, and the bias of the resistance, λ_R. The determination of these unknown parameters using a statistical approach is commonly referred to as the *reliability theory calibration method*. The purpose of the reliability theory calibration method is to quantify the magnitude of bias and variability within the deep foundation design technique used by the design agency, which allows for direct utilization of these values in the resistance factor calibration.

The first reliability theory calibration method to be discussed in this report was extensively utilized to calibrate the resistance factors reported in NCHRP Report 507 (Paikowsky et al. 2004). In this approach, λ_R and Ω_R are determined by obtaining a large database of field load test data for a particular deep foundation system installed in a particular type of soil (e.g. drilled shafts installed in sand). The field load test data can be comprised of either static load test results or dynamic field verification results. The load-settlement curve from each field load test is analyzed, using one of several possible load test ultimate capacity criteria, in order to obtain the *actual* resistance for the deep foundation system. The standard static capacity prediction technique employed by the design agency is then utilized to compute the *predicted* resistance for the deep foundation using the nominal soil properties or other pertinent information for the site. This process is repeated for each field load test in the database. The ratio of the actual resistance to the predicted resistance is computed for each load test and the statistics of this ratio data, specifically the mean and standard deviation, are computed for the entire database. The mean of the ratio for the database is taken as λ_R and the COV of the ratio for the database is taken as Ω_R as shown in Figure 4-12. These factors are substituted into Equation 4-12 to obtain a calibrated resistance factor for the static capacity prediction technique of interest. *This method results in a single, global geotechnical resistance factor that must be applied for all designs at the Strength Limit State*. To date, the method has not been extended to the Service Limit State.

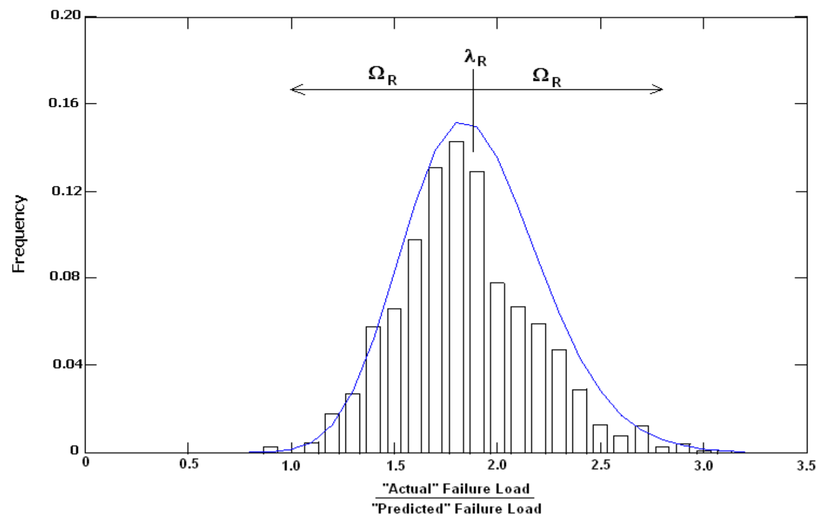

Figure 4-12: Global resistance factor calibration using the reliability theory calibration method.

The distinct advantage of this method is that it is flexible and adaptable to allow the design agency to calibrate a global geotechnical resistance factor using a local field load test database to any static capacity prediction technique. For example, the design agency can utilize local field test data along with any one of the numerous static capacity prediction techniques reported in the AASHTO LRFD Bridge Design Specifications, such as the α-method, β-method, or Nordlund method (AASHTO 2009), to obtain a calibrated global resistance factor that will better represent the typical local soil/rock and deep foundation design conditions. In addition, the design agency can utilize local field load test data along with their "in-house" static capacity prediction technique to obtain a calibrated global resistance factor that will also better represent the local conditions.

It should be noted that several assumptions are required during the resistance factor calibration process that may lead to significant variability within the magnitude of the calibrated global geotechnical resistance factor. For example, several methods exist to compute the actual resistance using field load test data. A review of the literature results in the identification of fourteen or more methods, such as the Davisson's method, FHWA method, Chin-Kondner's method, DeBeer's method and Brinch-Hansen's method (Fellenius 1990). The use of a different interpretation method for field load test data will result in a different magnitude for the actual resistance of the deep foundation and in *some* instances the difference in this magnitude has been observed to be over 50% (Yang et al. 2008). Although numerous methods also exist to compute the predicted resistance, it is often the internal assumptions made by the designer during the static capacity prediction that can lead to significant variability in this predicted resistance. For example, assumed values for the soil/rock parameters, assumptions regarding load transfer, geometry of the foundation, and variable interpretations from tables and graphs can all lead to different values for the predicted resistance at the same site. This prediction variability was highlighted in a symposium supported by the American Society of Civil Engineers in conjunction with Northwestern University (Finno 1989). The symposium involved twenty-three participates who were tasked with predicting the resistance of four pile types installed at different sites. Interestingly, the COV within the predicted resistance from the participants was discovered to be between 25% and 38% for the four sites (Finno 1989). Using this data, Roberts (2008) determined that the substantial number of possible combinations for the ratio of actual to predicted resistance resulted in a difference of over 25% within the calibrated resistance factor. It is therefore critical that the interpretation method for the field load test data, along with the assumptions that are utilized for prediction of pile resistance within the resistance factor calibration process are known and transparent to the design engineer and must be consistent with the internal procedures employed by the design agency.

4.5.2.2.3 *Site Specific Resistance Factors using Reliability Theory Calibration Method*

In recent years, several researchers have discussed the necessity of calibrating resistance factors on a site or project specific basis (Roberts et al. 2008, 2009; Kulhawy and Phoon 2009; Loehr and Huaco 2009). Since the magnitude of each of the sources of uncertainty can vary from site to site, it appears that such an approach would be beneficial in lieu of using a single global resistance factor (Roberts et al. 2008, Roberts and Misra 2010). One of the site specific resistance factor calibration methods reported in the literature uses a soil-structure interaction model, commonly referred to as the "t-z" model. In a t-z model, the resistance of the soil along the side and at the tip of a deep foundation is represented by a spring-slider system. Each spring is assumed to contain a magnitude of strength and stiffness that is directly related to the characteristics of the soil/rock at the site, deep foundation material, construction technique, etc. As the deep foundation is loaded, the springs will displace and the foundation will undergo settlement. An increase in the load will result in yielding of the springs beginning at the top of the deep foundation and progressing to the tip. At some load, all of the springs will yield and the foundation will fail by plunging. Thus, the use of a t-z model approach allows for the development of a load-settlement curve that represents the behavior of the deep foundation over a wide range of loads.

For a given site, the strength and stiffness value of the interface and tip soil springs are assumed to have a nominal magnitude, along with a defined level of uncertainty. The uncertainty in the model parameters is due to the previously identified sources and can be determined from the back-analysis of field load test data, the analysis of site investigation and laboratory testing, or the use of default uncertainty magnitudes provided in AASHTO (2009) (Roberts and Misra 2010). The nominal magnitude and uncertainty values are utilized to develop a PDF for each of the springs. Using each of the PDFs, a computer program randomly selects a strength and stiffness value for each spring and generates one complete load-settlement curve. This process is repeated in order to randomly generate of a very large set of hyperbolic load-settlement curves, as shown in Figure 4-13, that represent the uncertainty in the load-settlement behavior of a deep foundation at a given site.

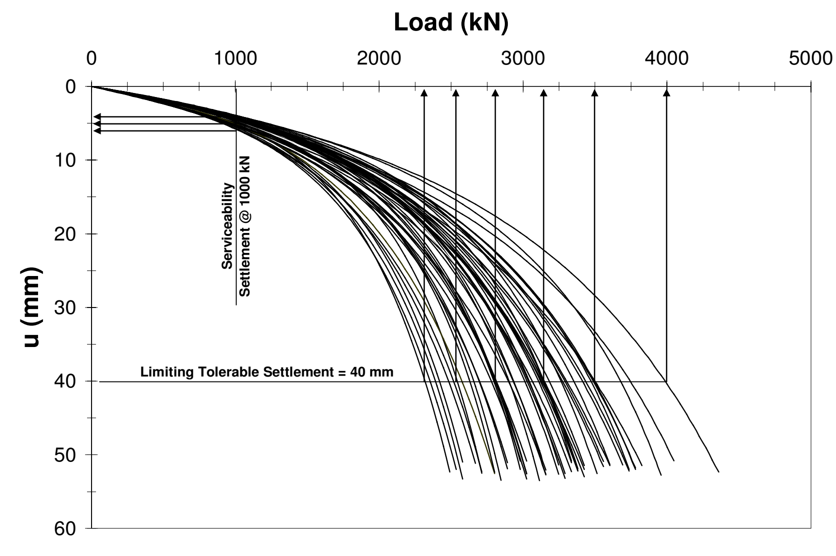

Figure 4-13: Randomly generated load-settlement curves using the t-z model method.

The randomly generated load-settlement curves are analyzed at a limiting tolerable settlement, which is the settlement where significant structural distress or collapse of the supporting structure is possible (Roberts and Misra 2010). As observed in Figure 4-13, each randomly generated load-settlement curve contains a resistance value that corresponds to the limiting tolerable settlement. Once each resistance value from the load-settlement curves is determined, the variability within the resistance values can be represented by a PDF as shown in Figure 4-14. This PDF characterizes the variability in the deep foundation resistance at the limiting tolerable settlement. The statistics of the PDF are computed and the COV of the PDF is assumed as Ω_R in Equation 4-12. Since the t-z model method can be employed to directly fit field load test data conducted at the site, the bias of the resistance, λ_R, is assumed as unity in this approach (Roberts et al. 2008, Roberts and Misra 2010). These factors are substituted into Equation 4-12 to obtain a calibrated resistance factor. This method results in a single, site specific resistance factor for design of deep foundations at the Strength Limit State. However, the method is flexible and resistance factors can be calibrated for the Service Limit State by merely analyzing the randomly developed load-settlement curves at a serviceability settlement. Therefore, by basing a deep foundation design methodology on a tolerable settlement magnitude at either the Strength or Service Limit States will ensure that a structure can functionally operate within a desired tolerance throughout its lifetime.

The distinct advantage of a site specific resistance factor calibration method is that the resistance factor can directly reflect the observed uncertainty at a site in lieu of a global magnitude of uncertainty. The use of the t-z model method is also advantageous over most static capacity prediction techniques as the approach can easily incorporate field load test data and the method properly accounts for load transfer based on the strength and stiffness characteristics of the soil-structure interface and tip soil components. This approach has been shown to lead to increased efficiency in design (Roberts et al. 2009). Furthermore, as suggested by Roberts and Misra (2010), the design can be conducted within various levels, which account for project scale, schedule, funding, and importance. These levels are designated as Level “A”, Level “B” and Level “C”. A Level “A” design is expected to yield the most efficient design and includes multiple field load tests at a site in order to characterize site uncertainty for the design. A Level “B” design requires only one or two field load tests at a site with the magnitude of uncertainty determined from extensive site investigation and laboratory testing. Level “C’, by contrast, does not require any field load tests at the site and the magnitude of uncertainty is based on upper bound magnitudes defined in AASHTO (2009), thereby resulting in the least efficient design. Each level can result in calibration at different target reliability indices as well, which can further the efficiency of the design, while ensuring safety and reliability.

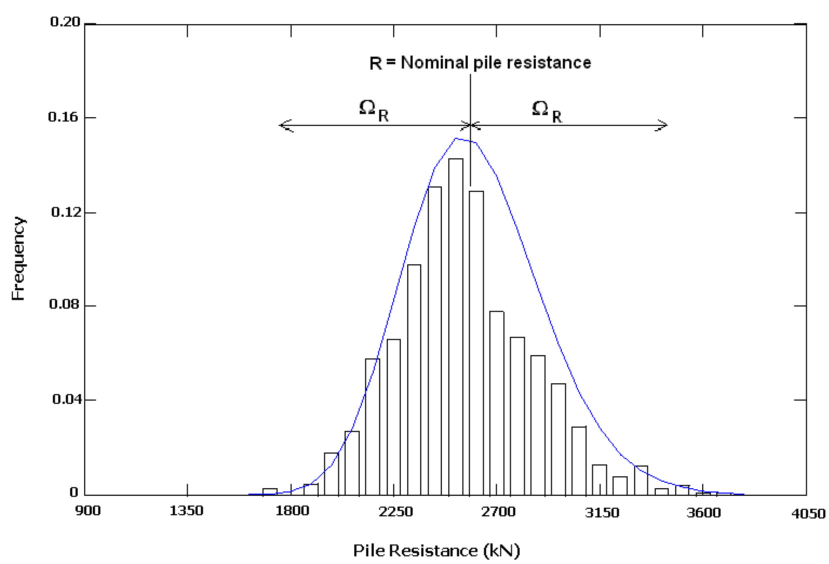

Figure 4-14: PDF for deep foundation resistance at the limiting tolerable settlement.

It should be noted that the site specific resistance factor calibration method does require greater involvement from the design engineer in the calibration process. For example, in the t-z model method presented herein, the design engineer must determine the magnitude of uncertainty within the model parameters based on the aforementioned design level and must perform the probabilistic modeling in order to determine the value of the resistance factor. The uncertainties are assumed to encompass the "within-site variability" (Zhang 2008) that are present at the design location. In addition, the probabilistic analysis requires the selection of a target reliability index, as provided in Table 4-7, for each design. However, as described in Kulhawy and Phoon (2009), the COV of the aforementioned uncertainties is not constant from site to site and thus sufficient flexibility for the design engineer in the calibration process must be allowed. To that end, Roberts et al. (2009), Roberts and Misra (2010), and Roberts et al. (2010) provide a structured and streamlined approach to apply the site specific calibration method using the t-z model method through the use of a developed software package. Finally, the use of the t-z model approach for site specific resistance factor calibration generally requires the initial development of a large load test database that is correlated to the t-z model parameters and ultimately utilized in subsequent designs. The use of developed correlations can result in bias between the t-z model predicted load-settlement performance and the actual load-settlement performance in the field. However, Park et al. (2010) demonstrated that the prediction of deep foundation performance using the t-z model method with a developed load test database does contain sufficient accuracy to achieve the desired level of reliability and consistency required within most deep foundation design projects.

4.6 SUMMARY

This chapter provided background information regarding the design of deep foundation systems based on a number of methodologies recommended by the FHWA and included in the AASHTO Specifications (AASHTO 2009). The chapter also provided information regarding the LRFD method and the calibration of resistance factors. This chapter will provide reference information for the remainder of the report.

5 CURRENT SDDOT METHODOLOGIES

5.1 INTRODUCTION

This Chapter documents the research team's understanding of the SDDOT's field and design methods for deep foundations. The SDDOT uses a field method for the design of deep foundations which is based on the use of a driving formula and field measured skin friction values. The method has been used by the state since prior to 1964 and is a unique method from driven pile and drilled shaft design used in engineering practice.

The design method employed by the SDDOT has been previously researched by projects funded by the SDDOT in the 1960s and subsequently in the 1990s. The 1960s study on piles by the SDDOT attempted to more accurately perform field verification of pile capacity, but the method of verification was never adapted. The 1990s study was conducted on small diameter drilled piles to verify the reliability of the field and design procedures of the SDDOT for determining shaft side resistance of deep foundations. The SDDOT geotechnical engineering activity is currently in the transition from the ASD-based method to the LRFD-based design method.

This chapter is based on many documents provided by the SDDOT for the research team's consideration. That list is provided in Appendix A.

5.2 SITE INVESTIGATION METHODS

The field test to evaluate site conditions is conducted by augering a hole to access the subsurface. This is followed by conducting a drive test and sampling using a Falling Drilling Unit (Turner and Sandberg 1992). Subsequent to sampling, the sampling device is retracted from the ground with the resistance to retract the sampler measured using a load cell and dynamometer, which is subsequently referred to as a line weight indicator.

The augering hole is accomplished by using a 4-1/2 inch continuous flight auger. Use of the auger provides information on major soil zones and the location of the water table. The auger is typically drilled to a depth of 20 feet or greater below the anticipated foundation depth (SDDOT 2007).

The SDDOT (2007) method outlines that a drive test and soil sampling are conducted next using the Falling Drilling Unit. The SDDOT drives a 2 7/8" diameter drill rod, known as a "PK-rod", with a 490 pound hammer to advance the rod. The PK-rod is equipped with a California retractable plug tube sampler and the hammer is dropped from a height of 30 inches. A count of blows per foot during the drive is performed and samples are taken at desired depths with the California sampler (Turner and Sandberg 1992). Driving is limited to a measured "blows per foot" (bpf) value of 300 or refusal to avoid mechanical restrictions and damage to equipment based on experience by the SDDOT.

Soil samples are collected in a disturbed manner with the use of a modified California retractable plug sampler. The sampler is driven with the 490-lb hammer and samples are obtained from the sampler with 2-inch nominal diameter brass liners that are each 4-inches long. The SDDOT is capable of collecting 16 samplers from the length of the sampler. The SDDOT retrieves the soil samples collected by the California modified sampler for subsequent laboratory testing. While the SDDOT's field sampling method is generally not considered an undisturbed sample (Sabatini et al. 2002), SDDOT performs unconfined compression tests on the collected samples, as well as soil classification. It was noted that sampling by this method is primarily used for soils classification and is not used as input into the SDDOT design methodology.

Upon the completion of the drive test, the drive head is removed and a pulling head attached, which is capable of pulling the PK-rod from the ground. The pulling system is equipped with a load cell and a line weight indicator to measure the soil-PK resistance (SDDOT 2007). The value measured by the load cell is recorded and used to determine the overall skin friction value of the soil.

5.3 CAPACITY PREDICTION

Since prior to 1964, the SDDOT has used a unique method of predicting the capacity of deep foundations. The method involves using the PK-rod as a miniature pile to simulate and model the forces on installed driven piles or the skin friction value for the drilled shafts at the test location.

The design methodology for driven piles and drilled shafts are similar, but drilled shafts are not considered to have an end bearing resistance. The drilled shaft design method does not appear to have been constant since 1964 and appears to have changed from a combination of laboratory and field drive test data to a drive test only method more closely resembling the design methodology of driven piles (Bump et al. 1971, Abu-Hejleh and Griese 2008). For both driven and drilled shafts the nominal resistance, R_n, is multiplied by the appropriate resistance factor as shown in Equation 4-11.

5.3.1 DRIVEN PILES

Bearing resistance for driven piles is determined with the combined use of drive test and pull test data. The drive test data for the pile is used to determine the pile resistance to the depth of the drive test data. The pull test (skin friction value) provides a value in relation to the penetration resistance of the soil to the PK-rod. The pull test data (skin friction value) is essentially used to supplement the drive test data to determine the supplemental capacity needed to achieve structural resistance of the pile. The values derived are related to the established baseline values for an 11-inch diameter timber pile and then modified to reflect the pile size and shape required for the project. This process allows the SDDOT to design pile depths to fully utilize the structural strength of the pile.

5.3.1.1 Drive Test Data

The basis of the SDDOT method is an adopted driving equation, similar to the modified ENR formula and the Michigan Engineering News formula (MENR). The MENR formula was originally calibrated with “long, slender, and relatively lightweight steel piling,” producing results as consistent as the ENR formula (Lowery et al. 1968). The form of the SDDOT driving equation is:

$$R_{tim} = \frac{FWH}{S+0.35} \times \frac{W}{W+M} \times Z$$

Equation 5-1

where,

R_{tim} = nominal geotechnical resistance of an 11 inch diameter timber pile at the end of driving (tons);

F = units conversion from ft-lbs to in-lbs as well as a reduction, if applied;

W = the weight of the moving components (lbs);

H = height of the hammer drop (ft);

Z = size and unit conversion factor (tons/lb);

S = the final set of the pile (in/blow); and

M = weight of the nonmoving components (lbs).

Historically the SDDOT has used an F = 3 as part of their ASD design and verification procedures. The FHWA initially assumed a factor of safety of 3.5 based on the past experience of the FHWA with relation to driving formulas, therefore converting Equation 5-1 to a nominal (ultimate) resistance for use in the LRFD approach results in an F value of 10.5. Based on the documents reviewed, it appears that SDDOT has historically used a factor of safety of 4; however, during the course of this research project, the built in factor of safety that was incorporated into Equation 5-1 could not be determined by documentation, past research studies, or interviews with SDDOT personnel. Through a literature review (Lowery et al. 1968; Reese et al. 2006), the research team ascertained that the equation used by the SDDOT likely involves a conversion from ft-lb of energy to in-lb of energy; therefore, the factor F corresponding to 3.0 likely results from a factor of safety of 4. The true nominal form of the equation incorporating a units conversion results in an F value of 12. For clarity, this form of the equation is referred to as the Ultimate Method (UM) so as to not confuse the reader with the existence of two nominal forms of the equation. Both LRFD (SDDOT current calibration-to-fit method) and UM method are included herein for clarity, the sole difference is the assumed factor of safety of the driving equation. When the factor of safety of Equation 5-1 for ASD is known during the calibration-to-fit process, the LRFD and UM forms would have the same F value. The use of the UM formula is appropriate as the calculated shaft resistance beyond the depth of driving of the PK-rod is not factored. The design which is beyond this depth uses the pullout force as the basis of its design value, which is never factored. During this portion of design ASD, LRFD, and UM are identical. The mechanism for conversion from ASD to LRFD or UM is the same, though the assumptions of the factor of safety are different. The conversion of the factor F from ASD to UM is:

$$F_{UM} = F_{ASD} \times FS = 3 \times 4 = 12$$

Equation 5-2

where,

F_{UM} = units conversion from ft-lbs to in-lbs for UM;

F_{ASD} = units conversion from ft-lbs to in-lbs and reduction due to the FS for ASD; and

FS = factor of safety used in the ASD method.

