Probabilistic Based Assessment of the Influence of Nonlinear Soil Behavior and Stratification on the Performance of Laterally Loaded Drilled Pier Foundations

by

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ABSTRACT

This thesis presents a probabilistic evaluation of multiple laterally loaded drilled pier foundation design approaches using extensive data from a geotechnical investigation for a high voltage electric transmission line. A series of Monte Carlo simulations provide insight about the computed level of reliability considering site standard penetration test blow count value variability alone (i.e., assuming all other aspects of the design problem do not contribute error or bias). Evaluated methods include Eurocode 7 Geotechnical Design procedures, the Federal Highway Administration drilled shaft LRFD design method, the Electric Power Research Institute transmission foundation design procedure and a site specific variability based approach previously suggested by the author of this thesis and others.

The analysis method is defined by three phases:

a) Evaluate the spatial variability of an existing subsurface database.
b) Derive theoretical foundation designs from the database in accordance with the various design methods identified.
c) Conduct Monti Carlo Simulations to compute the reliability of the theoretical foundation designs.

Over several decades, reliability-based foundation design (RBD) methods have been developed and implemented to varying degrees for buildings, bridges, electric systems and other structures. In recent years, an effort has been made by researchers, professional societies and other standard-developing organizations to publish design guidelines, manuals and standards concerning RBD for foundations. Most of these approaches rely on statistical methods for quantifying load and resistance probability distribution functions with defined

reliability levels. However, each varies with regard to the influence of site-specific variability on resistance. An examination of the influence of site-specific variability is required to provide direction for incorporating the concept into practical RBD design methods.

Recent surveys of transmission line engineers by the Electric Power Research Institute (EPRI) demonstrate RBD methods for the design of transmission line foundations have not been widely adopted. In the absence of a unifying design document with established reliability goals, transmission line foundations have historically performed very well, with relatively few failures. However, such a track record with no set reliability goals suggests, at least in some cases, a financial premium has likely been paid.

DEDICATION

For my loving and ever-patient wife.

ACKNOWLEDGMENTS

I owe a debt of gratitude to my committee chair, Dr. Sandra Houston, for her continuing advice, patience and confidence in me over the protracted effort which produced this work. She most certainly doesn't remember our first interaction which simultaneously sparked my interest in the geotechnical field and solidified my belief that I was headed in the right direction.

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I would also like to especially thank my friend, colleague and mentor, Peter Kandaris for the opportunity to work with and learn from him. Peter has been instrumental in my progress as a student and as a professional. For the formative role he has played in my life, I will always be grateful.

As students, we all find inspiration in those who dedicate their lives to the education of others, both in the classroom and the office next door. I consider myself exceptionally fortunate to have found the individuals noted here to help me along in both.

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1 BACKGROUND

Although Reliability Based Design (RBD) procedures exist for laterally loaded transmission line foundations, their adoption by practitioners has been limited. Designers generally show a reluctance to use these methods because they are not well understood, different than allowable stress design methods, seem new and untested, and are perceived as difficult to apply. Furthermore, for applications relevant to electric system foundations, a sufficiently robust database to implement full RBD does not necessarily exist (Brown, Turner, & Castelli, 2010). However, simplified methods are available. Application of a single resistance factor to the load model (typically based on soil type) is standardized in US highway bridge foundation design (AASHTO, 2012). Recent improvements in electric transmission design software incorporate a similar single resistance factor approach (DiGioia Gray and Associates, 2012). More elaborate partial factor approaches are seen in international codes, with various countries making strides to simplify the process. However, to a large degree, these methodologies are calibrated to achieve similar reliability to existing Allowable Stress Design (ASD) methods which only represents the first step in the progression toward full implementation of RBD.

ASD relies on application of a global factor of safety to achieve an acceptable margin against adverse performance in recognition of inherent uncertainties in foundation loads and resistance. The ASD factor of safety approach has been employed successfully, in terms of acceptably low rates of failure, over the history of the geotechnical engineering profession. However, the factor of safety is a value calibrated from an empirical observation of failure rates achieved in practice. This form of calibration is performed in the absence of a rational quantification of the design uncertainties which contribute to failures and is therefore prone to high levels of conservatism (Allen, 2005).

Conceptually, increasing the factor of safety similarly increases reliability, where reliability is defined by Eq. 1.1. However, the relationship between reliability and the factor of safety is not linear (Fig. 1.1). Increasingly high factors of safety only marginally increase reliability to an asymptotic maximum value of 1 (i.e. 0% probability of failure, which is not possible). Therefore, on a conceptual basis it can be seen that an optimum level exists where further increases to the factor of safety yield only limited improvements in reliability which is to the economic detriment of the design.

$$R = 1 - p_f \qquad \text{Eq. 1.1}$$

Where:

p_f = Probability of failure

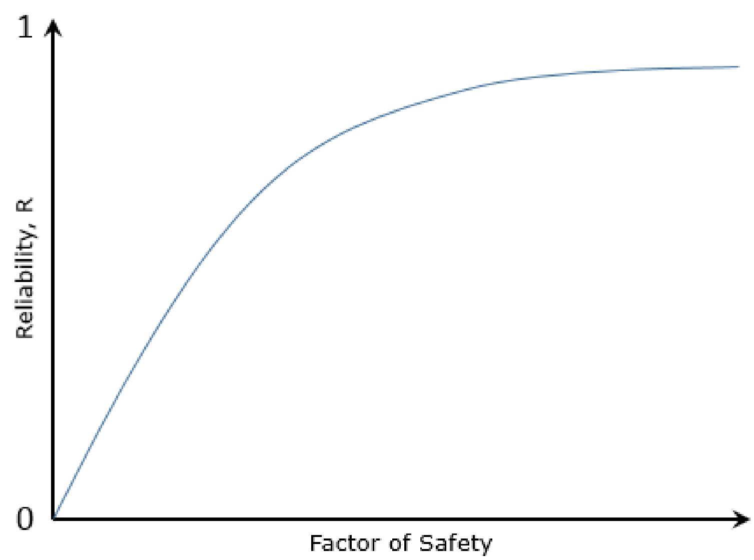


Figure 1.1 - Relationship between Factor of Safety and Reliability

The goal of RBD is to employ a rational assessment of each discrete source of uncertainty in the design model to derive a solution which has both an acceptable level of reliability and an acceptable level of economy. Computation of reliability is performed through derivation of demand (load, Q) and capacity (resistance, R) probability density functions which are representative of the net uncertainties present within each value (Fig. 1.2). The probability of failure, p_f, is represented by the region where the demand function is greater than the capacity.

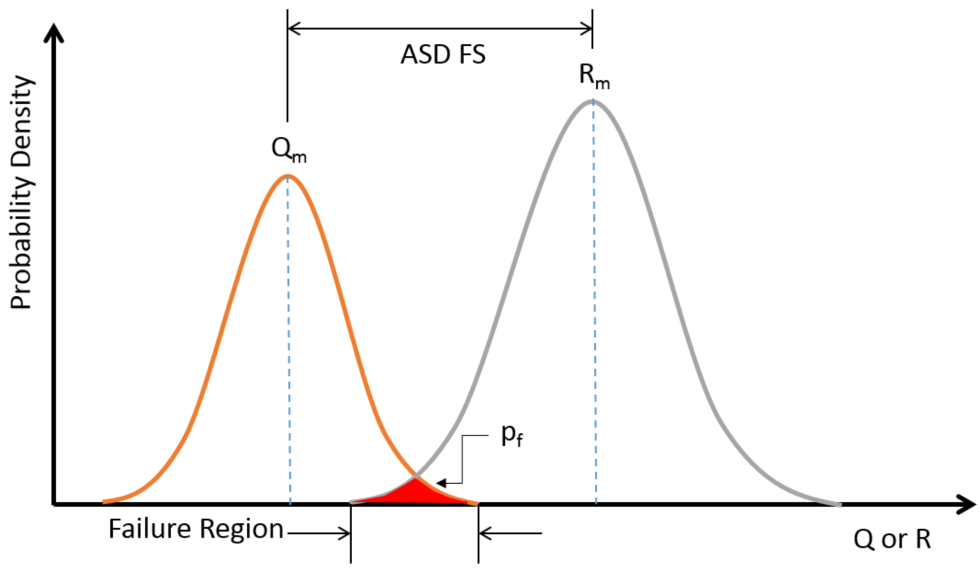


Figure 1.2 - Conceptual basis of RBD

While quantification of uncertainty provided by the RBD approach is valuable, it is not illustrative of the true advantage of RBD. Assessment of uncertainty to derive the probability of failure in design must be paired with a rational evaluation of what an appropriate probability of failure should be. Early implementation of RBD is generally formulated to derive similar levels of reliability to existing ASD practices in order to achieve continuity amongst both approaches. This is the case for the design methods evaluated within this thesis. Although important to the overall progression toward RBD, matching existing ASD results is not actual reliability base design because the selected level of reliability remains an empirically based value.

Ultimately, further refinements to RBD methodologies on the basis of an optimized assessment of reliability in comparison to cost are required to realize the full benefits of RBD.