The FS = 4 in the conversion from ASD to UM is derived based on the assumption that the units must be consistent in the equation, and that the ENR basis of the equation is consistent with that which is shown in published reports (Lowery et al. 1968; Reese et al. 2006). When the ENR equation is in the same units as Equation 5-1 it includes a unit conversion from ft-lb of energy to in-lb of energy, which is equal to 12in/ft. Using the definition of the factor of safety in Equation 5-3, the value of FS = 4 was derived.

$$FS = \frac{Ultimate}{Allowable} = \frac{12}{3} = 4$$

Equation 5-3

The LRFD nominal value for F = 10.5 was determined using Equation 5-2 with an FS = 3.5 as previously mentioned. The use of this value should be carefully considered by the SDDOT as it has been shown that it is likely based on an incorrect assumption.

The drive data is used to estimate the axial capacity of a driven pile at the depth of the end of driving. The blow count for the final foot of the drive is used in a modified ENR formula which takes into consideration an end area ratio between the drill stem and an average timber pile. The parameters are entered into Equation 5-1 (SDDOT 2007).

The Z factor in Equation 5-1 is the ratio of the end area of an 11 inch diameter timber pile to the 2-7/8 inch diameter PK-rod. The factor also includes a units conversion from pounds to tons. The resulting value for Z is 0.0073 ton/lbs, but the value of 0.0075 ton/lbs is used in practice.

The design of piles, other than a timber pile, requires the use of a size factor to account for size differences. The SDDOT method modifies R_{tim} by the ratio of the end area of the design pile to the end area of a timber pile. The resistance of the design pile, R', is expressed as:

$$R' = R_{tim} \times B_r$$

Equation 5-4

where,

R' = nominal pile resistance calculated using Equation 5-1 at depth; and

B_r = size factor that represents ratio of the design pile's end area to that of the end area ratio of an 11- inch timber pile (Table 5-1).

Table 5-1: Size Factors (after SDDOT 2007)

Pile Diameter	Size Factor
8" H-pile	N/A
10" H-pile	0.8
12" H-pile	1.2
16" Pipe	1.6

The skin friction of the PK-rod beyond the depth of the drive is determined by relating the pullout force to the blow count and then determining the average skin friction over the last four feet. The weighted skin friction value for each foot is expressed as:

$$SFV_d = \frac{F}{B_t} \times \frac{B_f}{C}$$

Equation 5-5

where,

SFV_d = skin friction value for specified foot of depth (psf);

F = calibrated line weight reading (lbs);

B_t = total blow count over the drive (blows);

B_f = blow count of specified foot (blows/ft), and

C = shaft surface area of the PK-rod per foot of length (ft^2/ft).

The average skin friction value is calculated as:

$$SFV_a = \frac{\sum SFV_d}{4}$$

Equation 5-6

where, SFV_a is the average skin friction value. The unit shaft resistance is then converted to the shaft resistance per unit length by multiplying the SFV_a by the perimeter area per foot of the design pile (SDDOT 2007). This conversion is expressed as:

$$RF = SFV_a \times PA$$

Equation 5-7

where,

RF = skin friction (resistance) per unit length (plf); and

SFV_a = skin friction value (psf); and

PA = perimeter area (ft^2/ft).

RF is then converted from pounds/foot to tons/foot.

The depth beyond the PK-rod drive needed to meet the structural resistance of the pile is defined as:

$$D = \frac{R_r - R'}{RF}$$

Equation 5-8

where,

D = required depth beyond the depth at the end of driving to match the structural resistance of the pile (ft);

R_r = factored structural resistance of the pile (tons); and

R' = nominal resistance of the pile at the end of drive depth (tons).

The geotechnical resistance at a specified depth beyond the end of drive depth is expressed as:

$$R_n = R' + \left(D_s - D_{EOD}\right) \times RF$$

Equation 5-9

where,

R_n= nominal resistance of the pile at the specified depth (tons);

R' = nominal resistance of the pile at the end of drive depth (tons);

D_s =specified depth (ft);

D_{EOD} = depth at end of drive (ft); and

RF = skin friction per foot, (tons/ft).

5.3.1.2 Pull Test Data

The skin friction of the PK-rod beyond the depth of the drive is determined by relating the pullout force to the blow count and then determining the average skin friction over the last four feet. The weighted skin friction value for each foot is expressed as:

$$SFV_d = \frac{F}{B_t} \times \frac{B_f}{C}$$

Equation 5-10

where,

SFV_d = skin friction value for specified foot of depth (psf);

F = calibrated line weight reading (lbs);

B_t = total blow count over the drive (blows);

B_f= blow count of specified foot (blows/ft); and

C = shaft surface area of the PK-rod per foot of length (ft^2/ft).

The average skin friction value is calculated as:

$$SFV_a = \frac{\sum SFV_d}{4}$$

Equation 5-11

where, SFV_a is the average skin friction value. The unit shaft resistance is then converted to the shaft resistance per unit length by multiplying the SFV_a by the perimeter area per foot of the design pile. This conversion is expressed as:

$$RF = SFV_a \times PA$$

Equation 5-12

where,

RF = skin friction (resistance) per unit length (plf);

SFV_a = skin friction value (psf); and

PA = perimeter area (ft^2/ft) as shown in Table 5-2.

Table 5-2: Perimeter Area (after SDDOT 2007)

Pile Diameter	Perimeter Area
8" H-pile	3.92 ft^2/ft
10" H-pile	4.83 ft^2/ft
12" H-pile	5.82 ft^2/ft
16" Pipe	-

RF is then converted from pounds/foot to tons/foot.

The perimeter area values are used for each pile, but the category of the factor is different depending on the pile type. When using square or H-piles, the SDDOT uses the diameter as the length of a side of the pile; therefore, the surface area is an approximation of the entire surface area for each size group of H-piling. The net perimeter is the perimeter of the circumscribed area of the pile assuming the H-pile is square; therefore, this is four times the diameter (side) of the pile. This value is used for solid square piles, such as a prestressed concrete pile. The circumference for skin friction is the SDDOT's method for relating an H-pile to an 11-inch diameter timber pile. The relationship is calculated as follows:

$$CF = \frac{NP}{SA} \times C_t$$

Equation 5-13

where,

CF = circumference for skin friction (ft);

NP = net perimeter of the H-pile (ft) given in Table 5-3;

SA = surface area of the H-pile (ft); and

C_t = circumference of an 11 inch timber pile (ft).

Table 5-3: Net Perimeter (after SDDOT 2007)

Pile Diameter	Net Perimeter
8" H-pile	2.67 ft
10" H-pile	3.33 ft
12" H-pile	4.00 ft
16" Pipe	-

This value is then modified for other H-pile sizes, as the ratio of the design pile's net perimeter to a 10-inch H-pile's net perimeter. The circumference for skin friction of the design pile is therefore determined as:

$$CF_d = \frac{NP_d}{NP_t} CF_t$$

Equation 5-14

where,

CF_d = circumference for skin friction of the design pile (ft);

NP_d = net perimeter of the design pile (ft);

NP_t = net perimeter of the 10 inch H-pile (ft); and

CF_t = circumference for skin friction of the 10 inch H-pile (ft) given in Table 5-4.

Table 5-4: Circumference for Skin Friction Values (after SDDOT 2007)

Pile Diameter	Circumference for Skin Friction
8"	1.6 ft
10"	2.0 ft
12"	2.4 ft
16" Pipe	4.2 ft

The depth beyond the PK-rod drive needed to meet the structural resistance of the pile is defined in Equation 5-8.

The design structural strength for a 12-inch square concrete pile as allowed by the SDDOT is 110 tons (SDDOT 2007). The geotechnical resistance at a specified depth beyond the end of drive depth is given in Equation 5-9. The SDDOT (2007) assumes that timber piles have an average diameter of 11-inches and uses an allowable bearing of 32 tons on timber piles.

5.3.1.3 Example Design

This section provides an example calculation showing the SDDOT's method for predicting driven pile capacity. The method is outlined by including calculations from data that was collected at the Cretex Plant Site, Highway 79, Rapid City. A plan view of the site investigation conducted during November 1998, as well as the location of the SDDOT investigation, is provided in Figure 5-1. The test piles were installed during April 1999. These calculations are based on using boring R2 to design a 12-inch square concrete pile which has a permissible structural resistance of 110 tons SDDOT (2007). The location of boring R2 is shown in Figure 5-1.

The site, which was initially investigated in 1998 by a private geotechnical engineering consultant, was also investigated by the SDDOT in 2010. The SDDOT conducted the field data collection for Boring R2 29 feet west and 33 feet south of the south center conveyor belt footing shown in Figure 5-1. The conveyor belt has three pairs of north/south footings located at approximately the quarter points of the belt span. The ground elevation of the boring was 3209.56 ft. No ground water table was encountered.

Detailed field data collected by the SDDOT on May 3, 2010 are shown in Table 5-5 and Table 5-6. The SDDOT data collected from the PK-rod drive is shown in Table 5-6. The calibrated line weight reading is calculated by the SDDOT to account for error in the line weight indicator; this was completed by using previous calibration procedures by the SDDOT. The total blow count is calculated by summing the blow count for each foot, as shown in Table 5-6 over the entire depth of the drive.

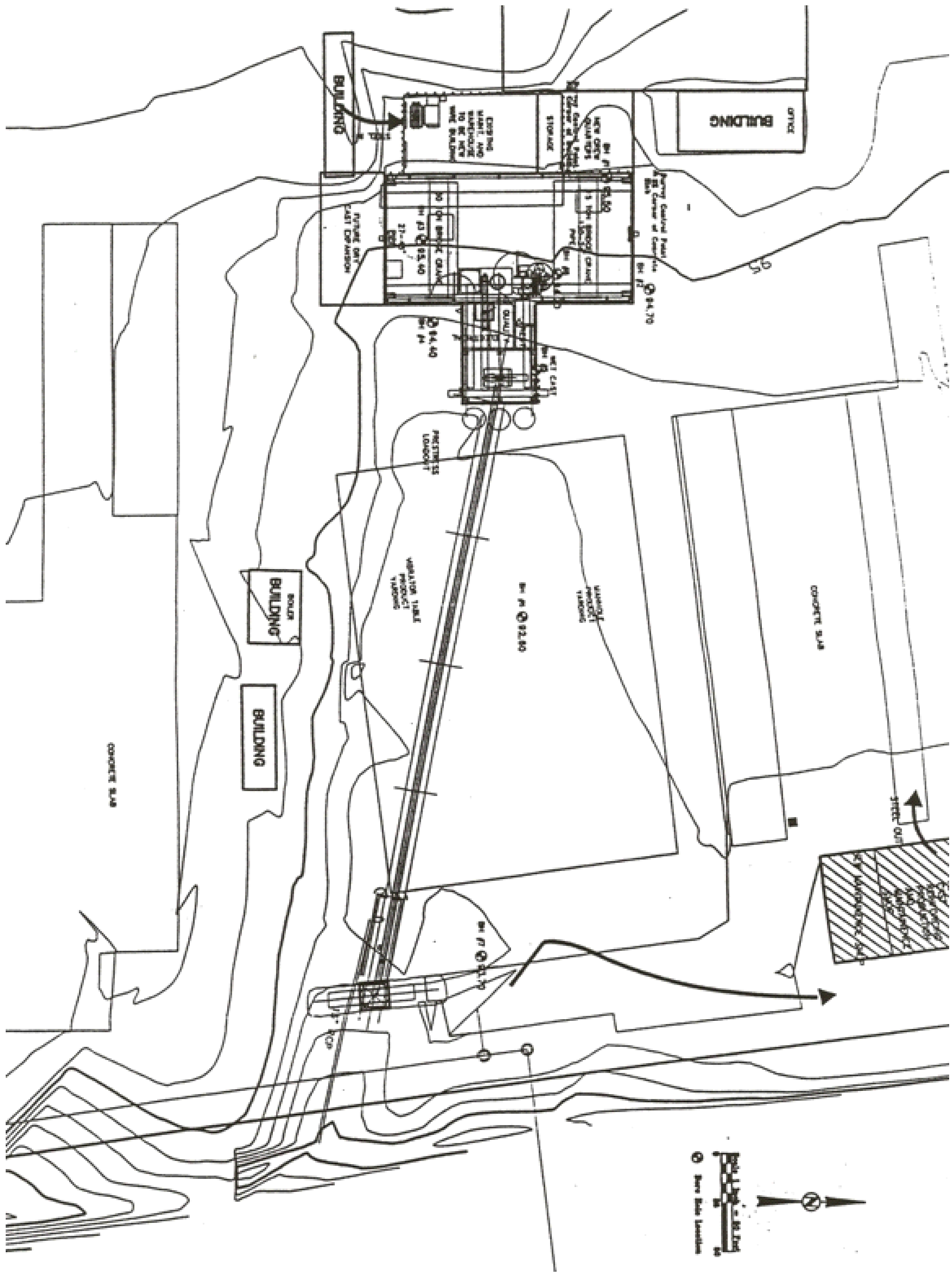

Figure 5-1: Cretex Site Plan View (Painter 1999).

Table 5-5: PK-rod Measurements

Measured line weight indicator reading	45,000 lbs
Calibrated line weight indicator reading	45617 lbs
Total blow count	1664 blows
Final Depth	35.92 ft. below ground surface

Table 5-6: PK-rod Drive Record

Depth (ft)	Blow Count (blows/ft)	Depth (ft)	Blow Count (blows/ft)
1	0	19	15
2	18	20	18
3	40	21	18
4	27	22	20
5	16	23	15
6	16	24	17
7	18	25	23
8	20	26	29
9	20	27	37
10	14	28	44
11	17	29	52
12	18	30	68
13	18	31	86
14	15	32	99
15	15	33	114
16	23	34	158
17	22	35	218
18	16	35.92	300

The data in Table 5-6 yields a final set of 0.04 in/blow by converting the blow count for the final foot to in/blow. The weight of the drive system components is summarized in Table 5-7.

Table 5-7: Weight of Nonmoving Driving Components

Moving Component(s)	Weight (lbs)	Nonmoving Components	Weight (lbs)
Hammer	490	(3) 10' PK-rods at 106 lbs each	318
		Anvil	122
		California sampler	103
Total	490	Total	543

Dynamic testing had been previously performed on three test piles using the PDA with CAPWAP at the site. Pile TP2 was tested both at the end of driving (EOD) and the beginning of restrike (BOR) 18 hours later. The resistance predicted by the PDA with CAPWAP is shown in Table 5-8. TP2 is the pile which was chosen for comparison to the SDDOT method and is the pile the PK-rod drive test was performed to investigate. The term resistance herein refers to the geotechnical resistance.

Table 5-8: Summary of Dynamic Testing Results (Modified after Painter 1999)

Pile Name	Approx. Depth Below Grade (ft)	Case Method Soil Resistance[1] (kips)	CAPWAP Soil Resistance (kips)	Time of Testing
TP2	41.0	370	340	EOD
TP2	41.0	455	470	BOR

1. Case Method Soil Resistance Calculated by Case Method.

5.3.1.3.1 Predicted Resistance

The resistance of a 12-inch square concrete pile using ASD is determined in this section using the data provided in Table 5-5 through Table 5-7. The permissible structural resistance for a 12-inch square concrete pile determined by the SDDOT is 110 tons (SDDOT 2007).

The nominal resistance at the end of driving depth of the PK-rod for an equivalent 11-inch timber pile, using Equation 5-1 is:

$$R_{tim} = \frac{3 \times 490 lbs \times 2{,}5 ft}{\frac{0.04 in}{blow} + 0.35} \times \frac{490 lbs}{490 lbs + 543 lbs} \times \frac{0.0075 tons}{lb} = 34 tons$$

The nominal resistance at the end of the PK-rod drive for a 12-inch square concrete pile using Equation 5-4 while using a value of B_f = 1.2 from Table 5-1 is:

$$R_n = 33.5 tons \times 1.2 = 40 tons$$

The value of B_f = 1.2 in this case was taken as the end-area-ratio of the design pile to the 11-inch round timber pile using SDDOT methods presented in Section 5.3.1.1.

The skin friction value of the PK-rod for each of the last four feet of driving is determined using Equation 5-10 is:

$$SFV'_{33} = \frac{45617 lbs}{1664 blows} \times \frac{\frac{123 blows}{ft}}{0.75 \frac{ft^2}{ft}} = 4170 psf$$

$$SFV'_{34} = \frac{45617 lbs}{1664 blows} \times \frac{\frac{155 blows}{ft}}{0.75 \frac{ft^2}{ft}} = 5780 psf$$

$$SFV'_{35} = \frac{45617 lbs}{1664 blows} \times \frac{\frac{218 blows}{ft}}{0.75 \frac{ft^2}{ft}} = 7970 psf$$

$$SFV'_{36} = \frac{45617 lbs}{1664 blows} \times \frac{\frac{300 blows}{ft}}{0.75 \frac{ft^2}{ft}} = 11000 psf$$

The average skin friction over the last four feet of the drive is then calculated using Equation 5-11:

$$SFV_a = \frac{4170 psf + 5780 psf + 7970 psf + 11000 psf}{4} = 7230 psf$$

The skin friction per foot of the design pile, using Equation 5-12 is:

$$RF = \frac{\left(7230\,psf \times 4.0\,\frac{ft^2}{ft}\right)}{\frac{2000lbs}{ton}} = \frac{14tons}{ft}$$

The required depth beyond the end of drive to achieve the permissible structural resistance of the design pile using Equation 5-5 is:

$$D = \frac{110tons - 40tons}{14\,\frac{tons}{ft}} = 5\,ft$$

Summing D = 5 feet and the depth of the PK-rod investigation of 36 feet results in a design pile depth of 41 feet. To verify the permissible structural resistance is matched by the geotechnical resistance using the current ASD approach for the design pile at a depth of 41 feet below the ground surface, Equation 5-9 is used and results in:

$$R_n = 40tons + (41\,ft - 36\,ft) \times 14.44\,\frac{tons}{ft} = 110tons$$

The capacity of the pile using the Ultimate Method (UM) approach is calculated using Equation 5-1, Equation 5-4, and Equation 5-9 through Equation 5-12 results in:

$$R_{tim} = \frac{12 \times 490lbs \times 2{,}5\,ft}{\frac{0.04in}{blow} + 0.35} \times \frac{490lbs}{490lbs + 543lbs} \times \frac{0.0075tons}{lb} = 130tons$$

$$R' = 130tons \times 1.2 = 160tons$$

$$SFV'_{33} = \frac{45617lbs}{1664blows} \times \frac{\frac{123blows}{ft}}{0.75\,\frac{ft^2}{ft}} = 4170\,psf$$

$$SFV'_{34} = \frac{45617lbs}{1664blows} \times \frac{\frac{155blows}{ft}}{0.75\,\frac{ft^2}{ft}} = 5780\,psf$$

$$SFV'_{35} = \frac{45617lbs}{1664blows} \times \frac{\frac{218blows}{ft}}{0.75\,\frac{ft^2}{ft}} = 7970\,psf$$

$$SFV'_{36} = \frac{45617lbs}{1664blows} \times \frac{\frac{300blows}{ft}}{0.75\,\frac{ft^2}{ft}} = 11000\,psf$$

$$SFV_a = \frac{4170\,psf + 5780\,psf + 7970\,psf + 11000\,psf}{4} = 7230\,psf$$

$$RF = \frac{\left(7230\,psf \times 4.0\,\frac{ft^2}{ft}\right)}{\frac{2000\,lbs}{ton}} = \frac{14\,tons}{ft}$$

The UM value of *R'* is greater than the pile design strength, therefore the calculation of *D* is not applicable. A depth beyond the PK-rod drive depth of 5 feet is used to show consistency in the calculation with the example ASD design. Therefore, using Equation 5-9, R_n is:

$$R_n = 160\,tons + (41\,ft - 36\,ft) \times 14.44\,\frac{tons}{ft} = 230\,tons$$

In design, the value R_n is multiplied by the appropriate reliability based resistance factor as determined by LRFD methods. The values for LRFD and UM are summarized in Table 5-9. A comparison of the resistance of ASD, LRFD, and UM at the depth of the tested installed pile is located in Table 5-10. The reader should note that using LRFD and UM, the pile is already beyond capacity at the end of the driving depth of the PK-rod, therefore a value for *D* is not applicable.

Table 5-9: Comparison of Design Values for LRFD and UM

Value	LRFD	UM
R_{tim} (tons)	120	130
R' (tons)	140	160
$SFV_{33'}$ (psf)	4167	4167
$SFV_{34'}$ (psf)	5776	5776
$SFV_{35'}$ (psf)	7968	7968
$SFV_{36'}$ (psf)	10966	10966
SFV_a (psf)	7219	7219
RF (tons/ft)	14.44	14.44
D (ft)	Not Applicable	Not Applicable
R_n at a depth of 41 feet below the ground surface (tons)	210	230

Table 5-10: Comparison of Resistance at the Depth of the Design Pile to CAPWAP Results for TP2

Value	ASD	LRFD	UM	CAPWAP EOD	CAPWAP BOR
R_n at a depth of 41 feet below the ground surface (tons)	110	210	230	170	235

The large difference in the ASD resistance between the other presented resistances shown in Table 5-10 is because the ASD value includes an unknown factor of safety. Given the SDDOT ASD method has an unknown factor of safety that is built into portions of the method, it cannot be systematically removed for the purposes of comparison.