Full implementation of RBD in a manner which will derive the full benefits of the approach requires significant data and computational effort. Most electric transmission line foundation designs are still performed via allowable stress methods for a variety of reasons. Phoon, Kulhawy and Grigoriu (1995) categorize designer reluctance with regard to RBD into three general classes:

(a) Relevance of using statistics to model soil property variability: Classifying soils statistically is difficult because of non-uniform populations/soil heterogeneity, insufficient data to define probability distributions, upper and lower bound soil properties not being adequately characterized by a mean and variance, and soil properties generated by statistics may not exist in nature.

(b) Unwarranted sophisticated computational treatment due to insufficient statistical information for complex calculations, greater risk of computational error, reducing soils evaluations to mere mathematical exercises that divert engineers from understanding of the real physical/chemical/mechanical processes.

(c) Difficulty in interpreting the theoretical probability of failure and usefulness in design since the theoretical probability of failure may not equal actual probability of failure since other important sources of uncertainty are not included in the analyses. Disagreement on the definition of failure and the desired probability of occurrence further complicates the issue.

Phoon, et al., note that all of the listed challenges can be overcome by use of appropriate statistical methods, judicious use of rational design methods and understanding that probabilistic methods have the capability to advance the profession beyond design with arbitrary factors of safety. However, to do so, the profession must look toward more rational ways to manage risk in order to derive greater economy in the foundation design process.

Efforts in establishing foundation RBD are slow to gain acceptance by electric system practitioners. Many question why RBD methods do not directly account for sample, soil and test variability. Current methods employed to develop subsurface strength factors for specific foundation models on the basis of nominal soil parameters derived from a high quality dataset. In comparison, low quality data is often supplemented by engineering judgment in practice and a tendency to use lower bound parameters in lieu of nominal parameters exists. The result is incompatibility amongst the parameter selection process and the design model which can yield overly conservative (expensive) designs.

Electrical transmission lines traverse large distances and cross widely varying geologic and geotechnical settings over many miles, creating difficulty in generating valid statistical characterization of the subsurface. Because of the breadth of geologic and geotechnical conditions encountered in electric transmission line projects, design investigations generally lack extensive data within specific geotechnical strata. Empirical correlations to subsurface properties are used extensively in transmission line design for selecting geotechnical parameters. Potentially wide data variability is intrinsic to the foundation design process.

For these reasons, RBD offers substantial opportunities to produce more reliable and cost effective foundation designs in the electric utility industry. Whatever the RBD methodology, the approach must be consistent. Target reliability, resistance factor calibration, data handling and model calculation must be consistent from site to site and foundation to foundation.

2 PURPOSE

In consideration of the historically very high reliability of transmission line foundations and recent surveys indicating the state of practice in their design, there is reason to believe room for economization exists. This paper will examine the existing state of practice for transmission line foundation design, available RBD guidelines for foundation design and methods previously proposed (Heim, Kandaris, & Houston, 2011) to evaluate the mechanisms influencing reliability in laterally loaded drilled piers. The study does not result in a recommended RBD method, but rather explores various aspects of RBD methods currently used in practice with an emphasis on the impact soil stratification has on calculated foundation reliability in consideration of varying design methodologies.

The non-linear deflection response of short, rigid laterally loaded piers is similarly explored on the basis of reliability implications. Toward this aspect of the foundation load response, a supplemental limit state evaluation considering load/deflection performance is proposed to augment RBD methodologies where service limit criteria govern design.

3 TRANSMISSION LINE DESIGN

Transmission lines transmit electric power over large distances from power generation facilities to regions of power consumption where voltage is reduced and energy is distributed locally to consumers. The long distances that transmission lines traverse present unique challenges for engineers because the terrain, geotechnical, meteorological and regulatory settings can vary widely over the length of a given project.

Transmission structures generally support a small number of circuits, 1 or 2 typically, with each circuit comprised of three sets of energized 'phase' conductors and one de-energized 'static' conductor for lightning protection and grounding (Fig. 3.1.1). Typical structures support spans of conductors ranging from 600 ft to 1700 ft or greater dependent upon terrain, Right of Way (ROW) width and loading among other considerations. The focus of this paper is transmission lines classified as High-Voltage (110kV – 345kV) and Extra High Voltage (345kV and greater) where kV = 1,000 volts. With increasing voltage, structure and foundation loading generally increases due to a number of factors. For operational and safety reasons, as circuit voltage increases, both the spacing between phases and the required clearance to adjacent features (the ground, buildings, etc.) increase significantly, leading to larger structures in terms of height and girth. Similarly, with increasing voltage, the size and number of conductors present within each phase generally increases, yielding higher loads imposed on the supporting structures and foundations.

The selection of a design span length is an optimization procedure aimed at finding the appropriate balance between short spans with a larger number of less expensive structures and longer spans with a smaller number of more expensive structures. While optimal span length generally increases with voltage, the appropriate value for a given project is highly dependent upon the line configuration, structure type, meteorological and geotechnical settings. Thus, foundation design plays an integral role in the ultimate configuration of a transmission line. To the extent foundations play a role in the total installed cost of the line asset, optimization of foundation sizes becomes an important consideration for the geotechnical and transmission line engineers.

3.1 Structure Configurations

Typical transmission structures are self-supporting single shaft steel poles, non-self-supporting single shaft steel poles, latticed steel towers, guyed latticed steel masts, as well as braced and unbraced H-Frames, each of which have unique foundation load transfer mechanisms (Fig. 3.1).

For the purposes of describing load transfer, transmission structures may be considered as either uplift/compression structures or lateral moment structures. Latticed steel towers and internally braced H-Frames require multiple foundations for support and largely transfer loads to the foundation system in the form of an uplift/compression couple about the structure's centroid, typically with some shear and small lateral moments. Latticed steel towers are three-dimensional space truss systems typically comprised of hot rolled structural steel angles. These structures are one of the most efficient support system available to transmission engineers in terms of load transfer and steel usage. The ability to support significant loads with minimal steel usage yields longer optimal span lengths as compared to other structures, yielding a lower total installed line cost due to the reduced steel weight per structure as well as the reduced number of structures afforded by the longer optimal span length. Alternatively, internally braced H-Frames also impose uplift/compression foundation loads through a planar truss system. The main vertical resisting members are most commonly tubular steel poles, but can also be standard hot rolled structural steel sections. In the case of drilled pier foundations, the size and type of the vertical members in an H-Frame can have strong influence on the foundation diameter and ultimately the cost efficiency of the foundation design. This is the case for all tubular steel pole structures and will be discussed later.

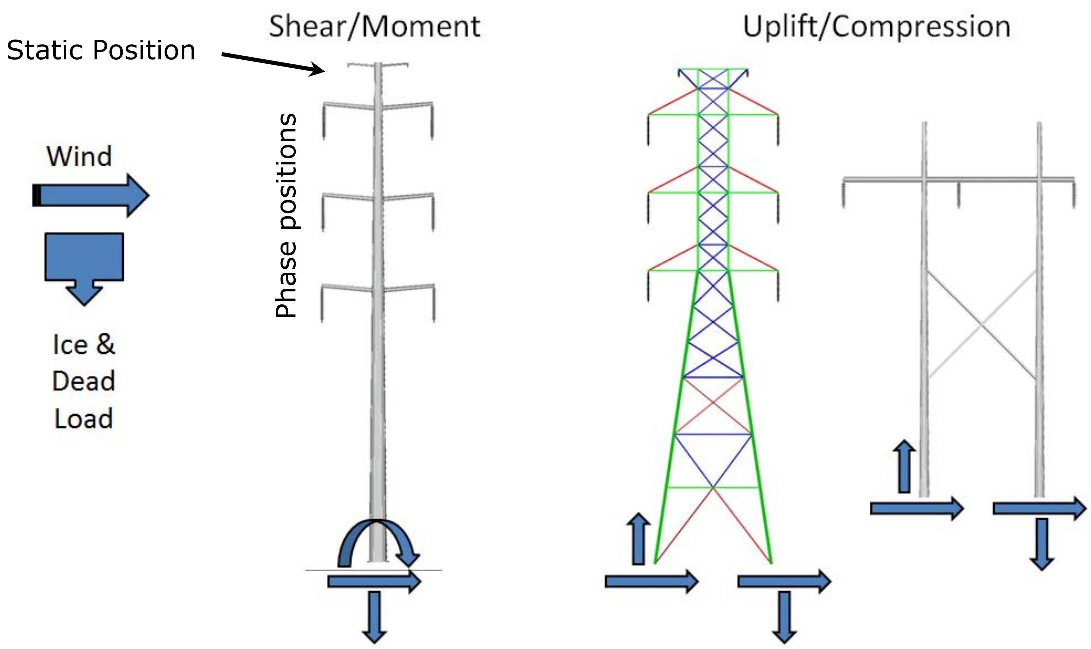


Figure 3.1 – Typical transmission structure configurations

Self-supporting single shaft steel poles and H-Frames without internal bracing are cantilevered structural systems and transfer line loads to foundations in the form of relatively large lateral moments and shear loads with small axial loads. In the United States, contemporary transmission line designs rely on self-supporting single shaft steel poles to a significant degree for a number of reasons. Much of the interstate bulk transmission grid system in the U.S. was built through the mid to late 1900s, primarily on latticed towers. Following the construction of these long distance line assets, much of the recent (late 20th century to the present) high voltage line construction has been associated with more localized system expansion around urban areas with concentrated electric system load growth. Construction in urban corridors is well suited for single shaft tubular steel pole structures because of their small structure footprint and because of aesthetic preference by the public for poles rather than latticed towers.