5.3.2 DRILLED SHAFTS

The design of drilled shafts by field testing is limited solely to the weighted average skin friction over the last four feet of the PK-rod investigation. The resulting value is then applied to the depth below that which was measured. Drilled shafts are used typically in areas where the shale is within 20 ft of the ground surface and use a permanent casing placed approximately two feet into the shale. The skin friction is applied beginning at a depth which is approximately one foot below the tip of the casing (Vockrodt and Jones 2010).

The skin friction of the PK-rod beyond the depth of the drive is determined by relating the pullout force to the blow count of the PK-rod investigation and then determining the average skin friction over the last four feet. The weighted skin friction value for each foot is expressed in Equation 5-10. The average skin friction value is calculated using Equation 5-11.

The SDDOT has developed presumptive skin friction values for the various rock geomaterials, shown in Table 5-11, which provides the basis for allowable skin friction values in shales. A factor of safety of 2 to 3.5 is typically applied to the SFV_a value (SDDOT 2007). The factor of safety is varied so as to match the allowable skin friction to the typical values in Table 5-11.

Table 5-11: Typical Shale Bedrock Skin Friction Values (SDDOT 2007)

Geomaterial	Typical Shale Bedrock Skin Friction Values, SFV_t (psf)
Pierre Shale	2,000
Graneros Group	2,250
Niobrara Chalk	2,500
Greenhorn Formation	2,000
Hell Creek Formation	2,000

The design factor of safety is calculated as:

$$FS = \frac{SFV_a}{SFV_t}$$

Equation 5-15

where,

FS = design factor of safety;

SFV_t = typical allowable skin friction value for shale bedrock.

The allowable skin friction value, SFV_{all}, is calculated as:

$$SFV_{all} = SFV_t \;= \frac{SFV_a}{FS}$$

Equation 5-16

The unit allowable skin friction is then converted to the skin friction per unit length by multiplying the SFV_a by the perimeter area per foot of the design pile. This conversion is expressed as

$$RF = SFV_{all} \times PA$$

Equation 5-17

where,

RF = skin friction (resistance) per unit length (plf);

PA = perimeter area (ft^2/ft);

RF is then converted from lbs/ft to tons/ft.

The depth beyond the PK-rod drive needed to meet the structural resistance of the shaft is determined using:

$$D = \frac{R_r}{RF}$$

Equation 5-18

where,

D = required depth beyond the PK-rod investigation depth to match the structural resistance of the pile (ft);

R_r = factored structural resistance of the pile (tons).

The design structural strength for a 13.5-inch diameter round concrete shaft is approximately 110 tons, as the area is within a square inch of the area of the 12-inch square concrete shaft (SDDOT, 2007). Based on previous AASHTO code, an allowable compressive stress limited to 40% of the compressive strength of the concrete, f'_c, and f'_c = 4000 psi, results in a strength of 114 tons for both a 12-inch square concrete shaft and a 13.5-inch round concrete shaft. The SDDOT has used the previously mentioned design strength of 110 tons for 12-inch square concrete piles, which will be used for this example design.

The nominal geotechnical resistance at a specified depth beyond the end of drive depth is expressed as:

$$R_n = \left(D_s - D_{EOD}\right) \times RF$$

Equation 5-19

where,

R_n = nominal resistance of the pile at the specified depth (tons);

D_s = specified depth (ft);

D_{EOD} = depth at end of drive (ft);

RF = skin friction per foot, (tons/ft).

The UM is applied in this method by using the SFV_a for skin friction rather than SFV_{all}.

5.3.2.1 Design Example

Using the data provided in Table 5-5 through Table 5-6, the resistance of a 13.5-inch round concrete shaft using ASD is determined below. A 13.5-inch round shaft was chosen as a similar size to that of the driven pile from the previous example; this allows a comparison of the driven pile capacities which are recorded in Table 5-8. The site and investigation material are the same, which were provided for the example in Section 5.3.1.3.

The skin friction value of the PK-rod for each of the last four feet of driving is determined using Equation 5-10 is:

$$SFV'_{33} = \frac{45617 lbs}{1664 blows} \times \frac{\frac{123 blows}{ft}}{0.75 \frac{ft^2}{ft}} = 4170 psf$$

$$SFV'_{34} = \frac{45617 lbs}{1664 blows} \times \frac{\frac{155 blows}{ft}}{0.75 \frac{ft^2}{ft}} = 5780 psf$$

$$SFV'_{35} = \frac{45617 lbs}{1664 blows} \times \frac{\frac{218 blows}{ft}}{0.75 \frac{ft^2}{ft}} = 7970 psf$$

$$SFV'_{36} = \frac{45617 lbs}{1664 blows} \times \frac{\frac{300 blows}{ft}}{0.75 \frac{ft^2}{ft}} = 11000 psf$$

The average skin friction of over the last four feet of the drive is then calculated using Equation 5-11:

$$SFV_a = \frac{4170 psf + 5780 psf + 7970 psf + 11000 psf}{4} = 7230 psf$$

The factor of safety to match the presumptive value for the Niobrara Chalk (Table 5-11) of having a value of 2500 psf is calculated using Equation 5-15 and is:

$$FS = \frac{7230 psf}{2500 psf} = 2.9$$

The allowable measured skin friction value is calculated using the *FS* determined from Equation 5-16 and applied in Equation 5-13, is:

$$SFV_{all} = \frac{7230 psf}{2.9} = 2500 psf$$

The skin friction per foot of the design pile, using Equation 5-17, is:

$$RF = \frac{\left(2500 psf \times 3.5 \frac{ft^2}{ft}\right)}{\frac{2000 lbs}{ton}} = 4.4 \frac{tons}{ft}$$

The required depth beyond the end of drive to match the structural resistance of the design pile using Equation 5-18 is:

$$D = \frac{110 tons}{4.4 \frac{tons}{ft}} = 25 ft$$

The geotechnical resistance for the design pile at a depth of 41 feet below the ground surface using Equation 5-19 is:

$$R_n = (41 ft - 36 ft) \times 4.4 \frac{tons}{ft} = 22 tons$$

Based on prior discussion, the upper portion of the shaft resistance does not typically develop due to the presence of casing that is used to install the drilled shaft. In this example, the entire depth is mostly shale and a shorter length of casing would likely be used. However, for consistency with the above discussion, it was assumed in this example that the shaft was cased over a 36-foot length.

The resulting capacity using the method is shown in Table 5-12; note that the CAPWAP results are for the driven piles of a comparable size to this drilled shaft and are located on the design site. While the CAPWAP results are for driven piles, they are shown for comparative purposes.

Table 5-12: Comparison of Resistance at the Depth of the Design Pile to CAPWAP Results for TP2

Value	ASD	UM	CAPWAP EOD	CAPWAP BOR
R_n at a depth of 41 feet below the ground surface (tons)	22	63	170	235

The large differences in resistance are again due to an unknown factor of safety in the ASD value, and the fact that the upper portion of shaft resistance is neglected in computing the ASD and UM value consistent with SDDOT design methodology.

5.4 MISCELLANEOUS DESIGN

Note that the focus of this research project was the strength limit state. However, during the course of discovery for this research project, the following was noted and is deemed appropriate to include in this report:

- As previously stated, the SDDOT driving equation is a modified ENR formula (SDENR), similar to the Michigan Engineering News formula (MENR) and takes the form of Equation 5-1 without the factor *Z*. The SDDOT has historically applied the SDENR formula with an unknown built in factor of safety. In the transition to LRFD, a factor of safety of 3.5 was assumed by the FHWA during the complimentary review, as is considered standard for driving formulas by the FHWA (Abu-Hejleh and Griese 2008). The FHWA noted that the SDDOT should consider using more reliable forms of a driving formula other than the ENR formula.
- The serviceability limit state is not addressed as part of the SDDOT design. As stated in AASHTO (2009), experience has shown that bridges do experience movement and/or rotation.
- Documents provided to the research team indicate that the SDDOT is currently assuming the yield strength of H-pile steel as 36 ksi instead of 50 ksi. The higher yield strength of steel has become an industry standard.
- The strength and serviceability limit states of lateral pile performance is not addressed as part of SDDOT design. Given that pile foundation systems are subjected to horizontal loads due to wind or ice, traffic loads, bridge curvature, and vessel or traffic impact, lateral pile performance should be evaluated in the design (AASHTO 2009).
- Pile driveability that assesses pile stresses, hammer compatibility, etc. is not conducted as part of SDDOT's current design practice. AASHTO (2009) recommends that establishment of the installation criteria of driven piles include a driveability analysis. A driveability analysis is useful in determining anticipated driving stresses during construction to avoid damage to pile sections. As noted in AASHTO (2009), this can be conducted during the design phase of a project or specified in the project specifications for the contractor to perform.
- It is noted that the SDDOT is using a driven sampling method in the field to obtain disturbed samples with liners in the California modified sampler. As discussed in Section 4.4.1.2, FHWA concurs that this type of sampling is disturbed and not recommended for the basis of testing that requires undisturbed sampling. The SDDOT stated they typically use these samples for general classification only. However, use of this sampling method for the intents of testing undisturbed soil specimens could significantly bias the testing results, particularly unconfined compression testing, consolidation testing, and strength testing.

5.5 LRFD TRANSITION APPROACH

The SDDOT approach to LRFD of deep foundations to date has been focused towards the axial strength limit state. The use of LRFD directly from the AASHTO Bridge Design Specifications (AASHTO 2009) is not possible due to the SDDOT's use of their unique field data collection and design procedures. The SDDOT design procedure is based on two field exploration test methods which are incompatible with test methods used in AASHTO design methodologies. It is further complicated by the fact that this research project was not provided documentation that completely supports the method or documents the basis of the process or parameters. The SDDOT approached the FHWA Research Center in 2007 in an attempt to receive advice on the transition. The FHWA provided the SDDOT with a courtesy review of the SDDOT design and construction specifications for driven piles. This review resulted in multiple recommendations for the transition to LRFD, as well as an initial calibration of resistance factors. This section documents that understanding and the use of transition resistance factors using the calibration-to-fit method.

5.5.1 FHWA REVIEW

FHWA communication with the SDDOT began by email on July 5, 2007 with the FHWA providing recommendations for SDDOT design and construction specifications for driven piles (Abu-Hejleh and Griese 2008). The FHWA noted that the design method which the SDDOT uses is unique and may provide high quality results due to its use of a test that is similar to driving a pile. However, similar to this project, no documentation was provided that completely supports the method or documents the basis of the process or parameters.

The FHWA's recommendations included the use of higher steel grades in design and a wave equation analysis to analyze driveability and create bearing graphs. The FHWA also recommended transitioning to a more accurate driving formula. The recommendation included the use of static or dynamic load testing and the use of the PDA to provide more accurate and cost-effective field verification of the pile capacity. FHWA also recommended the SDDOT sponsor a research study to assess the SDDOT design procedure and field verification method and identify improved methods. The transition to LRFD was to use reliability-based statistical analysis of local load test data, if available, or calibration-to-fit.

5.5.2 CALIBRATION-TO-FIT

Based on information provided by the SDDOT, most load tests for bridges were performed from 1960 to 1970, with a few tests from 1970 to present (Bump et al. 1971, Turner and Sandberg 1992). Based on documentation provided (Abu-Hejleh and Griese 2008), an acceptable transition method for load factor calibration by the FHWA is a calibration-to-fit method. The FHWA (Abu-Hejleh and Griese 2008) recommended the comparison of the SDDOT modified ENR formula in the ultimate form with the Gates formula as a field verification method. The use of higher grade steel, which is more appropriate for H-piles, was discussed in relation to the strength of a pile for calibration purposes which would produce nearly identical design results as ASD.

The calibration-to-fit method includes the assumption of a dead load to live load ratio (DL/LL) and the inclusion of dead load and live load factors since the LRFD process requires factored loads, while ASD does not. The assumed DL/LL ratio was 2, although the FHWA recommended the SDDOT structural group develop the DL/LL most commonly used for SDDOT bridge projects.

Given the SDDOT uses a factor of safety of 4 for the allowable design stress of piles, and assuming a dead load factor equal to 1.25 and a live load factor equal to 1.75, the calibration of a resistance factor for piles was performed with Equation 5-20, providing a resistance factor of 0.35.

$$\phi_R = \frac{\gamma_D \frac{DL}{LL} + \gamma_L}{FS\left(\frac{DL}{LL} + 1\right)}$$

Equation 5-20

where:

ϕ_R = resistance factor for LRFD design;

DL/LL = dead load to live load ratio

γ_D and γ_L = the load factors for the dead load and live load, respectively; and

FS = factor of safety used in ASD design.

The modified ENR formula the SDDOT uses for design was modified using the same method. The FHWA noted that the method used by the SDDOT provided an estimated allowable bearing capacity but with an unknown embedded factor of safety. Again, an embedded factor of safety of 3.5 was assumed for the driving formula (Abu-Hejleh and Griese 2008). Using Equation 5-20 provided a resistance factor of 0.40.

The maximum applied nominal geotechnical resistance to a pile was determined by FHWA as 87.5% of the nominal structural strength of the pile when the structural design strength is equal to the design geotechnical resistance. This allows the use of the nominal form of the driving formula to determine the maximum penetration resistance that may be applied to the pile during field verification. The value for the maximum penetration resistance which may be applied to the pile is 87.5% of the nominal structural strength of the pile.

5.6 Past Studies

The SDDOT has commissioned two previous studies on investigating the capacities of deep foundations. The initial study, which began in 1965, was completed by SDDOT personnel and considered the field verification methods used as well as the preliminary design method of the SDDOT drilled shaft formula.

The second study was performed by the University of Wyoming and began in 1991. The study compared the use of the SDDOT field test to standard of practice static analysis methods. It provided preliminary results, but a large percentage of the test piles used were compromised by defects during pile installation.

Both studies only considered the pullout test performed on the PK-rod. The use of the SDDOT driving formula and weighted skin friction in relation to blow counts as currently used for design was not considered.

5.6.1 SDDOT Study of Determination of Pile Bearing Capacities

The SDDOT completed a study in 1971 which was intended to determine if a more accurate method of pile capacity prediction was available in relation to the South Dakota Pile formula (Equation 5-1). The research study concluded that a compelling reason to continue using the current formula was that "the bridge inspectors are familiar with the formula, and that it normally has a large factor of safety," (Bump et al. 1971). The scope of the project shifted after it was determined that the study was, to a large extent, duplicating the work of Hasel (1965). The project was revised to consider a new approach to the dynamic analysis methods. This new approach was performed in parallel with the two aforementioned studies (Bump et al. 1971), also funded by the FHWA for dynamic methods.

The consideration of a new drilled shaft method was also performed in relation to the SDDOT drilled shaft formula of the time which was described by Bump et al. (1971) with Equation 5-21. The pull test results were solely based on the pullout value and area of the PK-rod. This particular design method is no longer used by the SDDOT to determine drilled shaft capacities.

$$B = 0.5 \times (S.F.B. + E.B)$$

Equation 5-21

where,

S.F.B. = skin friction with the unit value determined by the pull test; and

E.B. = safe load end bearing with a unit value of 1.5 times the unconfined compressive strength of the soil.

The dynamic method the SDDOT analyzed considered the pile as a rigid body in relation to the soil and that dynamic and static equilibrium must be satisfied. The study analyzed the results, both neglecting and including the inertial effects of the pile, finding that the inclusion of the inertial effects improved the accuracy of the method. The method presented was quite similar to the wave equation analysis presented in Section 4.4.5 of this report in that the soil resistances are separated into static and dynamic resistances. The method considered not only the forces applied to the pile, but the time duration in which the force from the hammer acted upon the pile head, thus used force-time and displacement-time data to derive a bearing capacity.

The study produced results which showed that the use of the SDDOT dynamic analysis method was more accurate than Equation 5-1 and that Equation 5-1 could over predict the capacity of the pile. The formula developed in the research project already had a factor of safety of 4 applied to it. At the time of the study, the use of drilled shafts was relatively new in South Dakota; therefore, part of the study was to determine the economic value of drilled shafts in relation to driven piles. The study found that drilled shafts provided bearing capacity per cost values as good as or better than driven piles. The SDDOT drilled shaft formula used appeared to produce valid preliminary data in comparison to the other methods studied and produces conservative results.

The study's main conclusions were as follows:

- Verifying the accuracy of the SDDOT's method for predicting pile capacity was inconclusive. Calculated load bearing capacity compared to load test results varied from 1 to 5 and in some cases the calculated capacity was less than the field verified capacity.
- Time dependence of capacity was observed in the testing. This is in agreement with current understanding of pile capacity. In some cases, capacity increases with time, in other cases capacity decreased.
- The dynamic testing was promising. This type of testing has developed over the last four decades with the most implemented method being the PDA.
- There is a significant advantage to knowing the true factor of safety in the SDDOT's method for pile capacity prediction.

The study's main recommendation was the following:

- Significant additional field testing should be conducted to verify the SDDOT's method for estimating pile capacity.

5.6.2 UNIVERSITY OF WYOMING PREDICTION OF SKIN FRICTION FOR PILE DESIGN

A study by Turner and Sandberg (1992) attempted to compare the SDDOT pullout test to current standard of practice static analysis methods for drained and undrained loading. The authors estimated that the pull test modeled drained behavior in cohesionless soils and undrained in cohesive soils. The authors also estimated that the use of the pull test needed adjustment for interface material and installation effects in sand while used directly in clays, thus making the use of the test difficult in soil conditions which are not uniform.

The pull test demonstrated moderate to relatively high variability for the calculation of average side resistances. The results reported side resistances which were corrected for interface friction and installation method. The authors considered the PK-rod as a large displacement driven pile and thus modified the coefficient of horizontal soil stress to a value 1.25 greater than the at-rest condition. The values for one tested site in the study showed corrected side resistance values of 145 psf, 269 psf, and 668 psf, showing the large variability of the test.

Small diameter drilled piles were modeled as shafts and ranged from six to seven inches in diameter and 12 to 18 feet in depth. One pile at each of the sites monitored was instrumented with strain gages at the quarter and end points of the pile depth. The piles were reasonably defined to fail at displacements of 0.5-inches of uplift loading.

The study indicated that the pull test values for each site, over an average of two to three tests per site, were within 25% of the measured value. The data showed considerable scatter with no consistent relationship between the pull test and measured unit side resistances. The values were somewhat more in agreement when modified, but still showed a poor relationship between the measured values and the pull test values.

Due to the presence of voids in a number of the test shafts, the study was unable to determine the reliability of the pullout test with confidence. Empirical best fit relationships for the Spearfish formation and Pierre Shale were determined. The study recommended the use of standard of practice methods and full scale testing of drilled shafts to supplement the SDDOT's method of pile prediction.

The study's main conclusions consisted of the following:

- The pull test currently used by the SDDOT appears to be moderately reliable, given the limited amount of test data. However, considering defective piles in the study, reliability was decreased.
- The SDDOT's current method offers several advantages over other predictive methods and the study qualified its continued use.

The study recommended the following:

- The SDDOT's method is dated and advances in foundation engineering in the last few decades have substantially contributed to pile foundation design.
- Significant additional field testing should be conducted to verify the SDDOT's method for estimating pile capacity.
- The SDDOT should consider full scale testing of drilled shafts in conjunction with the pull out test.

5.7 FHWA REQUIREMENTS

The transition from ASD to LRFD was mandated by the FHWA in a series of two proposals in accordance with recommendations provided by AASHTO. This chapter documents the requirements promulgated by the FHWA and recommended by AASHTO in the form of the Bridge Design Specifications. The goal of the shift to LRFD is to incorporate the aspects of a reliability-based design into bridge foundations, which is fundamental to LRFD.

The transition to the LRFD platform has been an incremental process. The initial step for most states has been calibration of resistance factors by calibration-to-fit, which was previously discussed as essentially a conversion from ASD factors of safety to LRFD resistance factors. The ultimate goal of implementing LRFD is the probabilistic calibration of resistance factor using state collected data that incorporates regional site and soil conditions into the calibration process.

5.7.1 FHWA LRFD REQUIREMENTS

The FHWA issued two memorandums to state officials describing the planned transition to LRFD. The initial memorandum was sent on June 28, 2000 and established a timeline under which states were to transition to the LRFD design process. The second memorandum was sent on January 22, 2007 to clarify the original requirements outlined in the June 28, 2000 memorandum.

The first memorandum (Densmore 2000) agreed with the AASHTO recommended time schedule for the LRFD transition. It stated that new bridges with preliminary engineering beginning after October 1, 2007 would require the use of LRFD specifications. States which would be unable to meet the timeline were required to provide justification for no implementation as well as a schedule for completion of the transition.

The clarification by Lwin (2007) defined the terms *preliminary engineering* and *initiate* in relation to the previous memorandum. It interpreted *preliminary engineering* as "the initiation of the studies or design activities related to identification of the type, size, and/or location of bridges," and the term *initiate* as the "date when Federal-aid funds are obligated for preliminary engineering." When Federal funds are not used in engineering but in other phases, the term *initiate* referred to the date the State "obligates or expends their own funds for preliminary engineering." Other clarifications in relation to modifications included shelved bridge projected already designed and the term *new bridges* was introduced. Lwin (2007) also noted that the policy was effective on "States-initiated Federal-aid" funded projects, and not just projects funded by Highway Bridge Program funds.

5.7.2 AASHTO RECOMMENDATIONS

AASHTO recommendations are in the form of the Bridge Design Specifications (AASHTO 2007, 2009). The Specifications are intended as a guideline presenting standard of practice methods for design. The specifications provide a commentary which provides a brief background to the material as well as suggested sources which carry out the intent of the Specifications.