Alternative structure configurations may employ down guys, which are cable elements that attach structure members to the ground to provide lateral support. Guyed structures are less prevalent in high voltage applications due to reliability concerns about this structural system's reliance on guy wires and anchors, which are subject to corrosion and vandalism.

3.2 Foundation Systems

As with many forms of geotechnical construction, the foundation systems used for transmission lines vary widely depending on the supported structure (type of foundation loading), the geotechnical setting, site access, availability of specialty equipment and the project owner's preferences. A short summary of typical foundation configurations is provided here and their applicability by structure and strata type is provided in Table 3.1.

Table 3.1 Typical transmission line foundation configurations

Structure Type	Geotechnical Formations	
	Soil	Rock
Self-Supporting Single Shaft Steel Poles	CP, DE, M	CP, DE, AR, M
Non-Self-Supporting Single Shaft Steel Poles	CP, DE, M	CP, DE, AR, M
Latticed Steel Towers	CP, DP, G, S	CP, AR, M
Guyed Latticed Steel Masts	CP, DP, G, S	CP, M
Braced H-Frame	CP, DE, S, M	CP, DE, AR, M
Unbraced H-Frame	CP, DE, M	CP, DE, AR, M

CP - Reinforced Concrete Drilled Pier Foundations
DE – Direct Embedment
AR - Anchored Rock Sockets
DP - Driven Piles
G – Grillages
S - Spread Footings
M – Micropiles

CP - Reinforced Concrete Drilled Pier Foundations

Drilled piers are a common foundation system due in large part to their versatility in terms of compatibility with all common structure types and the relative ease of construction in a wide variety of geotechnical formations.

DE – Direct Embedment

Lightly loaded tubular steel structures commonly utilize direct embedment foundations in which the tubular steel section extends below grade and is embedded to the depth necessary for adequate foundation performance. The annulus between the structure and the excavation is typically backfilled with a cementitous backfill to inhibit corrosion.

AR - Anchored Rock Sockets

Anchored rock sockets consist of a reinforced concrete pier embedded in a rock formation to the minimum depth necessary to achieve development length for the longitudinal members of an anchor bolt cage (tubular steel structures with base plate connections) or of the embedded stub angle (latticed steel towers). The reinforced concrete socket transfers structure loads to the rock formation through a series of rock anchors extending to depth.

DP - Driven Piles

Typical applications use a steel pipe pile driven to depth. Annulus soils are removed and replaced with concrete. The structure connection for latticed steel towers is typically achieved by embedding a stub angle in the annulus concrete, although pile caps have been used as well. These are not commonly used in lateral moment loading applications.

G – Grillages

A type of spread footing, these foundations are used to varying degrees in certain regions of the U.S. where difficult access limits concrete deliveries. Their application is limited to foundations subject to axial loading.

S - Spread Footings

Spread footings have largely been replaced by drilled concrete piers in modern construction, but have been used in softer strata extending to significant depth to limit settlement in specialty applications.

M - Micropiles

Micropiles are a relatively new technology within the transmission industry and are gaining acceptance for rock installations, particularly where access is limited or where helicopter installation is required. Micropiles, as implied by their name, are a smaller version of traditional piles. However, their installation is achieved by percussive drilling techniques which permit both rock and soil applications. These foundations are compatible with all structure types.

3.3 Governing Codes

The structural and electrical design of transmission lines in the U.S. is governed by the National Electric Safety Code (NESC) (IEEE Standards Association, 2012). The NESC prescribes methods for calculating the loads applied to and capacity of transmission line structures. However, with the exception of the loads imposed on foundations by the supported structure, the NESC provides no guidance on the methods for designing transmission line foundations. Geotechnical engineers tasked with the design of transmission line foundations are not required to adhere to a specified code document. In the absence of a unifying code, transmission foundation engineers follow accepted standards of practice in the form of published industry-specific guideline documents and applicable non-industry-specific design codes and guideline documents. Commonly referenced documents are described in Section 4 of this document.

For structural systems supporting transmission lines, the NESC prescribes design methods similar to an RBD design methodology by applying varying load factors corresponding to the type of load and strength factors assigned according to the type of structural materials in use. The load and strength factors applied during design are selected based on three designated grades of construction; N, C and B, with B representing the most stringent. Grades of construction are assigned based on the type of transmission facility and its proximity to other facilities or ROW. Under the most basic requirements of the NESC and excluding high importance ROW or electric facility crossings, high voltage transmission lines are only required to satisfy the constraints of grade 'C' construction. However, due to the importance of high voltage line assets, the cost of unplanned outages or of repairs to damaged components, the standard of practice is to design in accordance with grade B construction. For this reason, grade B construction will be the sole focus of this document.

Under the requirements of grade B, the NESC designates three load factors for the load components applied to transmission structures as summarized in Table 3.2.

Table 3.2 - NESC Grade B construction load factors

Adapted from (IEEE Standards Association, 2012)

Load Component	Overload Factor
Vertical Loads	1.50
Conductor Tensions	1.65
Wind	2.50

Similarly, the NESC provides three district loading cases, Light, Medium and Heavy, and two extreme loading cases, Extreme Wind and Extreme Ice for the development of structure and foundation loads. Adherence to the code requires the application of the appropriate district load case and extreme loading case based on the facility's geographic location (Fig. 3.2). The loading parameters of each district load case are summarized in Table 3.3. Extreme load cases are derived from mapped values that are derived from the recommendations provided in ASCE 7-05 (ASCE, 2005).

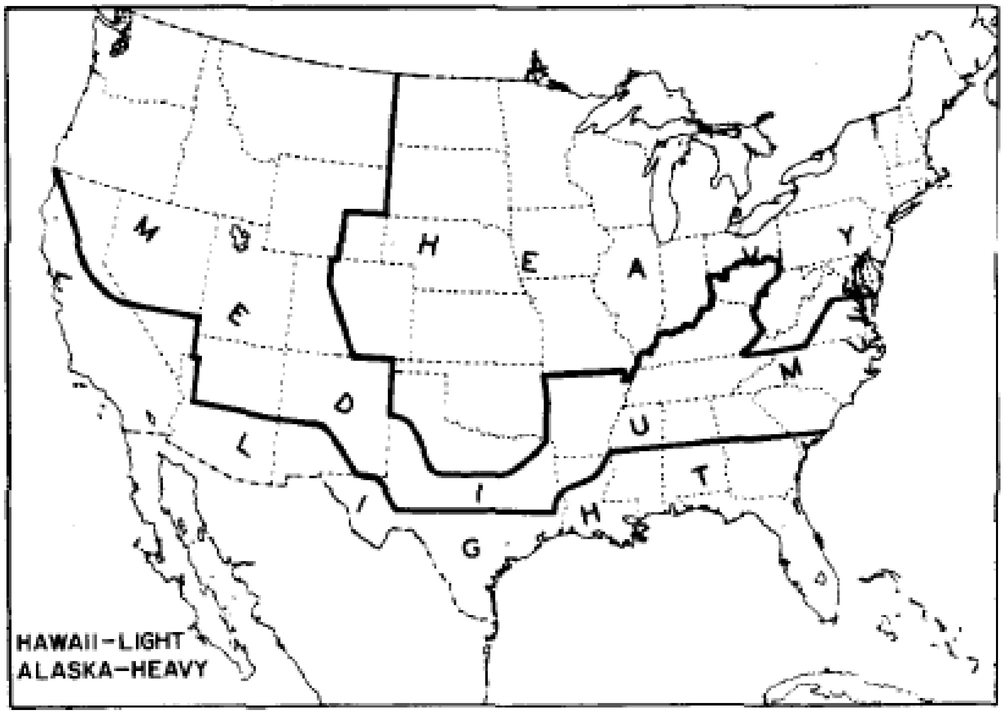


Figure 3.2 - NESC district loading regions

(IEEE Standards Association, 2012)

Table 3.3 - NESC district loading cases

Adapted from (IEEE Standards Association, 2012)

District Load	Air Temp (°F)	Wind Pressure (psf)	Radial Ice Thickness (in.)
Light	30	9	0
Medium	15	4	0.25
Heavy	0	4	0.50

3.4 Laterally Loaded Drilled Piers

The focus of this study is laterally loaded reinforced concrete drilled pier foundations for self-supporting single shaft tubular steel poles. Design of these foundations is generally governed by service limit design criteria and is discussed in Sections 4 and

7 of this document. The foundation loading associated with self-supporting single shaft tubular structures is characterized by a high lateral moment and shear relative to a much smaller axial load. With the exception of very heavily loaded foundations within weak strata, the diameter of foundations for these structures is commonly dictated solely by the anchor bolt circle diameter. Thus, for most soil strata and load magnitudes, these foundations generally exhibit fairly low L/B ratios (L=length, B=diameter), commonly in the range of 2 to 4 and therefore behave as rigid bodies.