The basic methodology established within the LRFD Bridge Design Specifications is that bridge structures shall be designed to meet all applicable limit states. This requires that all design analyses must result in the factored resistances being greater than the factored loads. The limit states provided by AASHTO (2009) are the service limit state, fatigue and fracture limit state, strength limit state, and extreme event limit states. The limit states directly applicable to deep foundations are the service limit state, strength limit state, and extreme event limit state. Corrosion and deterioration, determination of minimum pile penetration, and driveability should be considered when applicable. Indicator piles, or test piles, are also recommended.

5.8 DISCUSSION

5.8.1 SDDOT'S DESIGN METHOD

The SDDOT's current method for predicting axial compressive capacity is a unique method. Previous research studies show the method has promise, but were qualified with the recommendation for field testing to verify the design method. This research agrees that the underlying premise behind the design method does indeed show promise, but there are shortcomings that were discovered during the review.

The method appears to have developed over the course of the last 4 or 5 decades with incremental changes periodically implemented, but the timeline of actual changes could not be established through documentation. The majority of the method appears to have been developed in the 1960s and is therefore a somewhat dated method. As previously stated, many advances in foundation engineering have been developed in the profession that could benefit SDDOT's design approach to deep foundation design. During the course of reviewing the SDDOT's design method, no documentation could be obtained that showed where portions of the design method were developed. Documents provided by the SDDOT enabled an understanding of the process of design, but documentation supporting the qualitative input could not be obtained. As such, it was not possible to determine the basis or reliability of the drive or pull test or the information in Table 5-1 through Table 5-4. Of particular importance as it relates to the goals of this research project, it was not possible to determine with certainty the magnitude of the built-in factor of safety in the SDDOT design method. Past research commented that the method appears to be conservative in some cases; however, some of the research showed the method could over predict capacity, as well, resulting in a lower factor of safety in the design than desired. This makes it particularly difficult to assess the efficiency of the SDDOTs design method.

Another important aspect to bring to the SDDOT's attention is the lack of field data in verifying the design method. The existence of field data to verify the method would quantify many of the undocumented areas of the design method identified during this research study. As will be presented later in this report, an important recommendation this research provides is the need for field testing to verify the SDDOT's method, as well as further the resistance factor calibration process that will be discussed in the next chapter.

In regards to SDDOT's field verification method (driving formula), the FHWA in their courtesy review recommended the SDDOT investigate the use of other driving formula that could result in higher reliability. Past studies have shown that a driving formula based on ENR may not be reliable. Lowery et al (1969), for example, states that when the ENR formula is used for heavy concrete piles and large steel piles, the formula provides inconsistent results. As such, we concur with the FHWA's recommendation. It was also not possible to verify the built-in factor of safety in SDDOT's driving formula. We also noted that the SDDOT is not performing a driveability analysis or having contractors perform this as part of construction (i.e. required in the construction specifications). As discussed previously, a driveability analysis, developed using software such as GRLWEAP, ensures driving stresses are within tolerable limits and also provides bearing graphs developed using the wave equation. These types of analyses could add efficiencies if incorporated into the design.

In the final analysis, it is worth pointing out that based on our discussions with the SDDOT, the SDDOT has not observed any failures of bridge foundation systems relating to axial pile capacity. While this is an ultimate goal of any foundation design, it is inherent in the design that the designer know with a high degree of certainty how close or how far the design is from either unacceptable foundation performance or failure.

5.8.2 SDDOT'S METHOD RELATIVE TO FHWA REQUIREMENTS

Based on our analysis, it appears that the SDDOT currently is in compliance with FHWA requirements for the transition to LRFD within their design methodology. This research project initiates the SDDOT in the process of transitioning from a calibration-to-fit method of LRFD to a true reliability-based LRFD platform. The FHWA policy was created in conjunction with AASHTO input and therefore AASHTO recommendations are not only the standard of practice, but in essence are considered a part of the FHWA policy shift to LRFD.

AASHTO recommendations for LRFD include multiple limit states which must be considered and met (AASHTO 2009). Requirements to be in complete compliance with the Specifications (AASHTO 2009) are that all of the limit states and applicable recommendations are met. The SDDOT deep foundation methodology currently only considers the strength limit state. Within the framework of the AASHTO recommendations for limit state design, the Geotechnical Section of the SDDOT has not yet incorporated formal methods to address all limit states.

5.9 SUMMARY

The SDDOT capacity prediction method is a two phase process which involves a drive test and a pull test. Drilled shaft capacity is determined solely on the shaft resistance deeper than four feet above the end of the PK-rod drive test. Field verification of deep foundation capacity is performed by the use of a modified

ENR formula, which has not been calibrated to state methods using the LRFD approach. The same field verification formula is used for a large portion of the static capacity prediction method.

The SDDOT has funded prior studies including Bump et al. (1971) and Turner and Sandburg (1992). Bump et al. (1971) researched the use of a dynamic analysis method for field verification and the drilled shaft design equation used during the time of the study. Turner and Sandburg (1992) attempted to verify the use of the pull test by using small scale cast-in-place piles.

The FHWA has provided the SDDOT a complimentary review which included an initial transition to LRFD accomplished using the calibration-to-fit approach. The assumption made by the FHWA in the calibration-to-fit in relation to the factor of safety employed by the SDDOT may not be correct and highlight the need for a reliability based calibration. The FHWA review included multiple recommendations for the SDDOT, including the suggestion of a study to analyze the SDDOT's design method.

6 APPROACHES USED BY NEIGHBORING STATES

6.1 INTRODUCTION

As part of the literature search process for this project, the research team contacted Department of Transportation Offices in six surrounding states which include North Dakota, Minnesota, Iowa, Nebraska, Wyoming and Montana. The purpose was to discern what methods these states are using relative to transitioning to LRFD bridge foundation design and methods for resistance factor calibration. Two others familiar with similar projects to this research study were also contacted (National Academies and University of Massachusetts).

A questionnaire was produced from a standard list of questions that was developed by the research team. The questions were generated from topics that were deemed relevant to the subject research. The questionnaire was forwarded to the technical panel for review and comment. The comments were then incorporated into the questionnaire.

State DOTs were contacted via telephone and were asked the list of standard questions. The standard list of questions that forms the questionnaire is presented in Section 6.2. Specifically, the following neighboring State Department of Transportation personnel were interviewed using the survey questionnaire:

- Montana, Rich Jackson, Geotechnical Engineer
- Wyoming, Mark Falk, Assistant Chief Engineering Geologist
- Iowa, Ahmad Abu-Hawash, Chief Structural Engineer
- Iowa, Kyle Frame, Construction
- Iowa, Ken Dunker, Bridge Engineer
- Nebraska, Omar Qudus, Chief Geotechnical Engineer
- Minnesota, Gary Pearson, Foundations Engineer
- North Dakota, John Ketterling

The following other individuals that are known to have performed similar research or knowledgeable in LRFD implementation and resistance factor calibration were also interviewed:

- University of Massachusetts – Lowell (Dr. Samuel Paikowsky); and
- The National Academies (Jerry DiMaggio, formerly of the FHWA).

Section 6.3 provides a detailed summary of pertinent responses.

6.2 QUESTIONNAIRE

The following outline of questions was used as the basis for the Questionnaire.

General Questions

1. How long have you been involved with the design of deep bridge foundations in your state?
2. How long have you been involved with the use of LRFD in the design of deep foundations?

Field Explorations/Laboratory Testing

3. What procedures do you use for subsurface investigations in support of deep foundation design, e.g. SPT, CPT, GPR, others?
4. What information is collected in the field, e.g. drilling logs, etc.?

5. How is that information transformed/transferred to the design engineer for use in deep foundation design (e.g., any interaction, a combined effort)?
6. What procedures are used in the field to collect soil samples for subsequent laboratory testing?
7. How are undisturbed soil samples obtained? Can you comment on the quality of the undisturbed samples you obtain?
8. What laboratory testing/procedures are used in support of deep foundation design?
9. How are the results of the laboratory tests/procedures used in deep foundation design?
10. Are there any unique geological/geotechnical conditions in your area/state as it relates to deep foundation design?
11. Does your state perform any nonstandard or hybridized testing (in the field or laboratory) in support of deep foundation design? Why?

Design Procedures for Deep Foundations

12. What methods of design analysis do you use, e.g. Alpha Method, Beta Method, Meyerhof, Reese and Wright, Reese and O'Neil, etc. for pile/shaft design (driven, drilled, etc.)?
13. Repeat of question 9.
14. Have you developed any nonstandard or hybridized methods for analysis? If so, can you please briefly explain what they are and why you use them (this assumes that the methods in the AASHTO Specifications generally represent the standard)?
15. Does your organization/state use any dynamic analyses in support of the design phase of the project, e.g. WEAP analyses, etc.?
16. When was the last review of your design practices performed by the FHWA? (What were the results of the review, were there any specific FHWA direction or requirements, even from a district office? Exclusions or modifications allowed? Problems noted?)
17. Has your state/organization developed its own guidebook/standards for the design of deep bridge foundations based on LRFD design?

LRFD Calibration

18. Has your state/organization performed any analyses/calibration in order to further refine geotechnical reduction factors (beyond what is provided the Design Specifications) based on local geotechnical conditions?
19. If so, what method was used to calibrate resistance factors?
20. Has your state/organization developed a database of field testing and geotechnical data in support of LRFD calibration (and associated periodic improvement of resistance factors as the database is expanded)?

Field Testing of Pile Capacity

21. What field methods do you use to verify capacity/settlement/load distribution, etc.?
    a. Comments on no testing?
    b. Comments on driving formulas?
    c. Comments on PDA testing and the CASE method?
    d. Comments on dynamic refinements (CAPWAP)?
    e. Comments on load testing? Type of load testing, use of telltales, strain gages, etc.?

f. Comments on methods to reduce/interpret the load test data? Methods of determining ultimate capacity, load distribution, etc.?

Final Questions

22. From your experience, has the implementation of specified LRFD increased the reliability and economy of foundation substructures? Have you performed any formal studies in this regard?
23. How do you handle the serviceability limit state in LRFD design?
24. How do you design pile groups?

6.3 RESULTS

The results from the questionnaire for the surrounding states are summarized in Table 6-1. Iowa was not reached for questions 4-14, 23, or 24 and Nebraska was not asked questions 23 or 24.

Table 6-1: Questionnaire Response Summary Table

Question	Summary of Responses
1. How long have you been involved with the design of deep bridge foundations in your state?	Responses varied by individual, ranging from 9- 40 years.
2. How long have you been involved with the use of LRFD in the design of deep foundations?	Responses varied by individual, ranging from 0-2 ½ years.
3. What procedures do you use for subsurface investigations in support of deep foundation design, e.g. SPT, CPT, GPR, others?	All states use SPT. Most states use some CPT. Wyoming uses proprietary Drive Point.
4. What information is collected in the field, e.g. drilling logs, etc.?	All states collect SPT values.
5. How is that information transformed/transferred to the design engineer for use in deep foundation design (e.g., any interaction, a combined effort)?	Answers varied by state. Most provided engineer with boring log. Nebraska uses a developed database.
6. What procedures are used in the field to collect soil samples for subsequent laboratory testing?	All states use a split spoon and Shelby tubes. Minnesota and Nebraska pushed a CPT with a piezocone. North Dakota pushes a thin-walled tube behind a 5 foot continuous sampler. Montana does wire line triple tube for rock coring.
7. How are undisturbed soil samples obtained? Can you comment on the quality of the undisturbed samples you obtain?	All states used Shelby tubes.
8. What laboratory testing/procedures are used in support of deep foundation design?	All states did unconfined compression on undisturbed samples (UC) of either soils or rock. Some states do triaxial testing. Wyoming does direct shear on high quality cohesive samples.
9. How are the results of the laboratory tests/procedures used in deep foundation design?	Most states record friction angle, compressive strength, cohesion, blow counts. These are later used in either AASHTO methods or FHWA programs Wyoming uses boring logs.

Question	Summary of Responses
10. Are there any unique geological/geotechnical conditions in your area/state as it relates to deep foundation design?	Minnesota has the Red River Valley with large lateral movements over time and creep. North Dakota has a lot of soft expansive soils and deep clays and silts. Montana has Old Glacial Lake Missoula with 0 blow count silts. They also have some high end value sands. Wyoming has 90% shales and 10% clays with some rounded gravel in the Northwest which does not allow for end bearing of piles.
11. Does your state perform any nonstandard or hybridized testing (in the field or laboratory) in support of deep foundation design? Why?	Wyoming uses a drive point method.
12. What methods of design analysis do you use, e.g. Alpha Method, Beta Method, Meyerhof, Reese and Wright, Reese and O'Neil, etc. for pile/shaft design (driven, drilled, etc.)?	A combination of AASHTO methods, Reese and O'Neil for drilled shafts, and FHWA drilled (Nordlund) and driven programs comprised the majority of states. Wyoming is in a transition from Nordlund to the β-method.
13. Repeat of question 9.	See No. 9.
14. Have you developed any nonstandard or hybridized methods for analysis? If so, can you please briefly explain what they are and why you use them (this assumes that the methods in the AASHTO Specifications generally represent the standard)?	None have.
15. Does your organization/state use any dynamic analyses in support of the design phase of the project, e.g. WEAP analyses, etc.?	Half the states use WEAP for suspected problems. Wyoming and Montana use it on every design. Nebraska uses it to approve hammers on every design.
16. When was the last review of your design practices performed by the FHWA? (What were the results of the review, were there any specific FHWA direction or requirements, even from a district office? Exclusions or modifications allowed? Problems noted?)	Montana, 2003. Minnesota, 2007. Iowa, Spring 2009. Nebraska, Spring 2009. Wyoming, 2005 . North Dakota, Unsure.
17. Has your state/organization developed its own guidebook/standards for the design of deep bridge foundations based on LRFD design?	Iowa and Minnesota have design manuals on LRFD. Montana has a manual but it is not currently in LRFD. Wyoming and North Dakota are in the process of developing guidebooks. Nebraska is creating a LRFD design example.

Question	Summary of Responses
18. Has your state/organization performed any analyses/calibration in order to further refine geotechnical reduction factors (beyond what is provided the Design Specifications) based on local geotechnical conditions?	Most states are still in an interim stage of using calibration to fit. Minnesota and Iowa have studies in progress for developing reduction factors. Montana is beginning pile load testing for calibration. Nebraska finished calibration in 2007.
19. If so, what method was used to calibrate resistance factors?	Minnesota used the NCHRP 507 calibration method. Nebraska used the Washington State Method.
20. Has your state/organization developed a database of field testing and geotechnical data in support of LRFD calibration (and associated periodic improvement of resistance factors as the database is expanded)?	Iowa and Nebraska each have databases of tests. Wyoming is in the process of creating a database. Minnesota used the NCHRP database with selective properties.
21. What field methods do you use to verify capacity/settlement/load distribution, etc.?	
a. Comments on no testing?	Iowa uses WEAP on everything driven.
b. Comments on driving formulas?	Iowa, Wyoming, and Montana use the driving graphs from WEAP. Nebraska's study found the Gates formula inappropriate for local conditions and has a proprietary formula. North Dakota uses a Modified ENR but the geotechnical section is pushing Gates. Minnesota uses a Modified ENR but the study will likely recommend switching to a specifically Modified Gates.
c. Comments on PDA testing and the CASE method?	Half the states own a PDA. Nebraska uses the PDA on all large bridges and small bridges in problem areas. Wyoming uses the PDA on all friction applications, and is beginning to use it with piles in bedrock. Montana uses the PDA about half the time. Minnesota uses is sporadically but has increased the use in the past five years. Iowa uses the PDA for problem solving. North Dakota sporadically uses a PDA.
d. Comments on dynamic refinements (CAPWAP)?	All states do use CAPWAP when PDA testing is done.

Question	Summary of Responses
e. Comments on load testing? Type of load testing, use of telltales, strain gages, etc.?	No states have regularly performed static load tests, most are beginning to or have recently implemented static tests for calibration. Iowa has used telltales, strain gages, O-cell tests on shafts, and have performed three statnamic tests. Minnesota has performed top-deflection, some telltales, one statnamic, and O-cell tests on shafts. Montana has performed top-deflection testing. Nebraska has used O-cells on drilled shafts.
f. Comments on methods to reduce/interpret the load test data? Methods of determining ultimate capacity, load distribution, etc.?	All states which have performed testing used Davisson except Iowa, which had a consultant perform and analyze the static load tests.
22. From your experience, has the implementation of specified LRFD increased the reliability and economy of foundation substructures? Have you performed any formal studies in this regard?	No state has yet to find an economic gain in LRFD mostly due to the use of calibration to fit, although all who are performing calibration expect future savings. Wyoming has found less economy in some cases due to unusual geomaterials. Nebraska has actually found they use 1 to 2 more piles than in the past but they are applying a degree of conservatism in their design approaches at the present.
23. How do you handle the serviceability limit state in LRFD design?	Only North Dakota routinely checks serviceability. Montana checks serviceability if piles cannot be limited to deflections of less than 1”.
24. How do you design pile groups?	Minnesota uses the AASHTO methods for pile group action. Most states maintain a minimum pile separation to avoid group effects. Montana uses AASHTO when they use pile groups, but most bridges do not. Wyoming rarely has pile groups.

6.4 DISCUSSION

The responses indicate a wide variety of the level of practice in the states adjacent to South Dakota. Minnesota, Iowa, and Nebraska appeared the furthest into a reliability-based design method. Wyoming was in the process of creating a database for LRFD implementation but had not yet begun reliability-based calibration.

North Dakota was the only state continuing to implement a driving formula which was not specifically calibrated to the State. Minnesota has since adopted a specific driving formula and Nebraska had developed and implemented one; the other three states surveyed used bearing graphs from a WEAP analysis. All of the states surveyed use the PDA at least periodically, with half using it routinely and all use CAPWAP when the PDA is used. No state routinely performs load tests. The results indicate that capacity verification by reliability-based calibration, wave equation based bearing graphs or dynamic testing is the standard of practice for states surrounding South Dakota.

Relative to meeting the AASHTO (2009) recommended limit states, the standard of practice appears to only account for the strength limit state at this time. North Dakota was the only state to check the service limit state on every design. The stage of LRFD implementation for developing resistance factors for most states was the calibration-to-fit method with plans for future state specific resistance factor calibration. Five of the six surrounding states have or are in the process of creating an LRFD-based design example, guidebook or manual.

6.5 SUMMARY

The responses indicate a wide variety of the level of practice in the states adjacent to South Dakota. Most states appear to be only including the strength limit state in their design practices. South Dakota appears to be within the standard of practice for the region of surrounding states.

7 FIELD DATA DATABASE COMPILATION

7.1 INTRODUCTION

It is necessary to develop a database of load test data, either from static load testing or dynamic testing analyses, in order to calibrate resistance factors for geotechnical design at the Strength Limit State. During the course of this project, load tests were obtained from a number of sources, including the South Dakota Department of Transportation, the Nebraska Department of Roads (NDOR), the Montana Department of Transportation (MDT), and numerous private geotechnical consultants. The load tests were conducted either within South Dakota or within the states surrounding South Dakota and thus within similar geological conditions to South Dakota.

This chapter will discuss the compilation of those load tests into a database that can be utilized for resistance factor calibration. For this project, the developed database consists of both static and dynamic load tests conducted on driven piles and static and O-Cell tests conducted on drilled shafts. The results of each load test were reviewed to ensure sufficient information was available to compute an *actual* resistance for the deep foundation as discussed in Section 4.5.2.2.2. The database has been developed with respect to deep foundation type, and less with respect to soil type, as the SDDOT drive test predictive resistance approach does not vary based on the subsurface conditions.

7.2 FHWA DATABASE

A deep foundation load test database supported by the Federal Highway Administration (FHWA) is discussed in Kalavar and Ealy (2000). The idea of the database was to compile and organize the wealth of geotechnical and foundation load test data that exists in the literature, within various state DOT files, and at various laboratories. A database organized in this manner would certainly benefit projects such as this one, where high quality load test data, along with supplemental information such as boring logs, laboratory test data, etc. can be located in a single location. As described in Kalavar and Ealy (2000), an appropriate "front-end" interface would allow the user to find pertinent data quickly and easily based on deep foundation type, soil type, location, etc. It is the understanding of the research team, however, that a front-end interface was never developed for the database.

The research team asked the SDDOT and the FHWA for a copy of the FHWA load test database in the summer of 2009. The database was delivered to the research team in September 2009 in the form of a Microsoft Access file, and numerous Microsoft Excel files. The research team performed a thorough and exhaustive review of all of the obtained files and concluded that, although the database did contain a large number of load tests for driven piles, unfortunately none of the load tests could be incorporated into the database for this project. The predominant reason for the inability to utilize the load test data is that a lack of soil data, both from the field subsurface investigation and the laboratory, was realized. In general, the database divided the load tests into three general soil classifications: (1) sand, (2) clay, and (3) mixed. This classification system is much too broad to effectively utilize the data within the resistance factor calibration efforts. Further investigation of the database also highlighted that there were numerous cases where there were either inconsistencies within the data or data was lacking to fully utilize the load test. For reference, no load tests for drilled shafts could be located in the database files.