A method and computer program, Moment Foundation Analysis and Design (MFAD), was developed by EPRI for the analysis and design of piers exhibiting rigid body motion. This program is used widely throughout the utility industry for the design of transmission line foundations and is the sole program used for foundation analysis within this document. A description of the MFAD model and full scale load tests is provided in Section 7 of this document.

Transmission structures can be categorized according to the deflection angle in the conductor's path supported by the structure and the configuration of the framing supporting the structure. Structures that do not support a deflection angle are tangent structures. The design of tangent structures is generally governed by the wind component of the NESC district load case which applies an Over Load Factor (OLF) of 2.5. However, some tangent structures are configured to support a dead-end configuration in which a span of conductor terminates at the structure attachment points and the entirety of the conductor tension is transferred through the structure to the foundation in the form of a large moment and shear load. The design of these structures is commonly governed by the conductor tension component of the NESC district load which specifies a 1.65 OLF. Similarly, structures supporting large deflection angles, typically greater than 60°, are also governed by the conductor tension component of the NESC district with an OLF of 1.65. Under these structure configurations, factored loads can be reduced by the OLF of the governing NESC load component to derive a reasonable estimate of the nominal load.

For structures supporting medium to small deflection angles and not configured to support a conductor dead-end, there is no clear governing load component. The combination of conductor tensions and wind loading combine to form the governing design load in this configuration and it is conservative to reduce the factored NESC loads by the conductor tension OLF of 1.65 to derive the nominal loads.

This method of load reduction is specific to the load portfolio used by SRP on the Abel-Pinal Central Transmission Line and to single shaft self-supporting structures. Particularly in areas subject to wet snow and ice loading or other regional conditions, the NESC district loads may not govern the foundation design and additional considerations would be required to calculate nominal loads. Similarly, alternate structure configurations, such as guyed structures or latticed steel towers, exhibit more complex load flow characteristic than single shaft structures and the simple load reductions described are susceptible to error.

A notable design consideration particular to single shaft structures is that pole top deflections under normal loading conditions can be on the order of 5% of the structure height, while alternate structure types may exhibit deflection values one order of magnitude less. For the purposes of the line design, this aspect of single shaft structures is only important with regard to NESC required electrical clearances to the edge of the ROW and, to a lesser degree, aesthetic considerations. The flexibility of these structures is also a consideration for the foundation engineer. Multi-leg structures, such as latticed steel towers, are subject to high internal stresses should differential movement of the supporting foundations occur. In the case of single shaft structures, stresses induced by foundation movement are derived from the increased P-Δ effect, where lateral movement of the load application points relative to the foundation imposes additional lateral moments. The P-Δ effect typically results in only marginal increases in structure stress, presuming deflections are not extreme.

4 STANDARD OF PRACTICE - TRANSMISSION LINE FOUNDATION DESIGN

The execution of geotechnical investigations and foundation design for transmission lines is a unique area of practice for geotechnical engineers. The long distances covered by many projects requires evaluation of a broad range of subsurface materials for geotechnical hazards, constructability, accessibility and economy in the development of foundation designs, often with less data than may be attainable in other geotechnical projects. Unlike other long linear structures, such as pavements, were relatively near-surface soil profile data is considered adequate, foundation design for transmission lines requires knowledge of subsurface conditions to considerable depth within the soil profile (potentially up to 10 foundation diameters).

Relative to other areas of civil engineering, geotechnical engineering relies, to a larger degree, on accepted standards of practice in lieu of codified design methodologies. This aspect of the field stems from the variable nature of geotechnical materials and the need for regional experts to successfully execute projects. However, there are numerous geotechnical design codes and guides available that are largely industry-specific. The governing code for the design of transmission line structural elements, the NESC, provides extensive guidance with regard to the loads acting on structures and subsequently the foundations supporting them. The aspects of geotechnical engineering required to develop foundations capable of performing satisfactorily under NESC prescribed loads are not found in its pages. Consequently, as an industry technical organization, EPRI has invested considerable effort conducting research and developing industry guidelines for geotechnical design associated with transmission lines (Spry, Kulhaway, & Grigoriu, 1988); (Kulhaway & Mayne, 1990); (Phoon, Kulhaway, & Grigoriu, 1995); (DiGioia Gray and Associates, 2012). While not enforceable code documents, the EPRI research and the research-based industry guidelines represent an extensive body of transmission line foundation-specific research and are generally accepted as the standard of practice in the United States.

Various design codes, largely from the transportation sector, provide further guidance on the Load and Resistance Factor Design (LRFD) for laterally loaded drilled pier foundations and geotechnical investigations for large linear projects American Association of State Highway and Transportation Officials (AASHTO, 2007) and Federal Highway Administration (FHWA) Brown et al (2010). The FHWA and AASHTO design methods are comparable to the single resistance factor approach adopted in the EPRI Transmission Line Foundation Design Guide (DiGioia Gray and Associates, 2012). However, the resistance factor recommendations differ to some degree, with EPRI and FHWA being essentially equivalent, prescribing resistance factors of 0.63 and 0.67 respectively for laterally loaded piers. AASHTO prescribes a resistance factor of 1.0 under the same conditions.

Sabatini et al (2002) provide methods for selection of boring location and depth for roadway and bridge projects. This approach to subsurface investigation is a marked departure from the current state of practice in the transmission industry. Currently, there is no defined method for the planning of subsurface investigations for transmission lines. However, the continuous nature of roadway construction and the relatively low number of bridge piers compared to transmission line foundations calls for different approaches in the methods utilized for determining boring locations and depths. FHWA recommendations for roadway projects indicate a minimum of one boring every 200 feet and at each bridge pier location. These recommendations are incompatible with and far in excess of the normal practice for transmission line projects.

A survey conducted by EPRI (DiGioia Gray and Associates, 2009)evaluates the state of practice in five key areas of geotechnical engineering for transmission lines:

a) Subsurface Investigations
b) Foundation Design Process
c) Foundations for Single Poles
d) Foundations for Lattice Towers
e) Foundations for H-Frame Structures

The responses to items a, b, and c are an important reflection of current practices relevant to this research. The survey results include responses from 89 participants active in geotechnical engineering for the transmission industry and are discussed below.

4.1 Subsurface Investigations:

Transmission lines are long linear projects characterized by discrete structures separated by large distances, which presents challenges for geotechnical engineers tasked with planning and executing field investigations. Commonly, transmission projects are sufficiently long to traverse multiple geologic settings and all geotechnical strata contained therein. On projects of such magnitude, it is impractical and generally outside of industry practice to conduct subsurface geotechnical investigations at each structure location due to multiple constraints, including schedule, accessibility (at the time of design) and cost (DiGioia Gray and Associates, 2012). A successful investigation will provide foundation engineers with adequate information to design foundations for an entire project with an acceptable level of confidence concerning the engineering properties of the soil at each foundation location.

As a matter of practicality and due diligence, common practice utilizes information from multiple sources to supplement data collected during the field investigation (Fig. 4.1). The extent, type, and utilization of supplemental data in the industry vary. The role of supplemental information sources and the methods for determining representative locations are important aspects of the design process investigated by the recent EPRI research.

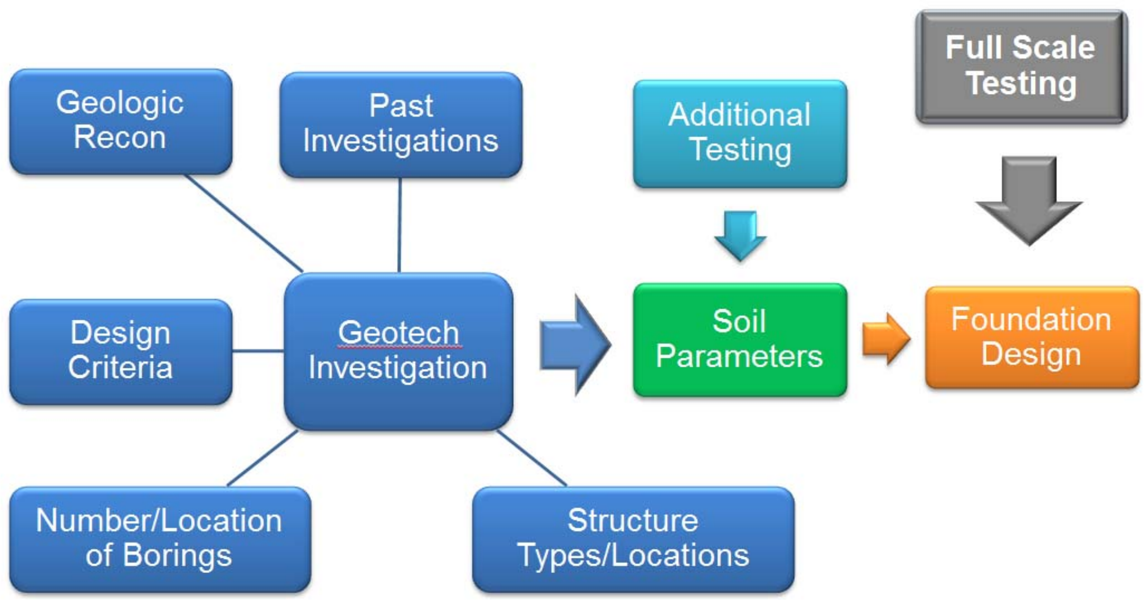


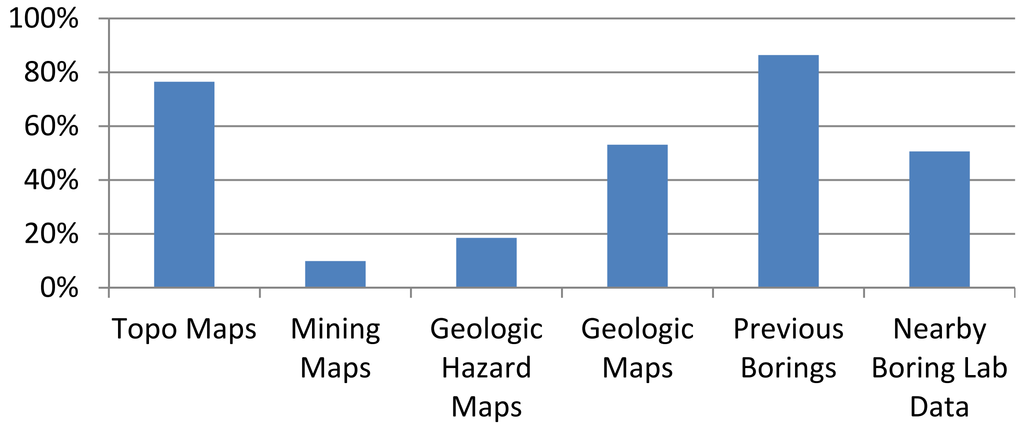
Figure 4.1 - Sample data flow for typical geotechnical investigation

Currently, no codified approach for the implementation of geotechnical investigations for transmission lines exists. The NESC provides no guidance with regard to any aspect of geotechnical engineering for overhead transmission lines. The Rural Utilities Service (RUS) *Design Manual for High Voltage Transmission Lines* is generally intended for use with direct embedment wood structures. However it provides a series of empirical calculations for embedment depths of direct embedment structures based on general classifications of soil strength, ‘Good’, ‘Average’ and ‘Poor’ (Rural Utility Service, 2005). The RUS document references the need to conduct a field investigation when structures are heavily loaded or where low strength soils are anticipated. Selection of boring locations is left to the judgment of the geotechnical engineer. The *EPRI Transmission Structure Foundation Design Guide* (DiGioia Gray and Associates, 2012) includes geotechnical investigation specifications with extensive information pertinent to the means and methods for executing a field investigation. This guide is the only transmission industry-specific document providing guidance on the methods and frequency of testing to be performed at boring locations. The guide recommends SPT testing at every observed change in stratum and at maximum depth intervals of 3ft. However, the guide is not a prescriptive document for planning and conducting geotechnical investigations and therefore does not provide information about methods for selecting boring locations or use of other testing methods to develop an adequate geotechnical database for design.

In the survey conducted by EPRI (DiGioia Gray and Associates, 2012), participants were asked seven questions designed to describe the current standard of practice for transmission line subsurface investigations:

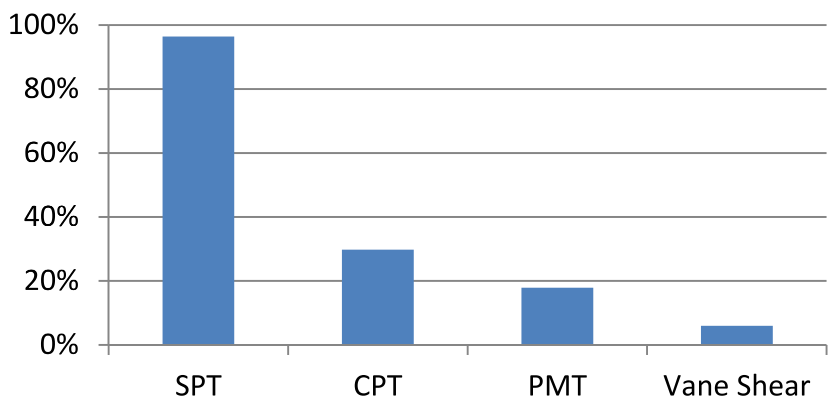
1. What existing information do you assemble for subsurface investigation planning?

    a. Topographic maps
    b. Mining Maps
    c. Geologic Hazard Maps
    d. Geologic Maps
    e. Boring Logs from Nearby Borings
    f. Lab Data for Nearby Borings



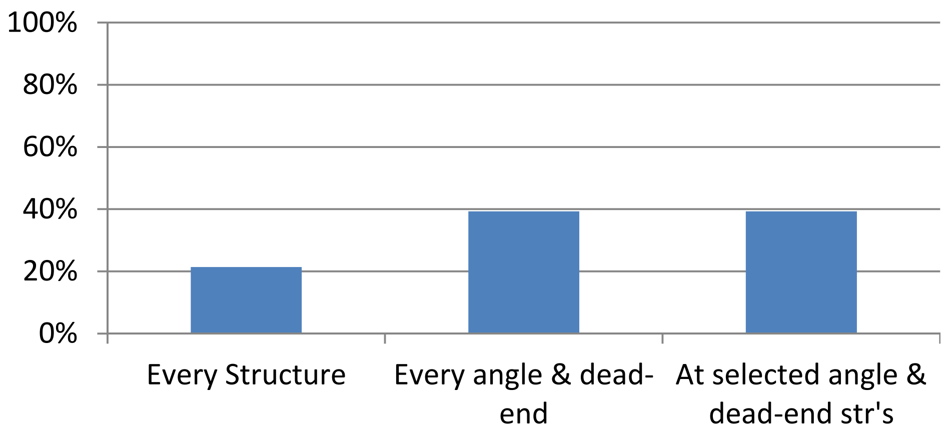
2. What in-situ tests do you normally conduct during a field drilling program?

    a. SPT – Standard Penetration Test
    b. CPT – Cone Penetration Test
    c. PMT – Pressuremeter Testing
    d. Vane Shear



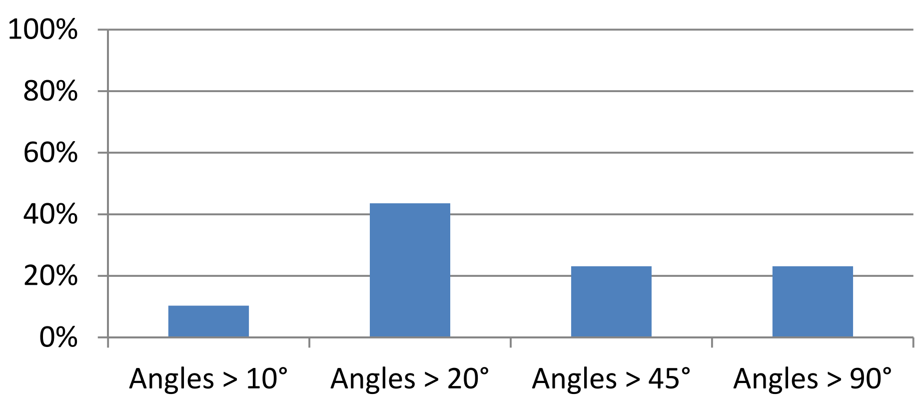
3. Where do you typically locate borings?

   a. At every structure
   b. At all angle and dead-end structures
   c. At selected angle and dead-end structures

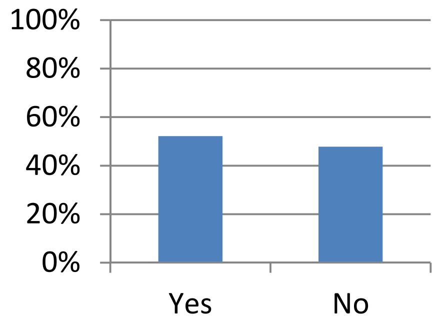


4. What criteria do you use to select which angle structures should be drilled?

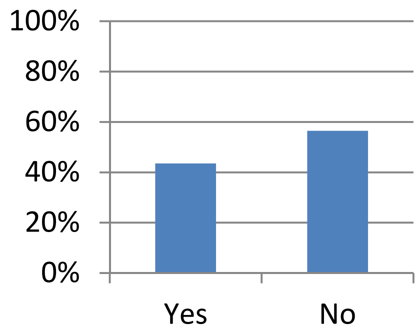
   a. > 10 degrees
   b. > 20 degrees
   c. > 45 degrees
   d. > 90 degrees



5. If you drill additional borings between angle structures, do you use a non-uniform spacing based on the longitudinal geologic profile? (Yes/No)