7.3 SDDOT DATABASE

The development of a load test database was initiated by reviewing the load tests that were in the files of the SDDOT. A thorough investigation of those files resulted in locating a number of load tests that were conducted in the late 1960's in support of a research project by the SDDOT. The results of those load tests are summarized in a report by Bump et al. (1971) and will hereinafter be referred to as the "1971 report" for discussion. The report contains static load test results for both driven piles and drilled shafts installed at a number of test sites across the state. The *actual* resistance of the deep foundation for each test is provided, along with the *predicted* resistance using the SDDOT drive test predictive resistance formula and site data. This allowed for easy incorporation of the load test data into the database. The load tests on driven piles included H-piles, timber piles, steel pipe piles, and precast concrete piles. Three additional static load tests were conducted by the SDDOT on timber piles and these results were located in the SDDOT files. These load tests were not included in the 1971 report. Two of these load tests were not included in the database because the piles were only loaded to two times the design load and it did not appear that the piles were failed in bearing. The third load test was included in the database; however, it was not utilized in the calibration effort since it was not possible to locate a predicted resistance for the pile, along with either a drive test log or subsurface information.

The SDDOT provided a report written by Turner and Sandberg (1992) to the research team. This report, hereinafter referred to as the "1992 report" for discussion, contained static pullout load test results conducted on a number of small diameter concrete shafts. The concrete shafts were installed at a number of load test sites across South Dakota in support of a research project funded by the SDDOT and conducted by the University of Wyoming. These load tests were incorporated into the database for information only and were not used in calibration.

The research team contacted a number of Departments of Transportation for the states surrounding South Dakota, along with a number of specialty geotechnical contractors and private geotechnical consultants. Based on these contacts, the research team was able to acquire a number of dynamic analyses of driven piles and O-Cell tests on drilled shafts from the Nebraska Department of Roads (NDOR), along with dynamic analyses of driven piles from the Montana Department of Transportation (MDT). The research team was also able to acquire a number of static load tests and dynamic analyses of driven piles from private geotechnical engineering consultants. The additional load tests for the driven piles consisted of tests conducted on H-piles, pipe piles, and precast concrete piles.

All of the basic testing information from these load tests was incorporated into the load test database as provided in Table 7-1.

7.4 RESULTS

The load test database was organized initially based on deep foundation type rather than geomaterial type. The first series of load tests are for driven piles and the latter load tests are for drilled shafts. Within each foundation type group, the load tests are organized by site for easy referencing. For each load test, the following information has been provided:

1. Source of Test – Either SDDOT, NDOR, MDT, or private geotechnical consultant. Those load tests that were located in either the 1971 or 1992 report from the SDDOT are also indicated.
2. Site – Load test site.
3. Test Type – Either static, PDA & CAPWAP analysis, or O-Cell.
4. Foundation Type – A description of the deep foundation type, along with any geometric information.
5. Pile Length Below Grade – The length of the pile installed in the geomaterial (feet).
6. Soil Type – A brief description of the geomaterial along the interface of the deep foundation and at the toe.

Table 7-1 Summary of Developed Load Test Database

1971	1992	Used in Calibration	Source of Test	Site	Test Type	Foundation Type	Pile Length Below Grade (ft)
*		*	SDDOT	Madison	Static Load Test	Timber pile	26
*		*	SDDOT	Madison	Static Load Test	HP 10x42	32.5
*		*	SDDOT	Madison	Static Load Test	Pipe pile (12")	34
*		*	SDDOT	Madison	Static Load Test	Precast concrete pile(12" x 14")	26.5
*		*	SDDOT	Madison	Static Load Test	Precast concrete pile(12" x 14")	26
*		*	SDDOT	Watertown West	Static Load Test	Timber pile	28.5
*		*	SDDOT	Watertown East	Static Load Test	Timber pile	29.5
*		*	SDDOT	Watertown	Static Load Test	Pipe pile(12")	29.5
*		*	SDDOT	Watertown	Static Load Test	Precast concrete pile (14" octagonal)	32
*		*	SDDOT	Spearfish North	Static Load Test	Timber pile	18.5
*		*	SDDOT	Spearfish South	Static Load Test	Timber pile	20.5
*		*	SDDOT	Spearfish North	Static Load Test	HP 10x42	30
*		*	SDDOT	Spearfish South	Static Load Test	HP 10x42 (Barbed)	30
*		*	SDDOT	Spearfish North	Static Load Test	Precast concrete pile	14
*		*	SDDOT	Spearfish South	Static Load Test	Precast concrete pile	20
*		*	SDDOT	Wendte	Static Load Test	Timber pile	23.25
*		*	SDDOT	Wendte	Static Load Test	HP 8x36	44.9
*		*	SDDOT	Wendte	Static Load Test	HP 10x42	43.8
*		*	SDDOT	Wendte	Static Load Test	HP 10x42 (Barbed)	43.6
*		*	SDDOT	Wendte	Static Load Test	Pipe pile (12")	25
			SDDOT	Walworth County	Static Load Test	Timber pile	30
		*	SDDOT	Highway 46 Beresford	Static Load Test	Precast concrete pile (14" x 14")	50
			Consultant	Rapid City, SD Block 75 Parking Structure	PDA & CAPWAP Analysis	Pipe pile (7”)	TP-1 = 38 TP-2 = 39 TP-3 = 40
			Consultant	Rapid City, SD Rapid City Journal Satellite Plant	Static Load Test	Pipe pile (7.625") close ended	40
			Consultant	Rawlins, WY Hampton Inn	PDA & CAPWAP Analysis	Pipe pile (8.625")	TP-1 = 27 TP-3 = 25 TP-5 = 26
			Consultant	Rock Springs, WY Holiday Inn Express	PDA & CAPWAP Analysis	HP 10x42	TP-2 = 33 TP-3 = 34
			Consultant	Sheridan, CO Chile's Restaurant	PDA & CAPWAP Analysis	HP 10x42	TP-1 = 62 TP-2 = 54 TP-3 = 53 TP-4 = 62 TP-5 = 54
			MDT	Bridge over Mud Creek (NB) Lake County, MT	PDA & CAPWAP Analysis	Fluted pipe pile (14")	TP-2 = 84 TP-3 = 55 TP-4 = 55 TP-7 = 55 TP-9 = 55 TP-11 = 55
			MDT	Bridge over Mud Creek (SB) Lake County, MT	PDA & CAPWAP Analysis	Fluted pipe pile (14")	TP-1-1 = 98 TP-15-1 = 98 TP-1-2 = 98 TP-15-2 = 98
			MDT	Bridge over Mud Creek (Old US 93) Lake County, MT	PDA & CAPWAP Analysis	Fluted pipe pile (16")	TP-1 = 40 TP-3 = 40 TP-7 = 40 TP-8 = 60
			MDT	Bridge over Blackfoot River Milltown, MT	PDA & CAPWAP Analysis	Fluted pipe pile (16")	TP-4-1 = 81 TP-4-4 = 45 TP-4-6 = 166
		*	SDDOT	Cretex Facility - Rapid City	PDA & CAPWAP Analysis	Precast concrete pile (12" x 12")	41
		*	Consultant	Civic Center - Rapid City	Static Load Test	Pipe pile (7.625")	30

Table 7-1 Summary of Developed Load Test Database (continued)

Soil Type	Boring Log	SPT/ Drive Test/ Pull Out Data	Actual Resistance (kip)			Predicted Resistance (kip)		
			Shaft	Toe	Total	SDDOT Drive Test	Gates Formula	FHWA Method
silty/sandy clay over clay till	YES	Drive Test	-	-	280	220	-	-
silty/sandy clay over clay till	YES	Drive Test	-	-	210	220	-	-
silty/sandy clay over clay till	YES	Drive Test	-	-	180	212	-	-
silty/sandy clay over clay till	YES	Drive Test	-	-	230	220	-	-
silty/sandy clay over clay till	YES	Drive Test	-	-	240	220	-	-
sand and gravel over till	YES	Drive Test	-	-	270	200	-	-
sand and gravel over till	YES	Drive Test	-	-	182	210	-	
sand and gravel over till	YES	Drive Test	-	-	330	211	-	-
sand and gravel over till	YES	Drive Test	-	-	660	200	-	-
silty, clay shale over gravel	YES	Drive Test	-	-	150	146	-	-
silty, clay shale over gravel	YES	Drive Test	-	-	116	152	-	
silty, clay shale over gravel	YES	Drive Test	-	-	150	152	-	-
silty, clay shale over gravel	YES	Drive Test	-	-	150	152	-	-
silty, clay shale over gravel	YES	Drive Test	-	-	180	180	-	-
silty, clay shale over gravel	YES	Drive Test	-	-	140	154	-	-
silty clay over shale	YES	Drive Test	-	-	100	156	-	
silty clay over shale	YES	Drive Test	-	-	140	205	-	-
silty clay over shale	YES	Drive Test	-	-	170	190	-	-
silty clay over shale	YES	Drive Test	-	-	230	190	-	-
silty clay over shale	YES	Drive Test	-	-	110	164	-	-
-	NO		-	-	92	-	-	-
silt/clay till over sand/gravel	YES	Pull Out	-	-	440	310	-	-
clay and gravel alluvium, shale bedrock	NO		TP-3: 123	TP-3: 69	TP-1: 260 TP-2: 260 TP-3: 192	-	-	200
clay, sandy gravel over shale	YES	SPT	-	-	190	-	-	-
loose to hard silty and clayey sands	YES	SPT	TP-1: 247 TP-3: 186 TP-5: 67	TP-1: 73 TP-3: 85 TP-5: 136	TP-1: 320 TP-3: 271 TP-5: 204	-	-	80
fill overlying bedrock	YES		TP-2: 71 TP-3: 35	TP-2: 235 TP-3: 170	TP-2: 306 TP-3: 205	-	-	-
fill overlying bedrock	YES		TP-1: 67 TP-2: 61 TP-3: 63 TP-4: 60 TP-5: 129	TP-1: 242 TP-2: 240 TP-3: 250 TP-4: 238 TP-5: 270	TP-1: 309 TP-2: 301 TP-3: 313 TP-4: 298 TP-5: 399	-	-	275
sand and silty clay over stiff clay	YES	SPT	TP-2: 183 TP-3: 120 TP-4: 173 TP-7: 80 TP-9: 56 TP-11: 40	TP-2: 46 TP-3: 30 TP-4: 5 TP-7: 1 TP-9: 82 TP-11: 88	TP-2: 229 TP-3: 150 TP-4: 178 TP-7: 81 TP-9: 138 TP-11: 127	-	-	-
silty sand over stiff clay	YES	SPT	TP-1-1: 294 TP-15-1: 185 TP-1-2: 222 TP-15-2: 248	TP-1-1: 60 TP-15-1: 59 TP-1-2: 25 TP-15-2: 32	TP-1-1: 354 TP-15-1: 244 TP-1-2: 247 TP-15-2: 281	-	-	338
silt/clay over stiff clay	YES	SPT	TP-1: 95 TP-3: 554 TP-7: 201 TP-8: 156	TP-1: 40 TP-3: 55 TP-7: 64 TP-8: 29	TP-1: 134 TP-3: 610 TP-7: 265 TP-8: 185	-	-	270
sandy gravel	YES	SPT	TP-4-1: 52 TP-4-4: 104 TP-4-6: 209	TP-4-1: 230 TP-4-4: 125 TP-4-6: 117	TP-4-1: 281 TP-4-4: 230 TP-4-6: 326	-	-	-
Sand/gravel over shale	YES	SPT & Drive Test	640	95	734	460	-	-
Clay over very stiff clay	YES	SPT & Drive Test	-	-	120	230	-	-

Table 7-1 Summary of Developed Load Test Database (continued)

1971	1992	Used in Calibration	Source of Test	Site	Test Type	Foundation Type	Pile Length Below Grade (ft)
		*	Consultant	57th Street Bridge Sioux Falls	PDA & CAPWAP Analysis	HP 12x53	88
			NDOR	Pile 1	PDA	HP 10x42	75
			NDOR	Pile 2	PDA	HP 12x53	95
			NDOR	Pile 3	PDA	Pipe pile	85
			NDOR	Pile 4	PDA	Pipe pile	45
			NDOR	Pile 5	PDA	HP 12x53	65
			NDOR	Pile 6	PDA	Pipe pile	100
			NDOR	Pile 7	PDA	HP 12x53	70
			NDOR	Pile 8	PDA	Pipe pile	100
			NDOR	Pile 9	PDA	Pipe pile	80
			NDOR	Pile 10	PDA	HP 10x42	70
			NDOR	Pile 11	PDA	Pipe pile	80
			NDOR	Pile 12	PDA	HP 14x89	35
			NDOR	Pile 13	PDA	HP 14x89	110
			NDOR	Pile 14	PDA	Pipe pile	60
			NDOR	Pile 15	PDA	HP 14x89	85
			NDOR	Pile 16	PDA	HP 12x53	70
			NDOR	Pile 17	PDA	Pipe pile	90
			NDOR	Pile 18	PDA	Pipe pile	70
			NDOR	Pile 19	PDA	Pipe pile	65
			NDOR	Pile 20	PDA	Precast concrete pile	50
			NDOR	Pile 21	PDA	Precast concrete pile	45
			NDOR	Pile 22	PDA	Precast concrete pile	50
			NDOR	Pile 23	PDA	Precast concrete pile	65
			NDOR	Pile 24	PDA	Precast concrete pile	60
*			SDDOT	Wendte	Static Load Test	Drilled shaft (18")	20.4
*			SDDOT	Okaton	Static Load Test	Drilled shaft (18")	20
*			SDDOT	Wall	Static Load Test	Drilled shaft (18")	20.5
*			SDDOT	Pierre	Static Load Test	Drilled shaft (18")	10.5
*			SDDOT	Pierre	Static Load Test	Drilled shaft (18")	20
*			SDDOT	Pierre	Static Load Test	Drilled shaft (8")	10
*			SDDOT	Pierre	Static Load Test	Drilled shaft (8" over void)	10
*			SDDOT	Pierre	Static Load Test	Drilled shaft (4.5")	10
*			SDDOT	Pierre	Static Load Test	Drilled shaft (4.5" over void)	10
	*		SDDOT	Belle Fourche Formation	Static Load Test	Drilled shaft (6")	5.5
	*		SDDOT	Belle Fourche Formation	Static Load Test	Drilled shaft (6")	16
	*		SDDOT	Belle Fourche Formation	Static Load Test	Drilled shaft (6")	9
	*		SDDOT	Rapid Creek	Static Load Test	Drilled shaft (8")	17
	*		SDDOT	Rapid Creek	Static Load Test	Drilled shaft (8")	15
	*		SDDOT	Rapid Creek	Static Load Test	Drilled shaft (8")	10
	*		SDDOT	Spearfish Formation	Static Load Test	Drilled shaft (6")	12
	*		SDDOT	Spearfish Formation	Static Load Test	Drilled shaft (6")	12
	*		SDDOT	Spearfish Formation	Static Load Test	Drilled shaft (6")	12
	*		SDDOT	Carlile Formation	Static Load Test	Drilled shaft (6")	15
	*		SDDOT	Carlile Formation	Static Load Test	Drilled shaft (6")	18
	*		SDDOT	Carlile Formation	Static Load Test	Drilled shaft (6")	18
	*		SDDOT	Pierre Formation	Static Load Test	Drilled shaft (6")	15
	*		SDDOT	Pierre Formation	Static Load Test	Drilled shaft (6")	15
	*		SDDOT	Pierre Formation	Static Load Test	Drilled shaft (6")	15
	*		SDDOT	Pierre Formation	Static Load Test	Drilled shaft (6")	6
	*		SDDOT	Pierre Formation	Static Load Test	Drilled shaft (6")	18
	*		SDDOT	Pierre Formation	Static Load Test	Drilled shaft (6")	16
			NDOR	Yankton, SD Highway 81 over Missouri River	O-Cell	Drilled shaft (96")	100
			NDOR	Wahoo, NE	O-Cell	Drilled shaft (66")	70

Table 7-1 Summary of Developed Load Test Database (continued)

Soil Type	Boring Log	SPT/ Drive Test/ Pull Out Data	Actual Resistance (kip)			Predicted Resistance (kip)		
			Shaft	Toe	Total	SDDOT Drive Test	Gates Formula	FHWA Method
till over very dens glacial outwash	NO	Drive Test	529	78	606	356	-	350
clay over till	NO	SPT	-	-	240	-	383	-
clay/sand over dense till	NO	SPT	-	-	288	-	421	
loess over dense sand	NO	SPT	-	-	353	-	325	-
sand/clay over gravel	NO	SPT	-	-	189	-	299	-
sand/gravelover limestone	NO	SPT	-	-	375	-	567	-
clay and coarse sand over clay	NO	SPT	-	-	744	-	677	-
clayey silt over till	NO	SPT	-	-	328	-	474	-
Clay over very dense till	NO	SPT	-	-	264	-	421	-
clay over till	NO	SPT	-	-	293	-	361	-
clay over silt	NO	SPT	-	-	148	-	303	-
clay	NO	SPT	-	-	281	-	490	-
clay over limestone	NO	SPT	-	-	995	-	962	-
sand over limestone	NO	SPT	-	-	1326	-	1124	-
silty clay over till	NO	SPT	-	-	212	-	286	-
silt/sand over clay	NO	SPT	-	-	416	-	521	-
clay over shale	NO	SPT	-	-	381	-	1459	-
sand/gravel	NO	SPT	-	-	400	-	476	-
sand/gravel	NO	SPT	-	-	178	-	257	-
sand	NO	SPT	-	-	273	-	295	-
clay over sand	NO	SPT	-	-	262	-	416	-
sand/clay over gravel	NO	SPT	-	-	118	-	238	-
sand/silt over gravel	NO	SPT	-	-	180	-	332	-
sand/gravel	NO	SPT	-	-	295	-	426	-
sand	NO	SPT	-	-	168	-	308	-
silty clay over shale	YES	Drive Test/ Pullout	-	-	290	150	-	-
shale	YES	Drive Test/ Pullout	-	-	150	174	-	-
silty clay	YES	Drive Test/ Pullout	-	-	300	290	-	-
shale	YES	Drive Test/ Pullout	-	-	120	116	-	-
shale	YES	Drive Test/ Pullout	-	-	350	242	-	-
shale	YES	Drive Test/ Pullout	-	-	46	50	-	-
shale	YES	Drive Test/ Pullout	-	-	38	53	-	-
shale	YES	Drive Test/ Pullout	-	-	36	28	-	-
shale	YES	Drive Test/ Pullout	-	-	36	21	-	-
shale	NO	Drive Test/ Pullout	19.8	-	20	29	-	-
shale	NO	Drive Test/ Pullout	27.5	-	-	-	-	-
shale	NO	Drive Test/ Pullout	21.5	-	21.8	48	-	-
sand, silt, gravel	NO	Drive Test/ Pullout	-56.1	0.9	57	12	-	-
sand, silt, gravel	NO	Drive Test/ Pullout	-44.1	-	45	11	-	-
sand, silt, gravel	NO	Drive Test/ Pullout	50.4	0.6	51	7	-	-
shale/sandstone	NO	Drive Test/ Pullout	30.4	1.1	31.5	40	-	-
shale/sandstone	NO	Drive Test/ Pullout	37.4	1.1	38.5	40	-	-
shale/sandstone	NO	Drive Test/ Pullout	20.9	1.1	22	40	-	-
siltstone/shale	NO	Drive Test/ Pullout	28.8	-	29.3	60	-	-
siltstone/shale	NO	Drive Test/ Pullout	23.4	-	24	72	-	-
siltstone/shale	NO	Drive Test/ Pullout	28.2	1.2	29.4	72	-	-
shale	NO	Drive Test/ Pullout	52.9	1.1	54	33	-	-
shale	NO	Drive Test/ Pullout	50	1	51	33	-	-
shale	NO	Drive Test/ Pullout	62	1	63	33	-	-
shale	NO	Drive Test/ Pullout	19.8	-	20	19	-	-
shale	NO	Drive Test/ Pullout	58.8	1.2	60	51	-	-
shale	NO	Drive Test/ Pullout	59.4	-	60	45	-	-
overburden overlying shale	YES	-	-	-	4522	-	-	-
alluvial soils overlying sandstone and shale	YES	-	2107	1576	3683	-	-	-

7. Boring Log – An indication of whether a boring log of the test site was included with the load test data.
8. SPT/Drive Test/Pull Out Data – An indication of the additional subsurface information that was included with the load test. Either standard penetration test (SPT) blow counts or a drive test/pull out test conducted by the SDDOT.
9. Actual Resistance – The actual resistance of the deep foundation from the load test (kips). If possible, the total resistance is divided between the side resistance and toe resistance.
10. Predicted Resistance – The predicted resistance of the deep foundation (kips). The resistance is reported with respect to the SDDOT drive test predictive resistance formula, FHWA static capacity equations, or Gates formula.
11. Used in Calibration – This column indicates the load tests that were included in the resistance factor calibration procedure described in detail in Chapter 8. Since the SDDOT utilizes a unique method for static capacity determination of pile capacity, only a select number of the load tests that are included in the database will be utilized in the resistance factor calibration efforts. This will be further discussed in Chapter 8.

7.5 SUMMARY

A load test database was developed for this project that included the results of both static load tests and dynamic analyses of driven piles, along with conventional static load tests and O-Cell tests of drilled shafts. The load test results were obtained from a number of sources in addition to the SDDOT and were tabulated within a spreadsheet based on deep foundation type rather than soil type. The development of a load test database is the first step in the geotechnical resistance factor calibration process. In general, the number of load tests appears to be sufficient to minimally institute the resistance factor calibration process, which will be explained in Chapter 8. However, further development of the database with new load test data, correlated to the SDDOT drive test predictive resistance formula, will significantly benefit and further the resistance factor calibration process in the future.