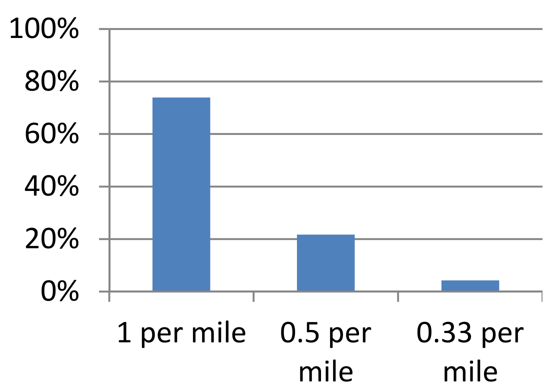


6. If you drill additional borings between angle structures do you use a uniform spacing (i.e., per mile)? (Yes/No)



7. What uniform spacing of borings do you use for additional borings?

    a. 1 boring per mile
    b. 1 boring per two miles
    c. 1 boring per three miles



The methodologies implemented in planning geotechnical field investigations can play an important role in the viability of the database constructed from the investigation activities. Selection of boring locations in the absence of an assessment of the likely areas of similar and dissimilar geotechnical strata may result in failure to adequately sample either anomalous or pervasive strata. Under-sampling of pervasive strata may yield a dataset insufficient for an adequate assessment of the soil properties and restrict ability to economize foundations accordingly. Alternatively, failure to sample anomalous data can lead to unconservative or difficult to construct foundations when anomalous conditions are encountered.

Responses to Question 1 describe methods typically used for planning transmission line subsurface investigations. High percentages of the participants report utilization of topographic maps and logs from nearby borings (77% and 86%, respectively). Fewer respondents report use of geologic maps or geologic hazard maps (53% and 19%, respectively). The method by which these information resources are applied in subsurface investigation planning is unclear from the survey results. Presumably, topographic maps are largely used for access planning and dictate the location of borings to the extent access is feasible. Similarly, the survey results do not tell us whether nearby historical boring results are used to determine where data is available or not, allowing new investigation to be performed solely in areas lacking data, or whether historical boring data are used to identify the extent of important strata for further investigation. Responses regarding topographic and existing boring information are ambiguous for these reasons and therefore potentially misleading. However, the use of geologic mapping does provide a strong indication of the number of respondents utilizing anticipated strata to plan investigations in areas of geotechnical interest.

In situ testing methods applied during field investigations vary regionally and by the soil properties of interest for foundation design. However, responses to Question 2 demonstrate the extensive use of the Standard Penetration Test (SPT), with 96% of the survey participants indicating its use. Pressuremeter testing was used at the relatively low rate of 18%. This is a surprisingly low result given the prevalence of laterally loaded foundations in the transmission industry and the importance of the modulus of deformation, E_p, for their design. The high percentage of positive responses to the SPT test and relatively low reported rate of using pressuremeter testing seems to indicate the industry's reliance on correlations to the SPT blow count for derivation of soil strength parameters.

Dead-end and angle structures are generally the most heavily loaded structures and of high importance to the reliability of the circuit(s) they support. In recognition of this, a common practice is to locate borings at dead-end and angle locations to reduce uncertainty about soil properties, enhancing reliability. Responding to Question 3, 40% report routinely placing exploratory borings at every angle and dead-end structure. An additional 20% of respondents indicate that they locate borings at every structure in a line. The remaining 40% locate borings at selected angle or dead-end structures. These participants were asked to respond to Questions 4 through 7 regarding methods for determining which structure locations are investigated.

It is unknown what size project the respondents had in mind when responding to the survey. Based on the author's observation of industry practices, there is some likelihood that those who indicated a practice of boring at every structure location were referring to smaller projects. In large-scale projects, which may consist of several hundred structures, this approach could become impractical from an economic and scheduling perspective. It is unclear, however, where the balance between excessive investigation and the economic advantages of generating abundant data on such large projects is. It is the author's opinion, this balance is somewhat unique to each project and is dependent upon structure type, the magnitude of foundation loads, variability of the geotechnical/geologic environment along the corridor and schedule constraints. For example, foundation design for a project with relatively light loading and traversing strong soils can be governed by minimum embedment depth requirements (2 x diameter). In this case, which occurred along portions of the ABL-PC project, the value of extensive subsurface investigation beyond that required to verify the presence of stronger soils is limited. Of course, the opposite is certainly true and as loading increases or in situ soil strength decreases, the benefit of extensive geotechnical investigation is clear. To some degree the appropriate balance cannot be known, at least prior to an initial subsurface investigation. The economic benefits of phased geotechnical investigations has been shown for large transmission project (Kandaris, 1994). Generally, the phased investigation approach identified by Kandaris (1994) does not sample at every structure location, rather emphasis is placed on regions where reduced uncertainty can produce economic savings. In regions where this is not the case, less extensive investigations are performed. It should be noted, this approach relies on verification of design assumptions during construction and requires qualified personnel in the field whom are capable to identify anomalous conditions.

Approximately 50% of the survey participants provided responses to Questions 4 through 6, suggesting that the non-responding 50% were either referring to smaller projects, work in areas where topography requires a large number of angle structures or utilize some alternate method for deriving soil properties at tangent structure locations (Fig. 4.2).

Dependent upon regional topography, transmission lines over flat, rural terrain may traverse large distances without any angles or dead-end structures. For long tangent sections, it is necessary to establish intermediate boring locations based on additional criteria. Questions 5 and 6 solicit a yes or no response regarding survey participants' practices of utilizing uniform or non-uniform spacing when selecting tangent boring locations. 47% of the responses indicated that a non-uniform spacing based on the geologic profile was used to select intermediate boring locations. Of the participants utilizing uniform spacing, 74% indicated a preference of 1 boring per mile.

Participants responding to Questions 5 and 6 provided specific information regarding their practice when choosing the location of tangent borings. Of the 9 responses, 5 indicated borings were located based on a predetermined spacing, such as one boring per mile. Two responses indicated borings were located based upon an anticipated geotechnical condition determined by an initial site reconnaissance. Only one participant indicated the use of a statistical approach to determine the optimal location of borings.

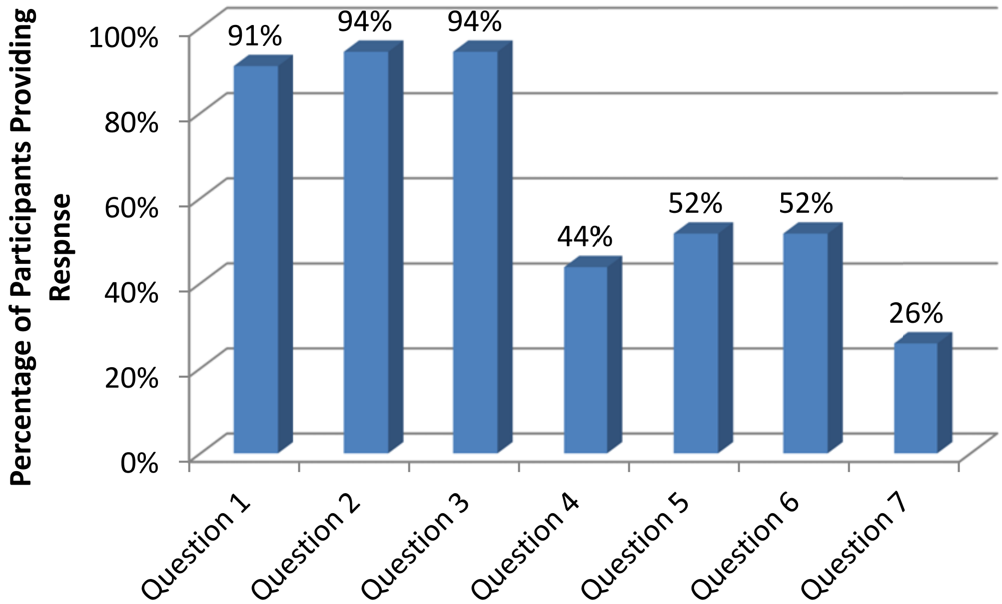


Figure 4.2 – EPRI Survey, question response rates

Adapted from (DiGioia Gray and Associates, 2009)

The variability in responses to questions regarding the selection of boring locations makes it difficult to draw reliable conclusions on how transmission engineers choose their boring locations. However, the level of participation for each survey question in combination with the responses does make it possible to draw some general conclusions based upon the following observations:

- 95% of the survey participants reported that they utilize structure type as an initial criterion when selecting boring locations.
- 48% of the responses to Question 5 indicated that the geologic profile was used to select boring locations. However, only 52% of the survey participants responded to this question, therefore the positive responses only account for 25% of the survey participants.

- Similarly, 44% of the responses to Question 6 reported that boring locations were selected by using a predetermined spacing. These responses account for 22% of the survey sample.
- 66% of the participants that provided a written response regarding the selection of tangent boring locations applied a predetermined spacing such as one boring per mile.
- Only 33% of the written responses gave any indication that the anticipated geotechnical profile was a factor in the selection of boring locations.