8 DATABASE ANALYSIS

8.1 INTRODUCTION

As discussed in Chapter 7, the development of an initial database for the calibration of geotechnical resistance factors was completed for this project. The database consisted of both state-wide and regional load test data conducted on various types of driven piles and drilled shafts. In all cases, the deep foundations were installed in the geomaterials that are common or similar to South Dakota. In Section 4.5, it was discussed that the calibration of resistance factors using the reliability theory calibration approach can be completed in a number of ways. In one possible approach, the design agency can calibrate geotechnical resistance factors using local load test data to the deep foundation design methods provided in AASHTO (2009). Another approach is to calibrate geotechnical resistance factors using local load test data to in-house design methods of the agency. To that end, this research project focused on the calibration of resistance factors for the latter approach, which would provide geotechnical resistance factors that could be utilized by the SDDOT for static capacity prediction of deep foundations based on the SDDOT design method outlined in Section 5.3.1.1 and Section 5.3.1.2. It must be noted that the preliminary calibration of resistance factors presented herein is only with respect to the Strength I Limit State as specified in AASHTO (2009) and does not address other limit states. For design of deep foundations at other limit states, such as the Service Limit State or Extreme Limit State, it is recommended that the SDDOT refer to the AASHTO LRFD Bridge Design Specifications (AASHTO 2009). It should be noted, however, that resistance factors have not yet been calibrated for the geotechnical design at these limit states.

8.2 RESISTANCE FACTOR CALIBRATION FOR SDDOT DESIGN METHODS

This section will describe the calibration of resistance factors using the developed database as described in Chapter 7. The calibrated resistance factors will be presented for the various deep foundation systems that are utilized in South Dakota and that are designed using a predictive method based on the SDDOT driving formula or pull test approach. The resistance factors are not separated based on geomaterial type as the SDDOT drive test predictive resistance formula does not distinguish between different geomaterials that may be encountered across the state.

8.2.1 DRIVEN PILES

The calibration of resistance factors was conducted for driven piles consisting of timber, steel (H-sections and pipe sections) and precast concrete. As discussed in Chapter 7, the database utilized in the calibration of resistance factors consisted predominately of load test data provided in Bump et al. (1971), which will again be referred to as the "1971 report" in this discussion. The calibration also included two additional static load tests and two additional dynamic load tests from the load test database. The additional static load tests consisted of a test conducted on a pipe pile installed near the Civic Center in Rapid City and a test conducted on a precast concrete pile installed near Beresford. The dynamic load tests consisted of a test conducted on a precast concrete pile installed within the Cretex facility located in Rapid City, along with a test conducted on an H-pile installed for the 57^{th} Street Bridge located in Sioux Falls.

As mentioned previously, the calibration of resistance factors involves the determination of the *actual* resistance of a pile based on a static load test (or dynamic load test analysis), along with a *predicted* resistance using the design methodology employed by the agency. Therefore, when using the 1971 report for the resistance factor calibration exercises, the research team utilized the data from Table A of the report as this table contained both an actual resistance and a predicted resistance for the various types of piles. For reference, Table A of the 1971 report has been included in Appendix B of this report. In addition, all load tests from the 1971 report that were used in the resistance factor calibration have been included in Table 7.1. It is important that the value of the actual resistance that is selected from the field load tests be consistent from one test to the next. As discussed, this can oftentimes be difficult as there are a number of interpretation techniques that are available for selecting this value from a load-settlement curve and this can lead to inconsistencies. Therefore, for this project, it was desired to select a consistent value from Table A of the 1971 report. To that end, the actual resistance was taken as the *largest* reported value of the <u>ultimate bearing</u> column from the respective load test within Table A. Interestingly, during the static load testing performed in the late 1960's that is included in the 1971 report, the SDDOT performed three load tests on each test pile. The time between each load test was generally several months and was always performed on the same pile. It is theorized that the repeated load testing was an attempt to capture the effect of pile setup and in several of the load tests, the ultimate bearing of the piles did indeed increase with time. However, there were also several load tests where the ultimate bearing decreased with each subsequent load test. This behavior could be the result of any number of phenomena and is beyond the scope of this project. However, the assurance of consistency within the resistance factor calibration computations was the major reason why the largest reported ultimate bearing value was utilized. As discussed, although there are a number of techniques to interpret the actual resistance from the load-settlement curve developed during a static load test, no specific details were provided in the 1971 report as to how the ultimate bearing values were determined. For most of the load tests, the raw field test data was located in old records and the research team was able to re-develop the original load-settlement curves in Excel in order to more easily analyze the data. In general, upon inspection of the load-settlement curves from each load test, it appears that the ultimate bearing values reported in Table A generally correspond to a load value between what is interpreted using the Davisson offset limit method and where the load-settlement curve becomes asymptotic. An example of the Davisson offset limit interpretation method is shown in Figure 8-1 using the load-settlement curves from one of the load tests conducted at the Spearfish site. The predicted static resistance values from Table A of the 1971 report were reported as <u>Test Hole (tons)</u> and were assumed to be based on the employment of the SDDOT drive test predictive resistance formula using the applicable data from the drive test at the test site of interest. However, when comparing the SDDOT drive test predictive resistance formula (Equation 5-1) to several dynamic formulas reported in Reese et al. (2006), it is realized that the SDDOT formula contains a built-in factor of safety. As discussed in Section 5.3.1.1, the built-in factor of safety appears to be equal to 4. To that end, the values reported under <u>Test Hole (tons)</u> in Table A must be multiplied by a factor of safety equal to 4 in order to compute the predicted nominal resistance (i.e. ultimate capacity) for use in the resistance factor calibration. These values are reported as "Test Hole (UM)" in Table 8-1 to Table 8-4 based on the Ultimate Method described in Chapter 4.

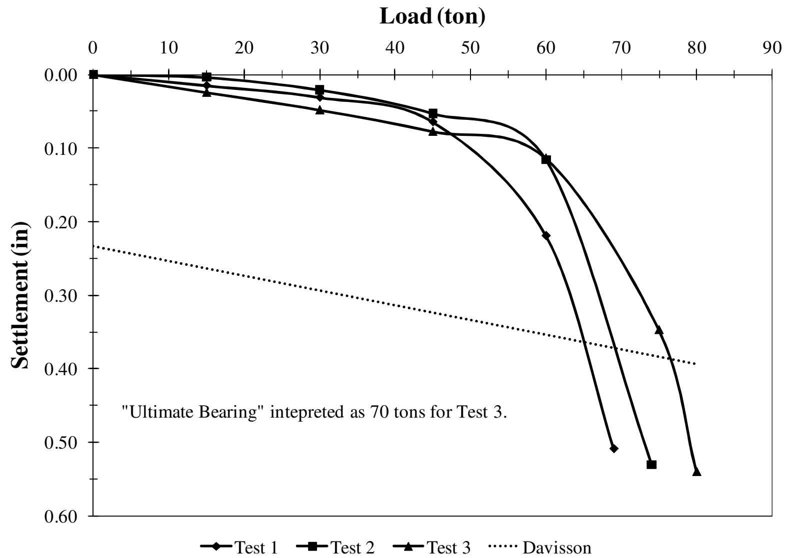

Figure 8-1: Load-settlement curve from load test of HP10x42 from Spearfish North site.

For the additional load tests that were used in the calibration, the actual pile resistance was generally provided in a report that described the results of the load testing. A review of the reports that accompanied the static load tests conducted on the piles installed at the Civic Center in Rapid City indicate that the actual resistance for these piles was also likely interpreted using a combination of the Davisson offset limit method, along with the point at which the load-settlement curve became asymptotic. For the dynamic load tests conducted on the piles installed at the Cretex facility in Rapid City and the 57th Street Bridge in Sioux Falls, the actual resistance was determined from a CAPWAP analysis of the PDA data, as described in Sections 4.4.6.2 and 4.4.6.3 and was provided directly in the project report for the dynamic load test results. The SDDOT was able to conduct a drive test at each of these additional load test sites and therefore computations for the predicted capacity based on the use of the Ultimate Method (UM) approach for a pile of the same length as the test pile at each site was provided to the research team to use in the calibration efforts. For reference, an example for computing the predicted resistance for a driven pile using the Ultimate Method approach is provided in Section 5.3.1.1 using the applicable drive test data.

In Chapter 7, a single database was developed utilizing load tests that were obtained by the research team during the course of the project. This database included the load tests for the 1971 and 1992 reports, along with a number of additional load tests that were conducted by private consultants or DOTs in several of the states surrounding South Dakota. Unfortunately, it was not possible to utilize a number of these additional load tests in the resistance factor calibration and thus an explanation of why these additional load tests were not utilized in the resistance factor calibration efforts is warranted. The resistance factor calibration method requires a predicted resistance that is determined using the static capacity methodology employed by the design agency at each load test site. In the case of the SDDOT, this would require that the drive test be conducted at the location of each of these additional load tests. Unfortunately, this was not possible for these particular load tests due to either access issues near the original load test site or the economics of traveling to each site to conduct the drive test. The research team did attempt to determine the predicted resistance for a number of these load tests using potential energy correlations between the traditional field investigation data, such as SPT, and the SDDOT drive test predictive resistance formula. However, such a correlation would have likely introduced too much uncertainty for a reliable calibration. Since the SDDOT does not currently employ an alternative capacity prediction design methodology to complement the SDDOT drive test predictive resistance formula, such as the methods specified in the AASHTO Bridge Design Specifications (Section 4.4.3), it was also not possible to include these load tests in the resistance factor calibrations based on those methods. However, since the results of these load tests have been summarized by the research team, it may be possible to incorporate them into future resistance factor calibration efforts, especially if a different static capacity prediction method is adopted by the SDDOT in addition to the driving formula.

Using the reliability theory calibration method outlined in Section 4.5.2.2.2, the value of the resistance factor for each pile type is computed using Equation 4-12. For ease of reference, Equation 4-12 has been provided below as Equation 8-1.

$$\phi_R = \frac{\lambda_R \left(\frac{\gamma_D E(Q_D)}{E(Q_L)} + \gamma_L \right) \sqrt{\frac{1 + \Omega_{QD}^2 + \Omega_{QL}^2}{1 + \Omega_R^2}}}{\left(\lambda_{QD} \frac{E(Q_D)}{E(Q_L)} + \lambda_{QL} \right) e^{\beta_T \sqrt{\ln\left[\left(1+\Omega_R^2\right)\left(1+\Omega_{QD}^2+\Omega_{QL}^2\right)\right]}}}$$

Equation 8-1

Recall that the unknown values in Equation 8-1, once all the factors corresponding to the load are determined, are the COV of the resistance, Ω_R, and the bias of the resistance, λ_R. These unknown factors are determined using the actual and predicted resistances, determined as explained previously, by computing the ratio of the actual resistance to the predicted resistance for each load test in the dataset. This value is referred to as the bias of each load test, λ, and essentially represents the magnitude of under or over-prediction between the true value of the pile resistance and the predicted value. Once this ratio is computed for each load test, the statistics of the ratio values, more specifically the average (i.e. mean), the standard deviation, and the coefficient of variation (COV) of the ratio values are computed. To that end, the mean of the ratio values is taken as the bias of the resistance, λ_R, while the COV of the ratio values is assumed as the COV of the resistance, Ω_R. The results of these statistical computations can be found in Table 8-1 to Table 8-4 for H-piles, timber piles, pipe piles, and precast concrete piles, respectively. The statistical computations can also be visually represented by the scatter plots shown in Figure 8-2 to Figure 8-5 for H-piles, timber piles, pipe piles, and precast concrete piles, respectively. Displaying the data in a scatter plot can be advantageous as it provides a clear visual representation of the spread of the data, the range of the data, and whether the data lies above or below the perfect correlation line. For most of the pile types, the scatter plots provide a clear indication that the number of data points used in the resistance factor calibration process is not statistically sufficient. This issue will be discussed in subsequent sections of this report.

The computed values of the bias of the resistance, λ_R, and the COV of the resistance, Ω_R, are simply inserted into Equation 8-1 in order to compute the value of the resistance factor, ϕ_R, for use in design. In the resistance factor calibration computations, the following values were assumed for the terms related to the load, based on recommendations provided in Baecher and Christian (2003) and Paikowsky et al. (2004):

λ_{QD} (bias of the dead load) = 1.05

λ_{QL} (bias of the live load) = 1.15

γ_D (dead load factor) = 1.25 (Strength I Limit State)

γ_L (live load factor) = 1.75 (Strength I Limit State)

Ω_{QD} (COV of the dead load) = 0.10

Ω_{QL} (COV of the live load) = 0.20

The ratio of the expected value of the dead load to the expected value of the live load, $E(Q_D)/E(Q_L)$, can range between 2 and 4 for a typical bridge structure. However, this value does not significantly affect the magnitude of the resistance factor in the computations (Baecher and Christian 2003) and was thus assumed as 2.0 for this project, which is consistent with the value used in the resistance factor calibrations in Paikowsky et al. (2004). The target reliability index, β_T, relates the probability of failure of the deep foundation to an expected performance level, as reported in Table 4-7. In deep foundation design, a β_T of between 2.0 and 3.5 is generally utilized (Kulhawy and Phoon 2006). For this project, a β_T of 3.0 was utilized, which is also consistent with the target reliability values reported for the Strength Limit State calibration in Paikowsky et al. (2004). For reference, a β_T of 3.0 corresponds to a probability of failure of approximately 0.1% (i.e. 1 out of 1,000 deep foundations will fail under the given combination of factored loads and resistance). The calibrated resistance factors are reported in the respective table of each pile type. For convenience, the calibrated resistance factors have also been converted into an equivalent factor of safety utilizing Equation 8-2 as provided in Paikowsky et al. (2004). This equivalent factor of safety is for visualization purposes only and is intended to provide the designer with an indication for the value of the calibrated resistance factor.

$$FS \cong \frac{1.4167}{\phi_R}$$

Equation 8-2

As discussed in Paikowsky et al. (2004), the value of the calibrated resistance factor is a complex combination of the bias of the design method (bias of the resistance), along with the variability within the method (COV of the resistance). Therefore, the value of the resistance factor alone does not necessarily provide a measure of the efficiency of the design method (i.e. a larger resistance factor does not always equate to a better design method). In recognition of this, it is desirable to present an efficiency factor, which is the ratio of the calibrated resistance factor to the bias of the resistance, or ϕ_R/λ_R, expressed as a percentage. The efficiency factor percentage is reported in each table. In general, the magnitude of the efficiency factor is greater for methods that predict the actual capacity of a deep foundation system more accurately, and thus more economically, regardless of the value of the calibrated resistance factor (i.e. the lower the "actual" factor of safety). However, it is important to note that general guidelines as to a minimum desired level of efficiency have not been reported in the literature (Paikowsky et al. 2004).

Interestingly, the 1971 report also contained a *field verified* resistance using both the South Dakota driving formula and the Engineering News Record (ENR) dynamic formula. These field verified resistances are based on rationally determining the ultimate capacity (nominal resistance) of the pile based on the penetrations of the pile and kinematics of the hammer energy (i.e. actual dynamic driving conditions at the site based on hammer weight, hammer efficiency, pile geometry, etc.), as outlined in Section 4.4.6.1. These dynamic capacity predictions were reported under the columns labeled S. Dak. (tons) and E.N.R.(tons) in Table A of the 1971 report for the SDDOT driving equation and Engineering News Record formula, respectively. Since this data was available to the research team, it was determined to calibrate the resistance factors for these dynamic field verification methods. However, as before, these formulas include a factor of safety that must be incorporated to compute an "ultimate" resistance. For the South Dakota driving formula, the factor of safety was again assumed as 4. For the ENR dynamic formula, Reese et al. (2006) reports that a factor of safety of 6 is included in the formula and therefore this factor of safety was incorporated to compute the "ultimate" resistance. After incorporating the respective factors of safety, the same resistance factor calibration approach as described above was utilized in these computations. In addition, an equivalent factor of safety, using Equation 8-2, and an efficiency factor were also computed for these methods and are summarized in Table 8-1 to Table 8-4.

Table 8-1: Statistical details of static and dynamic analyses of H-piles, calibrated resistance factors, equivalent factors of safety, and efficiency factors.

Site	Actual	Static Capacity Prediction (tons)		Dynamic Capacity Prediction (tons)				Resistance Factor Calibration		
		Test Hole	Test Hole (UM)	S.Dak.	S.Dak. (w/FS)	ENR	ENR (w/FS)	$\lambda_{Test\ Hole}$	$\lambda_{S.\ Dak.}$	λ_{ENR}
Madison (10x42)	105	27.6	110.2	19.8	79.2	29.8	178.5	0.95	1.33	0.59
Spearfish North (10x42)	75	19.0	76.0	9.8	39.32	11.5	69.2	0.99	1.91	1.08
Spearfish South (10x42 w/ barb)	75	19.0	76.0	20.8	83.12	25.0	150.0	0.99	0.90	0.50
Wendte (8x36)	70	25.6	102.5	34.5	137.8	43.9	263.4	0.68	0.51	0.27
Wendte (10x42)	85	23.7	94.8	36.7	146.8	45.4	272.4	0.90	0.58	0.31
Wendte (10x42 w/barb)	115	23.7	94.8	78.6	314.4	105.3	631.8	1.21	0.37	0.18
57th Street Sioux Falls (12x53)*	303	101.0	178.0		-	-	-	-	1.70	-
* Additional data point not included in Bump et al. (1971).							λ_R	1.06	0.93	0.49
							Ω_R	0.30	0.63	0.67
							φ_R	0.45	0.17	0.08
							FS_{eq}	3.15	8.33	17.71
							Φ_R/λ_R (%)	42.4	18.3	16.4

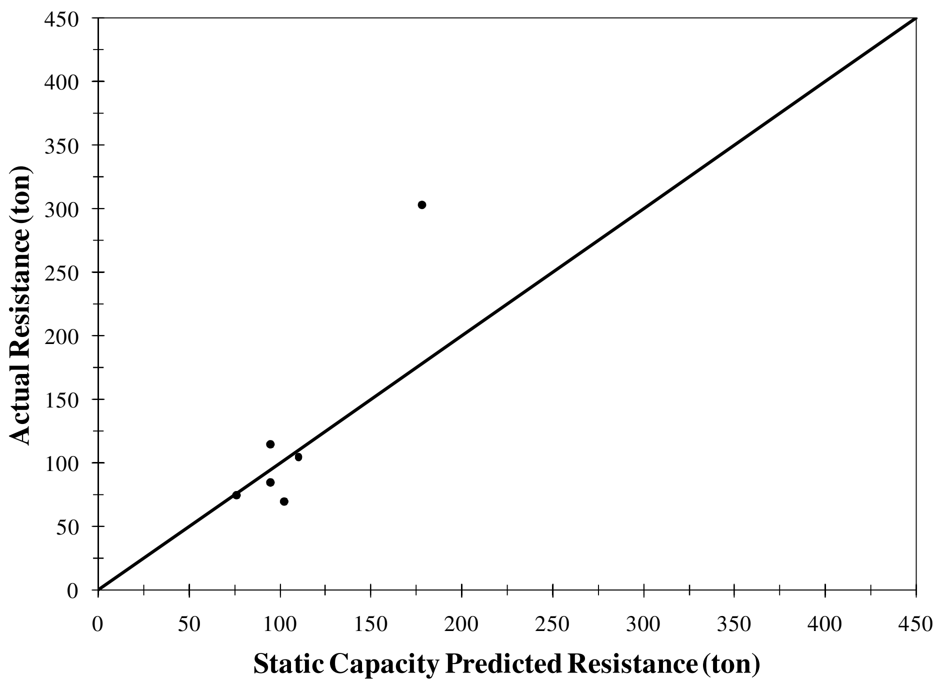

Figure 8-2: Scatter plot for 7 load test cases of H-piles in South Dakota geomaterials.

Table 8-2: Statistical details of static analyses of timber piles, calibrated resistance factors, equivalent factors of safety, and efficiency factors.

Site	Actual	Static Capacity Prediction (tons) Test Hole	Static Capacity Prediction (tons) Test Hole (UM)	Dynamic Capacity Prediction (tons) S.Dak.	Dynamic Capacity Prediction (tons) S.Dak. (w/FS)	Dynamic Capacity Prediction (tons) ENR	Dynamic Capacity Prediction (tons) ENR (w/FS)	Resistance Factor Calibration $\lambda_{Test\ Hole}$	Resistance Factor Calibration $\lambda_{S.\ Dak.}$	Resistance Factor Calibration λ_{ENR}
Madison	140	27.6	110.2	48.2	192.8	56.6	339.6	1.27	0.73	0.41
Watertown West	135	25.0	99.8	29.6	118.4	39.0	234.0	1.35	1.14	0.58
Watertown East *	91	26.3	105.4	22.0	88	28.8	172.8	0.86	1.03	0.53
Spearfish North	75	18.2	72.8	53.5	214	60.0	360.0	1.03	0.35	0.21
Spearfish South	58	18.9	75.6	22.5	90	25.0	150.0	0.77	0.64	0.39
Wendte	50	19.5	77.8	8.3	33.2	9.7	58.2	0.64	1.51	0.86
* Pile broke during driving.							λ_R	0.99	0.90	0.49
							Ω_R	0.29	0.46	0.44
							φ_R	0.43	0.25	0.14
							FS_{eq}	3.29	5.67	10.12
							Φ_R/λ_R (%)	43.5	27.8	28.3

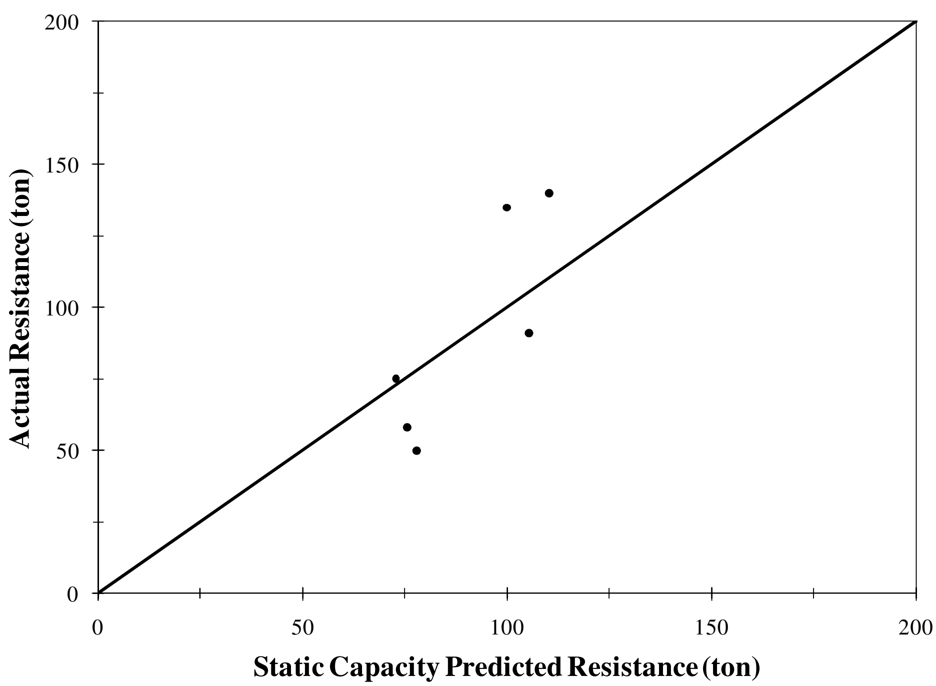

Figure 8-3: Scatter plot for 6 load test cases of timber piles in South Dakota geomaterials.

Table 8-3: Statistical details of static analyses of pipe piles, calibrated resistance factors, equivalent factors of safety, and efficiency factors.

Site	Actual	Static Capacity Prediction (tons) Test Hole	Static Capacity Prediction (tons) Test Hole (UM)	Dynamic Capacity Prediction (tons) S.Dak.	Dynamic Capacity Prediction (tons) S.Dak. (w/FS)	Dynamic Capacity Prediction (tons) ENR	Dynamic Capacity Prediction (tons) ENR (w/FS)	Resistance Factor Calibration $\lambda_{Test\ Hole}$	$\lambda_{S.\ Dak.}$	λ_{ENR}
Madison (12")	90	26.6	106.4	34.7	138.7	39.0	234.0	0.85	0.65	0.38
Watertown (12")[1]	165	26.3	105.4	73.5	294	88.5	531.0	1.57	0.56	0.31
Wendte (12")	55	20.4	81.6	10.3	41.2	11.6	69.6	0.67	1.33	0.79
Civic Center Rapid City (≈7")*	60	50.0	115.0	-	-	-	-	0.52	-	-
[1] Anchor pile broke during testing.							λ_R	0.90	0.85	0.50
* Additional data point not included in Bump et al. (1971).							Ω_R	0.51	0.50	0.52
							φ_R	0.22	0.21	0.12
							FS_{eq}	6.44	6.75	11.81
							Φ_R/λ_R (%)	24.4	24.8	24.2

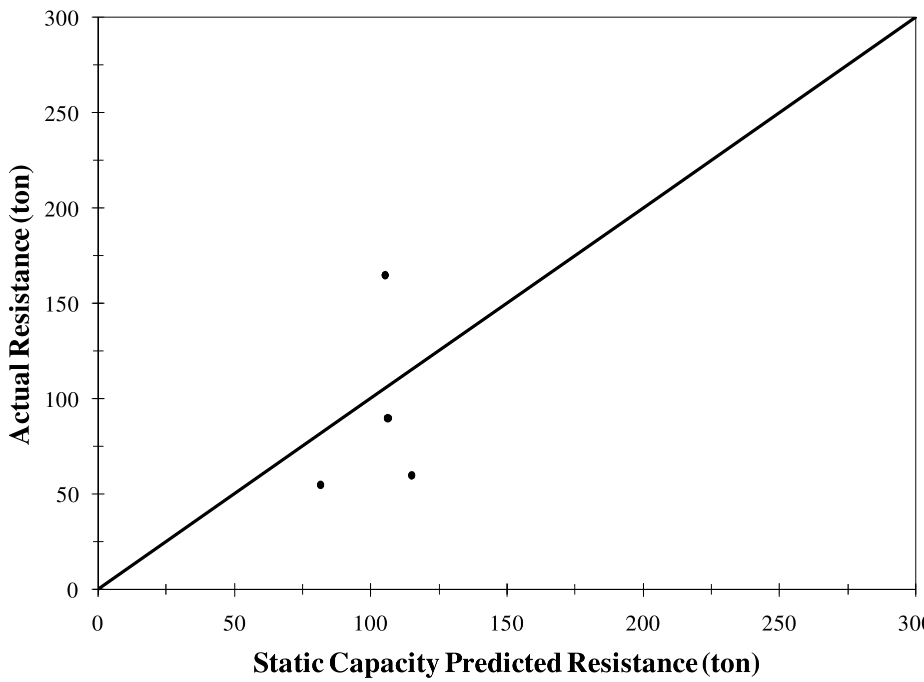

Figure 8-4: Scatter plot for 4 load test cases of pipe piles in South Dakota geomaterials.

Table 8-4: Statistical details of static and dynamic analyses of precast concrete piles, calibrated resistance factors, equivalent factors of safety, and efficiency factors.

Site	Actual	Static Capacity Prediction (tons) Test Hole	Static Capacity Prediction (tons) Test Hole (UM)	Static Capacity Prediction (tons) S.Dak.	Dynamic Capacity Prediction (tons) S.Dak. (w/FS)	Dynamic Capacity Prediction (tons) ENR	Dynamic Capacity Prediction (tons) ENR (w/FS)	Resistance Factor Calibration $\lambda_{Test\ Hole}$	Resistance Factor Calibration $\lambda_{S.\ Dak.}$	Resistance Factor Calibration λ_{ENR}
Madison (12"x14")	115	27.6	110.2	26.6	106.2	47.9	287.3	1.04	1.08	0.40
Madison (12"x14" tapered)	120	27.6	110.2	30.2	120.7	53.3	319.7	1.09	0.99	0.38
Watertown (14" octagonal)1	330	25.0	99.8	53.0	212.0	127.5	765.0	3.31	1.56	0.43
Spearfish North	90	22.4	89.6	28.4	113.4	37.5	225.0	1.00	0.79	0.40
Spearfish South	70	19.2	76.8	25.8	103.2	37.5	225.0	0.91	0.68	0.31
Beresford (14"x14")*	220	110.0	155.0	-	-	-	-	1.42	-	-
Cretex Rapid City (12"x12")*	367	110.0	230.0	-	-	-	-	1.60	-	-
							λ_R	1.48	1.02	0.38
							Ω_R	0.57	0.33	0.12
							φ_R	0.31	0.40	0.24
							FS_{eq}	4.57	3.54	5.90
							Φ_R/λ_R (%)	20.9	39.2	62.6

* Additional data point not included in Bump et al. (1971).

[1] Jack limit was achieved during load test.

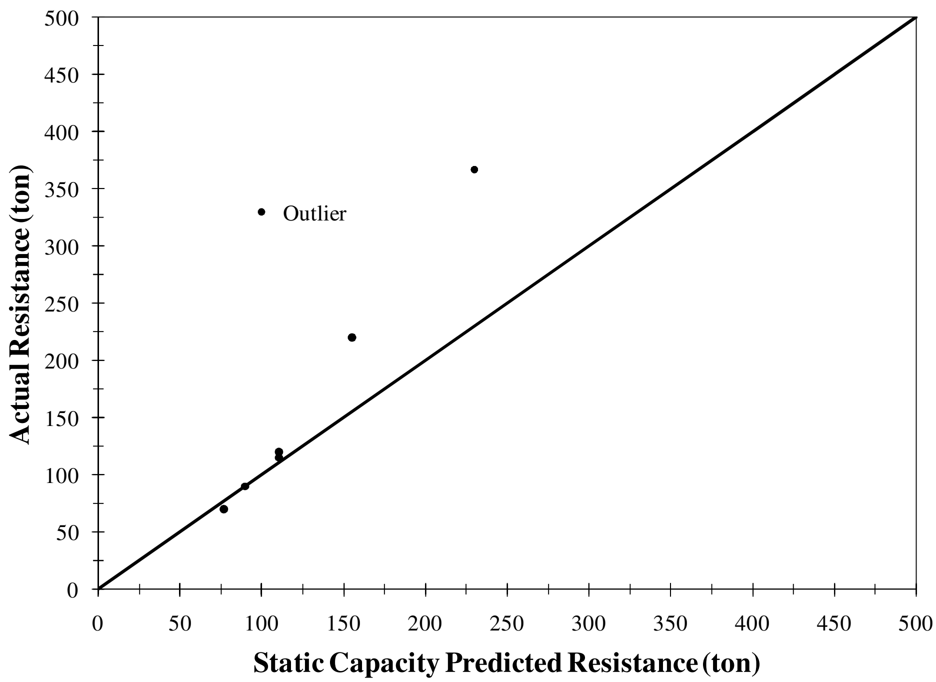

Figure 8-5: Scatter plot for 7 load test cases of precast concrete piles in South Dakota geomaterials.

8.2.2 *DRILLED SHAFTS*

The research team attempted to calibrate a geotechnical resistance factor for drilled shafts using load test data from the 1971 report and from a research project conducted in 1991-1992 by the SDDOT in cooperation with the University of Wyoming (Turner and Sandberg 1992). A report was issued from that project, hereinafter referred to as the "1992 report", and is discussed in detail in Section 5.6.2. However, after a thorough review of the available data, along with a review of the SDDOT drive test design method, it was determined that it was not possible to calibrate a resistance factor for use in the design of drilled shafts. The design methodology used in the 1971 report to compute the capacity of the drilled shafts is slightly different than the design methodology currently employed by the SDDOT. The current design methodology for drilled shafts appears to rely solely on the results of the pull test, as described in Section 5.3.2, while the design methodology employed in the 1971 report utilized a combination of the pull test for the side friction, along with a conventional static capacity approach for the end bearing. The calibration of resistance factors based on this data would have led to a significant inconsistency in the resistance factor calibration for the current design procedure.

For the research project conducted in cooperation with the University of Wyoming in 1991-1992, the SDDOT performed tension load tests on small concrete anchors with a nominal diameter equal to 6-inches or 8-inches. After a review of this data, which consisted of a data set of 18 tests, the calibration of resistance factors was not possible for a number of reasons. In the 1992 report, it is commented that the study "failed to generate a comprehensive data base upon which the reliability of the pull test could be firmly established" (Turner and Sandberg 1992). This creates a potential problem with attempting to utilize this data for resistance factor calibration. In addition, the diameter and length of the shafts tested for the 1992 report were substantially smaller than the diameter and length of typical drilled shafts for bridge structures. As discussed in Section 4.4.2.1, the side resistance of a deep foundation element is a function of a number of parameters including soil type, installation method, and foundation material, all which can have a direct effect on the horizontal effective stress acting along the foundation interface. A change in the magnitude of the horizontal effective stress can significantly affect the total resistance of the deep foundation element and it is possible that the magnitude of the horizontal effective stress along the interface of a drilled shaft that is of diameter and length of that typically installed for bridge structures will be significantly different than that for the small shafts tested for the 1992 report, even when installed in the same soils. This difference will be predominately due to the variances in the installation techniques for the various diameters of drilled shafts, which can create differing stress release conditions during installation. Since the side resistance of a drilled shaft, and thus the total resistance, can be sensitive to the diameter and length of the shaft, it was unknown how the nominal geotechnical resistance of a drilled shaft computed by the SDDOT drive test formula would correlate to a drilled shaft with a diameter equal to what is typically installed in the field for bridge projects. Extrapolation of the data in this manner should be avoided because the bias factor computed using the data from the 1992 report could be substantially different from the bias factor for a normal diameter drilled shaft with the resistance computed using the SDDOT pull test and thus the calibrated geotechnical resistance factor using the 1992 data would not reflect the true design conditions. This belief is further corroborated within the recommendations in the 1992 report, which strongly encourage the load testing of full-sized drilled shafts in order to provide final verification of the pull test method.

8.3 DISCUSSION

The calibration of geotechnical resistance factors for various types of driven piles designed using the SDDOT drive test predictive resistance formula has been conducted based on the reliability theory calibration method described in Section 4.5.2.2.2. A review of Table 8-1 to Table 8-4 provides these resistance factors under the "Resistance Factor Calibration" column for the SDDOT drive test predictive resistance formula (highlighted in the dark box). In design, these resistance factors would be applied to the predicted static nominal resistance computed using the drive test approach discussed in Section 5.3.1.1 and 5.3.1.2. For convenience, the calibrated resistance factors for the various pile types have been summarized below, after rounding to the nearest 0.05:

H-Piles: $\phi_R = 0.45$

Timber Piles: $\phi_R = 0.45$

Pipe Piles: $\phi_R = 0.20$

Precast Concrete: $\phi_R = 0.30$

Currently, the SDDOT is utilizing a resistance factor in design that has been determined by employing the calibration-to-fit method as described in Section 4.5.2.2.1 and 5.5.2. The value of this resistance factor for geotechnical resistance is 0.35 and is applied to all designs, regardless of pile or soil type. Interestingly, an examination of the calibrated resistance factors above indicates that in the case of pipe piles and precast concrete piles, the calibrated resistance factor is actually lower than the resistance factor determined using the calibration-to-fit method. For the pipe piles, this is likely the result of too few data points for a reliable calibration. For the precast concrete piles, it is apparent in Figure 8-5 that one of the data points could be considered an outlier, as the bias factor for that particular load test is significantly greater than the other load tests in the database. In fact, by eliminating the outlier in the data and performing a recalibration will result in a resistance factor, ϕ_R, of approximately 0.60, after rounding to the nearest 0.05, for the precast concrete piles as provided in Table 8-5. For reference, the magnitude of the resistance factors reported in AASHTO (2009) for the static analysis of driven piles under an axial compression load ranges from 0.25 to 0.50, depending on the design method employed.

Table 8-5: Statistical details of static and dynamic analyses of precast concrete piles, calibrated resistance factors, equivalent factors of safety, and efficiency factors with elimination of data outlier.

		Static Capacity Prediction (tons)		Dynamic Capacity Prediction (tons)				Resistance Factor Calibration		
Site	Actual	Test Hole	Test Hole (UM)	S.Dak.	S.Dak. (w/FS)	ENR	ENR (w/FS)	$\lambda_{Test\ Hole}$	$\lambda_{S.\ Dak.}$	λ_{ENR}
Madison (12"x14")	115	27.6	110.2	26.6	106.2	47.9	287.3	1.04	1.08	0.40
Madison (12"x14" tapered)	120	27.6	110.2	30.2	120.7	53.3	319.7	1.09	0.99	0.38
Spearfish North	90	22.4	89.6	28.4	113.4	37.5	225.0	1.00	0.79	0.40
Spearfish South	70	19.2	76.8	25.8	103.2	37.5	225.0	0.91	0.68	0.31
Beresford (14"x14")*	220	110.0	155.0	-	-	-	-	1.42	-	-
Cretex Rapid City (12"x12")*	367	110.0	230.0	-	-	-	-	1.60	-	-
* Additional data point not included in Bump et al. (1971).							λ_R	1.18	0.89	0.37
[1] Jack limit was achieved during load test.							Ω_R	0.23	0.21	0.11
							φ_R	0.60	0.47	0.24
							FS_{eq}	2.36	3.01	5.90
							Φ_R/λ_R (%)	51.0	53.0	64.6

The resistance factor calibration tables also provide an indication of the "efficiency" of the SDDOT drive test predictive resistance formula, along with an equivalent factor of safety, for each pile type. The efficiency within the drive test predictive resistance method is described using an efficiency factor that has been computed during the calibration process. Recall that the efficiency factor is the ratio of the calibrated resistance factor to the bias of the resistance and can provide a better understanding of how accurately a particular design method can be in predicting the actual resistance of a deep foundation system. In the instance of the SDDOT drive test predictive resistance formula, the efficiency factor and equivalent factor of safety was found to be approximately 42% and 3.15 for the H-piles, respectively, and approximately 44% and 3.29 for the timber piles, respectively. As with the calibrated resistance factors, the efficiency factor for the pipe piles and precast concrete piles is relatively low at 24% and 21%, respectively, thereby resulting in a large magnitude of the equivalent factor of safety of 6.44 and 4.57, respectively. However, when eliminating the outlier in the data for the precast concrete piles, it is observed that the efficiency factor increases to over 50% with a corresponding equivalent factor of safety equal to 2.36.

Resistance factors were also calibrated for the dynamic field verification methods used by the SDDOT. These methods include the SDDOT driving formula and the ENR formula. A field verified resistance using these formulas was included in the data within the 1971 report. The results of this calibration effort indicate relatively low values for the resistance factors for all types of piles, regardless of the field verification formula. The magnitude of the resistance factors range from 0.10 to 0.40, with efficiencies ranging from 16% to over 60%. The wide magnitude of resistance factor and efficiency values certainly indicates a significant level of variability within the field verification prediction methods. This is further supported by the magnitude of the COV of the resistance within a number of combinations of field verification methods and pile types. Overall, the quality of the data for these field verification methods is suspect and thus the calibrated resistance factors have been provided for information only. To that end, the SDDOT should examine other field verification methods for driven piles, particularly those discussed in Section 4.4.6, such as the Gates driving formula or Pile Dynamic Analyzer (PDA).

The design of driven piles must satisfy all applicable limit states, including those for serviceability and extreme loading, if applicable. In addition, for the Strength Limit State, the design of driven piles is based on ensuring that the factored load does not exceed both the factored structural resistance and the factored geotechnical resistance. The designer must first compute the factored structural resistance of the pile using the appropriate pile cross-sectional area, pile yield stress, concrete compressive stress, and resistance factor from the AASHTO LRFD Bridge Design Specifications (2009). The factored structural resistance is thus assumed to be equal to the factored load applied to the pile. For the SDDOT, the results of the drive test are incorporated into the SDDOT drive test predictive resistance approach as outlined in Section 5.3.1.1 and Section 5.3.1.2 to compute a nominal geotechnical resistance. For LRFD, the SDDOT must incorporate the Ultimate Method (UM) approach in order to compute the nominal geotechnical resistance. The computed nominal geotechnical resistance is then multiplied by the calibrated geotechnical resistance factor to obtain the factored geotechnical resistance. The factored geotechnical resistance must be greater than the factored structural resistance (i.e. factored load). If this is not the case, the length of the pile must be increased and the nominal geotechnical resistance of the pile recomputed using the UM approach. In general, for piles that will be driven to "practical refusal", generally defined as 0.05 to 0.25 inches/blow, the nominal geotechnical resistance will be greater than the nominal structural resistance and thus the structural resistance will control. The nominal geotechnical resistance of the piles should be verified in the field using a Pile Dynamic Analyzer (PDA) for quality control purposes.

Although calibration of resistance factors for driven piles was completed for this project, the application of these resistance factors in design must be executed using extreme caution. The resistance factors have been calibrated using a very small set of data, in some cases only 4 load tests were used in the process. Although the practice of calibrating resistance factors using a small quantity of load test data has been conducted by others for various DOT projects (Yang et al. 2008, Abu-Farsakh et al. 2009), it is advisable that the quantity of data be such that extrapolation is minimized while simultaneously achieving a given confidence level (Allen et al. 2005). Unfortunately, a review of the literature fails to provide a recommendation for the minimum number of load tests that should be conducted in order to perform resistance factor calibration. However, the load test data should represent a number of sources, pile types, soil conditions, driving and construction conditions, and any other conditions to adequately characterize the statistical variation within the design procedure. In addition, the statistics of the load test data should result in a confidence interval for the bias of the resistance, λ_R, which will not significantly affect the magnitude of the calibrated resistance factor. For example, the calibration of resistance factors for the H-piles in this project was based on 7 load tests. Utilizing the statistics of those load tests and assuming a 95% two-sided confidence interval, the value of λ_R for the prediction of the resistance of H-piles using the SDDOT driving formula ranges from 0.826 to 1.294. This range is quite significant and results in a wide range for the calibrated resistance factor. By tripling the number of load tests, however, the true value of the calibrated resistance factor is found to vary only slightly, assuming the statistics of the additional load tests are the same. Therefore, it is recommended that a minimum of 15-20 load tests should be conducted for each driven pile type to further develop the load test database. After supplementing the database, the new load test data should be assessed against the existing load test data to ensure statistical compatibility for recalibration of resistance factors for use in design.