The high level of response to Questions 1 through 3 and relatively few responses to Questions 4 through 7 provide a strong indication that the majority of transmission line engineers select boring locations based primarily on the location of high value structures. Similarly, the responses to Question 5 in tandem with the written responses indicate that only 25% to 35% of engineers participating in the survey take anticipated soil conditions into consideration prior to selecting their boring locations.

Survey questions regarding the spacing of tangent borings were answered by fewer than 50% of survey respondents so it is impossible to draw conclusions about the methods used by those not responding. The missing responses to these questions may be due in part to the following:

- Some survey participants may be using techniques other than those supposed by the survey question. It may be reasonable to assume that the techniques used by those not responding would demonstrate a similar distribution of answers as those offering written explanations of their current practices.
- The survey did not segregate survey responses according to small and large transmission projects. As a result, participants who have not worked on larger

projects would have a lesser need to select boring locations beyond those at high value structures.

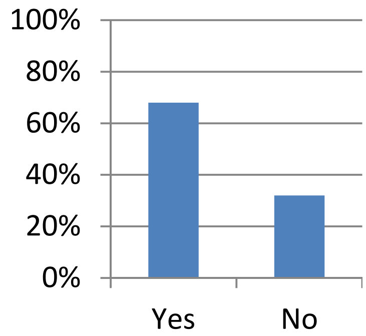
- Some participants may typically work in areas where the terrain requires a large number of angle and dead-end structures, therefore boring at these locations may provide an adequate data set representative of the majority of the line foundation conditions.

Mitigating factors aside, the survey responses with regard to subsurface investigations indicate that the majority of transmission line engineers do not take the anticipated geotechnical conditions into consideration when planning a geotechnical investigation. Based on the survey results, the primary selection of boring locations is most often based upon the location of high value structures followed by a method for selecting intermediate borings typically by use of an established interval. These are telling findings with regard to the readiness of the profession for implementation of a comprehensive RBD.

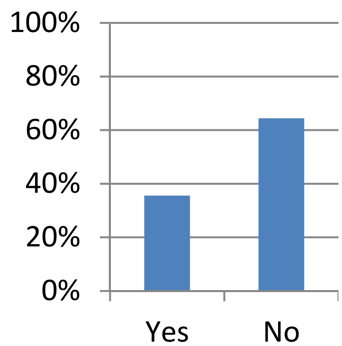
4.2 Foundation Design Approaches:

In the absence of a unifying code document, transmission line foundation designers have the option to perform foundation design based upon an Allowable Stress Design (ASD) approach or a Reliability Based Design (RBD) approach. Generally, ASD design is generally considered "Standard of Practice" with RBD considered "State of the Art". A discussion of these different design methods is provided in Section 5 of this document. The focus of Questions 8 through 10 of the EPRI survey is the foundation design processes used by the survey participants:

8. Do you use an allowable stress design approach (i.e., use safety factors on the strength side)? (Yes/No)

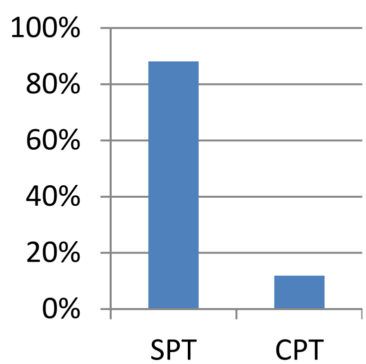


9. Do you use a Reliability Based Design approach (i.e. probability based strength factors)? (Yes/No)



10. What correlations do you use to assign geotechnical design parameters using the following in-situ tests?

a. SPT
b. CPT
c. Other



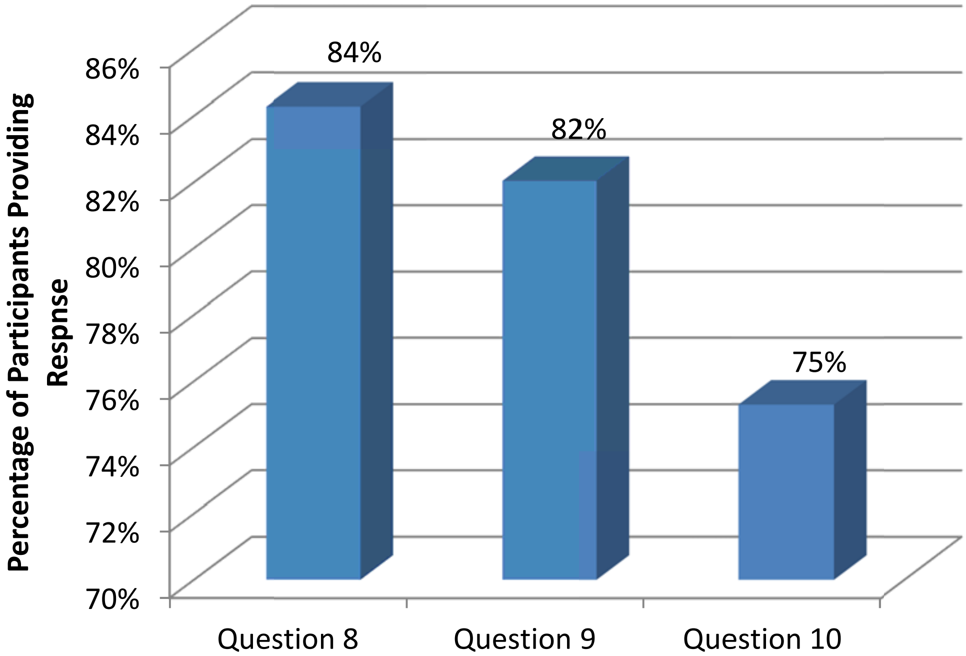


Figure 4.3 - EPRI Survey, question response rates

Adapted from (DiGioia Gray and Associates, 2009)

Responses to Questions 9 and 10 indicate a strong majority (68%) of the survey participants use an ASD methodology in lieu of RBD (36%). It should be noted this survey was conducted in 2009, fourteen years after the first RBD guide document for transmission foundation design was published (Phoon, Kulhaway, & Grigoriu, 1995). The initial RBD guide document by EPRI utilizes a multiple resistance factor approach, which required more complex analysis than many practitioners were willing to carry out due to perceived additional complexity and additional data requirements. Subsequently, EPRI has published a foundation design guide (DiGioia Gray and Associates, 2012) in accordance with the simplified RBD methods recommended by ASCE Manual 74 "*Guidelines for Electrical Transmission Line Structural Loading*" (ASCE, 2010) and a single resistance factor approach similar to those implemented for transmission line structure foundations outside the U.S. (CIGRE, 2008); (CSA, 2010), bridges (AASHTO, 2007) and buildings (ACI, 2011); (AISC, 2006). In support of the effort to implement RBD for transmission line foundations, EPRI has also revised the MFAD design model to incorporate the single resistance factor design method defined by the EPRI 2012 design guide.

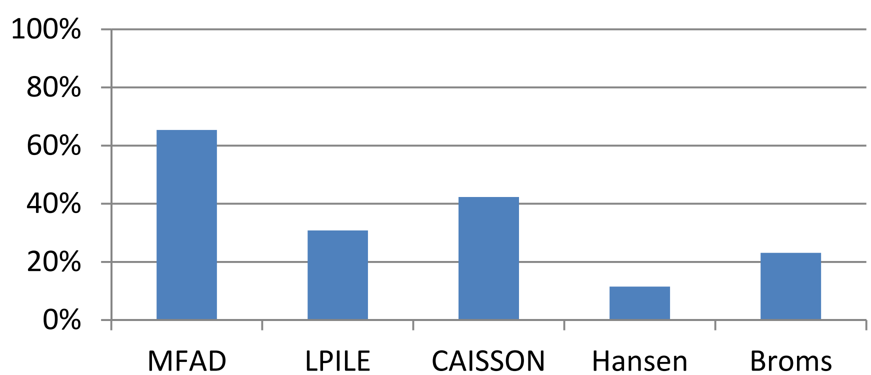
Through the recent publication of these design guides and related software, it's apparent that the standard of practice for transmission line foundation design is in flux. It seems likely that the simplified nature of the recent EPRI guide documents will gain wider acceptance than past recommendations. This most recent EPRI work is derived from statistical calibration of a design model built upon an estimated probability of failure, P_f, corresponding to a return period load application in accordance with ASCE Manual 74. Adjustments to the desired level of reliability are then achieved through adjustment of the load return period. However, this simplified method does not address many of the uncertainties explicitly incorporated in past work (Phoon, Kulhaway, & Grigoriu, 1995). The extent of acceptance among practitioners is unknown at the time of this writing; however the most recent data suggests ASD methods are the current standard of practice. Presumably this will change within the foreseeable future, although it is likely ASD will remain a "benchmark" as future RBD practice develops (Section 5.6)

As indicated by the strong preference for the SPT test, responses to Question 10 reflect a heavy reliance (88%) on SPT correlations to derive soil strength parameters. In the particular case of laterally loaded foundations, in the absence of pressuremeter testing, which only 18% of respondents reported using, the most commonly available means available for calculating the lateral modulus is through correlations to SPT blow count. The survey did not solicit information regarding lab testing performed by the survey participants. It is unknown to what extent SPT correlations are used to calculate other important soil parameters for design.