The quality of the load test data is also very important in the calibration process. For this project, the load test database used in the calibration predominately consisted of data from the late 1960's and was supplemented with only four additional load tests. A number of assumptions were necessary in order to calibrate the resistance factors using the older load test data. For example, it was assumed that the predicted resistance values from the 1971 report were indeed computed using the SDDOT drive test predictive resistance method outlined in Section 5.3.1.1. The research team was generally able to verify these values using the drive test data to within ±20% of the published values. However, since "official" computations for these values were not located for this project, it is possible that the calibrated resistance factor values are invalid if a slightly different static capacity prediction approach was indeed employed to determine the predicted resistance values in the 1971 report. Secondly, since the late 1960's, there have been substantial technological advances in pile driving equipment efficiency and monitoring and it is likely that higher ultimate capacities are achievable with this equipment today. This introduces a possible inconsistency between the use of older load test data and the design of piles that are installed with newer pile driving technology. Lastly, it is beneficial in the calibration process to fully quantify the load transfer mechanism for a driven pile and to separate the total resistance into the side resistance and toe resistance. Once separated, calibration of resistance factors can occur for each component of the resistance since the uncertainty within the quantification of side and toe resistance can vary (Roberts and Misra 2010). To that end, it is highly recommended that the SDDOT conduct additional static load tests on driven piles, coupled with PDA and CAPWAP analyses, to further develop the load test and calibration database that was initiated for this project. The static load tests should be instrumented with strain gages and/or telltales in order to distinctly separate the load carrying mechanisms. Since the SDDOT drive test predictive resistance method does not explicitly distinguish between the side and toe resistance, significant design efficiencies may be gained by revising the SDDOT drive test method to separate the resistance components or by adopting alternative static capacity or soil-structure interaction analysis and design methods. As new load tests are added to the database, recalibration of the geotechnical resistance factors can occur. The new load test data should be assessed against the old data and, if necessary, it is recommended that the new load test data replace the 1971 report data in the database. In addition, the new load tests will provide the SDDOT with the ability to further assess the efficiency and reliability of the SDDOT drive test predictive resistance formula based on the results of the resistance factor calibration efforts and, if possible, to refine the approach to address the aforementioned issues.

As mentioned previously, it was not possible to perform resistance factor calibration for drilled shafts due to a lack of quality data, along with an inconsistency in the design methodology currently used by the SDDOT and past methods. Therefore, the SDDOT is strongly encouraged to conduct load tests on drilled shafts with a diameter equal to that typically used to support bridge structures in order to calibrate resistance factors for these systems. Due to their size (i.e. diameter and length), most drilled shaft load tests must utilize the O-Cell static load testing technology, as the size of a conventional static testing frame and anchor system can become prohibitively large and expensive. In addition, dynamic test methods, such as GRL's Apple system, are also being used quite extensively in the industry to perform load testing of drilled shaft systems. If the immediate use of geotechnical resistance factors calibrated using a reliability theory approach is desired, the SDDOT can make direct use of the calibrated resistance factors for drilled shafts reported in the AASHTO Bridge Specifications by utilizing one of the static capacity design methods discussed in Section 4.4.3 (AASHTO 2009).

8.4 SUMMARY

The calibration of geotechnical resistance factors for various types of driven piles has been conducted. It is observed that the magnitude of the calibrated resistance factors and efficiency of the SDDOT drive test predictive resistance formula is highly variable between different pile types. It was not possible to calibrate geotechnical resistance factors for drilled shafts due to the quality of the load test data. To that end, it is recommended that the SDDOT perform extensive load testing of both driven piles and drilled shafts in order to supplement the database developed for this project. The additional load testing will allow for recalibration of resistance factors as well as an increase in the efficiency of the SDDOT design method.

9 SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

9.1 SUMMARY

It is well-known by practicing engineers that uncertainty exists in design and that this uncertainty arises from numerous sources. Typical sources of uncertainty are inherent variability, measurement errors, transformation uncertainty, construction uncertainty and model error. In deep foundation engineering, nominal values for the pertinent soil properties and parameters are utilized in a prediction model to determine the ultimate capacity of the system. Design uncertainty is currently managed using an allowable stress design (ASD) approach, where either a global factor of safety is applied to the calculated ultimate capacity of the foundation or nominal safety factors are assigned to the individual components that comprise the total design (i.e. soil friction angle or cohesion). Regardless of the method used, safety factors are typically assigned based on an individual design engineer's comfort level and without quantification of actual design uncertainty magnitudes. This often leads to designs that are extremely conservative, and thus very inefficient, or designs that are unsafe.

A reliability-based design (RBD) addresses these issues by incorporating uncertainty in the design procedure and ultimately in the design process for deep foundations. In an RBD procedure, the magnitude of design uncertainty can be rationally incorporated within the overall design process, thereby resulting in a safe, efficient, and consistent design approach. An RBD procedure based on the Load and Resistance Factor Design (LRFD) approach is becoming increasingly utilized in the design of geotechnical structures, such as deep foundations (AASHTO 2009). In fact, the Federal Highway Administration (FHWA) has mandated the use of the LRFD methodology for all federally funded transportation infrastructure projects starting in October 2007. In the LRFD method, resistance factors are applied to the calculated ultimate resistance of the foundation, while load factors are applied to the calculated loads on the foundation. Load and resistance factors are determined using a probabilistic approach to ensure that design uncertainties are explicitly integrated and quantified in the design. In addition, resistance factors can be calibrated to produce designs that consistently achieve a desired level of reliability. The application of load and resistance factors is thus intended to replace the global factor of safety in the design process at both the strength and service limit states.

This research project examined the SDDOT's current design methodologies relative to deep foundation support of bridges and gathered data in an attempt to generate state specific resistance factors for LRFD deep foundation design. Given the SDDOT uses a unique method in determining the strength limit state of axial pile capacity, current default values in the AASHTO specifications cannot be used in the design of pile foundations. The first phase of this research examined documentation available in the literature as well as documentation provided by the SDDOT to determine the basis for current design relative to LRFD implementation. That documentation included the AASHTO Bridge Design Specifications, FHWA published design guidance, past research projects initiated by the SDDOT and internal design guidance documentation. As part of this discovery phase, the researchers also contacted surrounding states to discern the geotechnical process by which they have implemented LRFD requirements. A standardized list of questions was generated and the information was gathered via telephone interviews.

The second phase of this project was designed around assisting the SDDOT in transitioning to a full LRFD design platform. The SDDOT currently uses an interim "calibration-to-fit" method of determining resistance factors for LRFD design. The disadvantage to this method is that it does not use state specific data in determining resistance factors that account for regional geotechnical conditions. Furthermore, the calibration-to-fit method is essentially the ASD method of determining pile capacity (i.e. strength limit state). As part of this research, the research team collected field test data from the SDDOT, FHWA, regional and local geotechnical consulting firms, and specialty contractors to construct a database for the purposes of determining resistance factors for LRFD design. Field test data consisted of full scale load test data, dynamic field test data, driving records, and site explorations.

The database consisted of assessing the quality and character of the data relative to the ability to account for uncertainly in the variables that are used in deep foundation design. Once data was deemed suitable for inclusion in the database, a probabilistic analysis of the data was performed to calibrate resistance factors specific to South Dakota. Although a number of static load tests and dynamic capacity analyses for different types of deep foundation systems were collected, only a limited number of these load test data could be utilized in the resistance factor calibration process. For this project, resistance factors were calibrated for several different types of driven piles; however, a sufficient quantity of data was not available for a strong probabilistic analysis. In addition, the quality of the data could be improved by supplementing the database with new data that reflects current construction methodologies. Therefore, recommendations are made regarding advancing a load test and pile drivability database for calibration of resistance factors in the future.

9.2 CONCLUSIONS

The results of this study can be summarized in the following main conclusions:

1. The SDDOT uses a unique design method in predicting the axial compressive capacity of bridge pile and drilled shaft foundations. Based on past research, the method has promise, but sufficient data does not exist to quantify its efficiency and reliability.
2. Documentation does not exist that quantifies the built-in factor of safety of the current design method. This could lead to cost inefficiencies due to overdesign or potential losses in the case where the predicted resistance is greater than the actual resistance.
3. The SDDOT's current method of incorporating LRFD into bridge design appears to be adequate based on FHWA's communication, provided that the SDDOT fully implements the LRFD platform by continuing the process of determining local resistance factors.
4. Resistance factors were calibrated for the SDDOT drive test predictive resistance design method for several different types of driven pile systems. However, the quantity of the load test data used in those calibrations is relatively small and the quality of the data should ideally reflect current construction methods and conditions.

9.3 RECOMMENDATIONS

Based on the results of this study, the following recommendations are made:

1. A research project should be instituted with the specific purpose of verifying and improving the efficiency of the SDDOT's current drive test predictive resistance design method. This research project was not provided documentation that completely supports the method or documents the basis of the process or parameters. This recommendation would require that extensive collection of load test data be instituted on full-scale driven piles and drilled shafts installed in the field. The data collection would include multiple test sites and would include static load testing and high strain dynamic pile testing using a Pile Driving Analyzer (PDA) and the CAse Pile Wave Analysis Program (CAPWAP). The static load testing would serve as a means to verify the results of the PDA and CAPWAP analyses, which would potentially lead to the widespread adoption of the PDA and CAPWAP technologies in future field verification and load testing programs. The load testing should incorporate instrumentation, such as strain gages or telltales, which can provide a distinct separation of the side and toe resistance components of driven piles and drilled shafts. The separation of the resistance components in the load testing may allow the SDDOT to

refine the drive test predictive resistance method with respect to side and toe resistance since the design method includes both a drive component and pullout component. The load test data may also result in the adoption of other design methods, such as those reported AASHTO (2009), or a soil-structure interaction method, such as the t-z model method, in order to further increase design efficiency and reliability.

2. The SDDOT should perform recalibration of the resistance factors that were developed for this project. The conduction of additional field load tests on driven piles and drilled shafts will allow for further development of the load test database initiated by this project and therefore subsequent recalibration of geotechnical resistance factors for use in design. For resistance factor calibration, 15 to 20 field load tests should ideally be included in the database for each driven pile type and for drilled shafts in order to ensure adequate statistical representation of the SDDOT's drive test predictive resistance design approach. Since an initial load test database was developed for this project in order to calibrate resistance factors for driven piles, the SDDOT can easily add the additional load test results to the database and perform the recalibration exercise. As new load test data becomes available, it is recommended that the new data be assessed with respect to the old data and that the old data be replaced since the new load test data will reflect current construction methods and technologies.
3. The SDDOT should address other limit states with respect to LRFD design, especially the serviceability limit state. Design methodologies for the serviceability limit state can be adopted through the AASHTO LRFD Bridge Specifications. The serviceability limit state can be critical to the performance of bridge structures, especially with respect to maintenance and functionality. In addition, the AASHTO LRFD Bridge Design Specifications require the verification of all deep foundation designs at the serviceability limit state.
4. The SDDOT should formally address lateral pile performance in their designs per the design methodologies provided in the AASHTO LRFD Bridge Specifications or through the adoption of the LPILE software, which is widely utilized in the industry.
5. The SDDOT should use the rated steel strength for driven piles in their designs.
6. The SDDOT should formally address group effects in their designs per the design methodologies provided in the AASHTO LRFD Bridge Specifications.
7. The SDDOT should review their method for field sampling, especially when undisturbed soil samples are desired for laboratory testing such as for the consolidation test, unconfined compressive strength test, direct shear test, and triaxial shear test.
8. The SDDOT should update their driving formula for field verification of pile capacity. Current and past research has shown that more reliable field verification formulae exist than what is currently used by the SDDOT.
9. The SDDOT should perform driveability analyses, such as using the GRLWEAP software, as part of their designs in the future.

9.4 IMPLEMENTATION PLAN

This study has shown that enough information does not currently exist to either verify the current SDDOT's design method for the strength limit state of axial pile capacity or to reliably determine resistance factors from a constructed database. Furthermore, other potential improvements in the design process have been identified during the course of this research project. Therefore, the following implementation plan is recommended:

1. Address other aspects of design relative to Load and Resistance Factor Design (LRFD).

  a. Settlement
    i. Conduct serviceability limit state design using the methodologies provided in AASHTO (2009) or using a soil-structure interaction technique such as the t-z model method.
    ii. Ensure compatibility between tolerable and serviceability settlement.

2. Implement the missing pieces in current design.
   a. Conduct driveability analyses using improved field verification formulas or GRLWEAP.
   b. Incorporate lateral load analysis components in design using the methodologies provided in AASHTO (2009).
   c. Address group effects using the methodologies provided in AASHTO (2009).
   d. Per AASHTO (2009), the LRFD approach is not fully implemented until all limit states have been addressed in the design process.
3. Develop a plan to collect field load test data to assess the current SDDOT drive test predictive resistance approach and to develop a database of state data for further calibration of resistance factors for LRFD.
   a. Perform PDA/CAPWAP analyses on current and future projects.
      i. Through research projects, and
      ii. Using consultants, or
      iii. Buy the equipment and develop the skills internally.
   b. Develop a broad, yet extensive program to perform field load testing.
      i. Through research projects.
      ii. As part of construction.
4. Follow through with assessing current method for inefficiencies.
   a. Use the collected field load test data to identify the actual deep foundation resistance and settlement performance and re-evaluate the current design method.
   b. Determine the built-in factor of safety.
   c. Concurrently evaluate other design methodologies, such as those provided in AASHTO (2009) or other approaches such as the t-z model method, to see if another methodology is more efficient in predicting load-settlement behavior.
5. Once sufficient field load test data is collected that is compatible with the SDDOT drive test predictive resistance method, perform resistance factor calibration as appropriate.
   a. SDDOT needs to decide how resistance factor calibration will be performed relative to their goals.
6. In accordance with LRFD principles, resistance factor calibration and assessment of design methods is an ongoing process.
   a. Develop a process to continually collect field load test data.
   b. Periodically re-evaluate design procedures.
   c. Periodically recalibrate to see if continuing efficiencies can be gained in resistance factors.

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APPENDIX A: DOCUMENTS PROVIDED BY SDDOT

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APPENDIX B: TABLE A FROM BUMP ET AL. (1971)

SPEARFISH SITE DATE: 1. 12-67, 2. 2-68, 3. 9-68

PILE TYPE	LENGTH IN PLACE (Ft.)	DRIVEN WEIGHT (LBS.)	"S" (In.)	FORUMULA BEARING (Tons)			ULT. BEAR. (Tons.)	YIELD POINT (Tons)	DATE
				S. DAK.	E.N.R.	TEST HOLE			
* NORTH TIMBER	18.5	870	0.075	53.5	60.0	18.2	75	60-65	1
							65-70	60	2
							60-65	45-55	3
SOUTH TIMBER	20.5	766	0.5	22.5	25.0	18.9	45-50	40-45	1
							45-58	30-35	2
							45-48	30-35	3
NORTH 10 BP 42	30	1260		9.83	11.54	19.0	60-64	45-50	1
							65-68	45-55	2
							75	60-65	3
SOUTH -BARBED 10 BP 42	30	1470		20.78	25.0	19.0	50-55	30-40	1
							60-65	45	2
							75	60-65	3
NORTH PRECAST CONCRETE	14	4325	0.3	28.35	37.5	22.4	90	75	1
							90	70-75	2
							65-70	50-55	3
SOUTH PRECAST CONCRETE	20	4393	0.3	25.8	37.5	19.2	60-65	45-50	1
							60-70	45-50	2
							65-70	45-50	3

WENDTE SITE DATE: 1. 1-68, 2. 5-68, 3. 7-68, 4. 10-68

PILE TYPE	LENGTH IN PLACE (Ft.)	DRIVEN WEIGHT (LBS.)	"S" (In.)	FORUMULA BEARING (Tons)			ULT. BEAR. (Tons.)	YIELD POINT (Tons)	DATE
				S. DAK.	E.N.R.	TEST HOLE			
TIMBER	23.25	950	1.2	8.3	9.7	19.45	45-50	40-45	2
							40-45	30-35	3
							40-45	30-35	4
8 BP 36	44.9	1642	0.4	34.45	43.9	25.62	70	60-65	2
							70	55-60	3
							60-70	55-60	4
** 10 BP 42	28	1176	**	**		21.03	30	25	2
	43.8	2357	0.375	36.7	45.4	23.7	75-80	60-65	3
							75-85	60-70	4
10 BP 42 WITH BARB	35	1700	0.375	36.6	46.2	19.6	65-75	55-60	2
	43.6	2282	0.1375	78.6	105.3	23.7	110-115	80-90	2
							100-105	80-90	3
							95-100	75-85	4
12" PIPE	25	678	1.08	10.3	11.6	20.4	50-55	40-48	2
							45	30-35	3
							40-44	30-35	4
18" DIA. CAST IN PLACE	20.4	WEIGHT	S. F.	END BEAR.		TOTAL	145	115	1
		5404	61.9	15.83		75.25	105	92.5	2
							100-105	91	3

*See site notes the recorded formula bearing appears to be too high.

** Drove too fast and easy to be recorded.

MADISON SITE DATE: 1. 5-67, 2. 6-67, 3. 9-67

PILE TYPE	LENGTH IN PLACE (Ft.)	DRIVEN WEIGHT (LBS.)	"S" (In.)	FORUMULA BEARING (Tons)			ULT. BEAR. (Tons.)	YIELD POINT (Tons)	DATE
				S. DAK.	E.N.R.	TEST HOLE			
TIMBER	26	1130	0.2125	48.2	56.6	27.56	90-91	60-70	1
							120-122	85-95	2
							135-140	120-130	3
10 BP 42	32.5	1365	0.125	19.8	29.75	27.56	65	43-50	1
							90-93	73-83	2
							102-105	85-95	3
12" PIPE	34	762	0.400	34.67	39.0	26.6	70	60-63	1
							85-88	70-80	2
							88-90	70-80	3
12" X 14" PRECAST CONCRETE	26.5	5916	0.325	26.56	47.88	27.56	80-85	58-65	1
							92-97	75-85	2
							115	90-100	3
12" X 14" TAPERED CONCRETE	26	5635	0.25	30.17	53.29	27.56	82-85	58-65	1
							95-99	80-90	2
							120	90-100	3

WATERTOWN SITE DATE: 1. 9-67, 2. 11-67, 3. 10-68

PILE TYPE	LENGTH IN PLACE (Ft.)	DRIVEN WEIGHT (LBS.)	"S" (In.)	FORUMULA BEARING (Tons)			ULT. BEAR. (Tons.)	YIELD POINT (Tons)	DATE
				S. DAK.	E.N.R.	TEST HOLE			
WEST TIMBER 30' PREBORED	28.5	2223	0.400	29.6	39.0	24.95	135	75-80	1
							135	80-90	2
							135	85-95	3
* EAST TIMBER 30' PREBORED	29.5	2261	0.375	22	28.8	26.34	91	55-75	1
							85-90	55-65	2
							65-68	50-60	3
** 12" PIPE 30' PREBORED	29.5	1356	0.100	73.54	88.5	26.34	150	135-145	1
							165	150-160	2
							150+	150+	3
*** 14" OCTAGONAL CONCRETE 30' PREBORED	32	10475	0.075	53.0	127.5	24.95	150+	150+	1
							200+	200+	2
							330+	270-280	2

* Pile broke during driving.

** Anchor pile broke @ 153 ton on 3rd loading.

*** (1) Anchor pile broke @ 150 ton on 1st test.
(2) 200 ton limit of Jack on 2nd test.
(3) 330 ton limit on Jack on 3rd test.

OKATON SITE DATE: 1. 8-68, 2. 9-68, 3. 11-68

PILE TYPE	LENGTH IN PLACE (Ft.)	DRIVEN WEIGHT (LBS.)	FORUMULA BEARING (Tons)			ULT. BEAR. (Tons.)	YIELD POINT (Tons)	DATE
			S.F.B	END BEAR.	TOTAL			
18 IN. DIA. CAST IN PLACE	20.0	5300	58.07	31.07	86.64	45-50	24	1
						60-65	49	2
						73-75	55	3

WALL SITE DATE: 1. 8-68, 2. 9-68, 3. 11-68

PILE TYPE	LENGTH IN PLACE	DRIVEN WEIGHT	FORUMULA BEARING (Tons)			ULT. BEAR.	YIELD POINT	DATE
			S.F.B	END BEAR.	TOTAL			
18 IN. DIA. CAST IN PLACE	20.5	5433	91.06	56.57	145.13	120-125	75	1
						135	110	2
						150	123	3

PIERRE SITE DATE: 1. 10-65, 2. 7-67, 3. 10-68, 4. 11-68, 5. 2-69, 6. 11-69

PILE TYPE	LENGTH IN PLACE (Ft.)	DRIVEN WEIGHT (LBS.)	FORUMULA BEARING (Tons)			ULT. BEAR. (Tons.)	YIELD POINT (Tons)	DATE
			S.F.B	END BEAR.	TOTAL*			
18 IN. DIA. WEST CAST IN PLACE	10.5	2650	38.57	21.25	58.57	40-45	24.5	3
						45-50	35.5	4
						60	36	5
18 IN. DIA. EAST CAST IN PLACE	19.7	5220	84.17	39.90	121.57	165-170	113	3
						165-170	120	4
						165-175	125	5
8 IN. DIA. CAST IN PLACE	10	525	19.20	6.27	25.21	16	12	1
						21-22	19	2
						23	19	6
8 IN. DIA. CAST IN PLACE-VOID	10	525	27.02	0.0	26.77	13-14	11	1
						16-17	14	2
						19	16	6
4½ IN. DIA. CAST IN PLACE	10	166	12.26	2.08	14.24	16	13	2
						17-18	14.5	6
4½ IN. DIA. CAST IN PLACE-VOID	10	166	10.80	0.0	10.70	17-18	15.5	2
						17-18	13	6

S.F.B. = Calculated Skin Friction Bearing.
END BEAR. = Calculated End Bearing.
*Note weight of pile has been subtracted.