4.3 Foundation Design:

Laterally loaded foundation design for transmission structures is iterative in nature and somewhat unique due to typically short pier dimensions, which generally result in rigid body motion. Foundation engineers have a number of computer programs at their disposal for foundation design including industry specific modules such as MFAD. Survey Questions 11 and 12 ask about the software and design parameters utilized by transmission foundation engineers.

11. What design methods do you used for laterally loaded drilled shaft foundations?
    a. MFAD
    b. LPILE
    c. CAISSON
    d. Hansen
    e. Broms



12. What safety factors or strength factors do you use for laterally loaded drilled shaft foundations?

Single Pole/Drilled Shaft Design Model	Range of Safety Factors (ASD)	Range of Strength Factors (RBD)
MFAD	3-4	0.6
CAISSON	1.0-3.0	0.75
LPILE	1.0-3.0	0.75
HANSEN	0.6-3.0	-
BROMS	0.6-2.5	-

Table 4.1 – EPRI Survey factor of safety responses

Courtesy of ((DiGioia Gray and Associates, 2009)

Responses to Question 11 indicate that a substantial majority of survey participants (65%) utilize MFAD as the analysis method for laterally loaded drilled shaft foundations. In recognition of this, all analyses for this research are developed using the MFAD model. A description of the MFAD model is provided in Section 7.6 of this document.

5 RELIABILITY BASED DESIGN

The most basic goal for engineering design is to sufficiently predict the behavior (stability, deflections, etc.) of engineered systems under the conditions the system will encounter over the specified design life to achieve the desired result. In the particular instance where the loading, material strength and the design model represent 'real world' conditions perfectly, satisfactory performance of the engineered system is essentially guaranteed. However, it is explicitly recognized by practicing engineers that loading conditions are governed by naturally occurring phenomena (wind, ice, stream flows, etc.) and are impossible to know with absolute precision. Similarly, materials utilized in the constructed system possess inherent variability in their strength to a greater (soils) or lesser (steel and concrete) extent.

Recognizing these inherent variabilities, it is insufficient to represent structure and foundation performance with certainty. Instead, the engineer must carry out design in a manner where the risk of adverse performance (structural collapse, excessive deformation, etc.) is reduced to an acceptably low threshold value. Historically, this has been achieved through application of a global factor of safety to achieve the desired margin for error between the predicted system capacity and the anticipated load regime (largely implemented in the form of ASD). This global factor of safety approach has been applied successfully to foundation design in various forms essentially since the development of geotechnical engineering as a profession. The global factor of safety, however, is inherently problematic from a reliability perspective as has been documented by a number of authors (Burland, Potts, & Walsh, 1981); (Simpson, Pappin, & Croft, 1981); (Kulhaway, 1984); (Phoon, Kulhaway, & Grigoriu, 1995) and the profession continues to move toward more sophisticated methods of design.

During the latter half of the twentieth century, enhanced awareness of structural safety and a desire to enhance economy in design practices contributed to a shift away from ASD toward RBD in structural engineering practice (Freudenthal, 1947); (Pugsley, 1955). This progression led to implementation of RBD code documents for structures in the U.S.: for concrete (ACI, 1983), steel structures (AISC, 1986), bridges (AASHTO, 2007) and (ASCE, 2006) and for transmission structures (ASCE, 2010). Presently, for the purposes of foundation design, ASD remains a widely used methodology. This is untrue in some sectors, such as transportation, where recognition of inherent difficulties with the ASD approach, incompatibility with RBD structural codes, economics and a desire for consistent achieved levels of reliability have led to implementation of RBD code documents (AASHTO, 2007) (Allen, 2005) and (Brown, Turner, & Castelli, 2010) for bridge and highway foundations. Although, only limited components of the aforementioned documents are derived from true RBD analysis efforts. Many of the load and/or resistance factors presented are derived from either back calculation to existing factors of safety or some alternate analysis in the absence of sufficient data to perform comprehensive RBD with the intent of further refinement as additional data becomes available (Brown, Turner, & Castelli, 2010). Considerable effort by ERPI has been made to develop RBD design methodologies for transmission line foundations (Phoon, Kulhaway, & Grigoriu, 1995); (DiGioia Gray and Associates, 2012) as well.

This section provides a discussion of the theoretical aspects associated with current ASD practice for foundation design and the challenges to deriving consistent reliability from foundations developed using a global factor of safety. This discussion of ASD will document the desire to implement rational methods to achieve consistent reliability among differing foundation installations and compatibility with the structures they support. A similar discussion is provided on the conceptual basis of limit state design as the basis of RBD, more recently developed probabilistic methods classified as RBD and the procedures required for rigorous development of RBD methodologies. This discussion is presented as a summary of a more extensive version provided by Phoon, Kulhaway & Grigoriu (1995). A further discussion of the RBD foundation codes of interest for this research is provided in Section 6.

5.1 Allowable Stress Design:

Engineering analysis is achieved through simulation of physical phenomena with mathematical algorithms that may or may not adequately depict the complete behavior or complexity of the mechanisms they represent. For this reason, among others, engineering analyses involve an inherent risk of failure. Structural collapse and poor performance are among the scenarios most commonly associated with failure and rightfully so, given the potential safety, economic and legal ramifications should they occur. However, systems that perform adequately at greater economic expense than necessary can be readily identified as economic failures as well. Thus engineers should strive for satisfactory performance as well as economy, knowing that assurance of the former comes at the expense of the latter. In recognition of this, engineers aim to reduce the risk of adverse performance to an acceptably low probability. Through experience, engineers know uncertainties exist in calculated loads, design models, material properties and construction procedures, all of which contribute to the aggregate probability of adverse performance. In traditional design, none of these uncertainties are dealt with in an explicit manner individually. Rather, their estimated cumulative effects are addressed through application of a global factor of safety (Phoon, Kulhaway, & Grigoriu, 2000).

Allowable stress design is a subdivision of traditional design methodologies that apply a global factor of safety. This factor of safety is readily applied to either structure loads (Load Factor Design) or to working stresses within the engineered system (Allowable Stress Design). Selection of the appropriate design method can be made from an assessment of the most variable or uncertain value in the design model. In the instance of foundation design, soil strength properties are generally recognized as the most variable component of the design model and the factor of safety is traditionally applied to the foundation capacity (Phoon, Kulhaway, & Grigoriu, 1995). This approach of applying the factor of safety to foundation capacity falls within ASD and is the focus of this discussion.

For transmission line foundations, generally accepted factors of safety range from 2 to 3 as applied in Eq. 5.1.1 where Qd is the unfactored design load, Rn is the nominal capacity and FS is the factor of safety (Kulhaway & Phoon, 2002).

$$Qd \leq Rn/FS \qquad \text{Eq. 5.1.1}$$

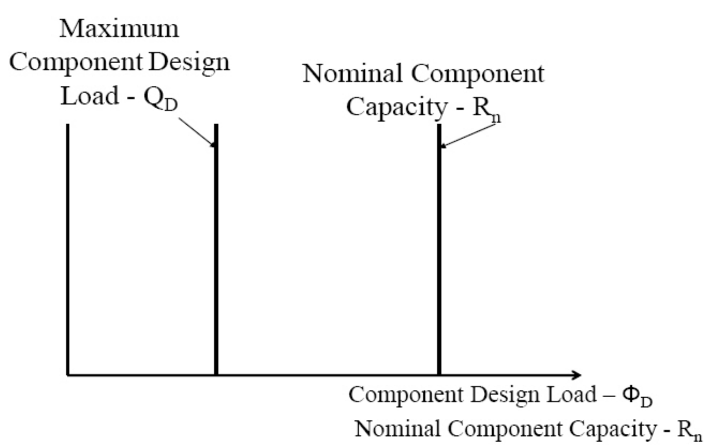


Figure 5.1 - ASD design model

(Figure courtesy of (DiGioia Gray and Associates, 2012))

The factor of safety is a value strongly rooted in empiricism, relying heavily on professional judgment and past experience to gauge the appropriate value for the specific design environment. This is a justifiable aspect of geotechnical design in recognition of its site specific nature and often sparse data available on a project basis. Engineers may rely on regional expertise to verify observations made during the investigation process and the factor of safety is adjusted according to the perceived level of confidence in the project database relative to previous experience. As noted by Allen (2005), this method of FS calibration tends to become progressively more conservative over time:

"In past, and current, allowable stress design practice…, the FS was based on engineering judgment and long-term experience. If failures started occurring when using the selected FS values, increases in the FS were made, again based on judgment, to reduce the recurrence of performance problems to an acceptable level. If no failures occurred, FS values were in general not reduced to get closer to the level of safety desired (i.e., to just above the level where an unacceptable number of failures begins to occur), causing FS values to tend to be overly conservative. Therefore, while not theoretically rigorous, the development of FS values has at least, based on judgment and long-term experience, considered some desired level of safety, though that level of safety may not be consistent across limit states and may not be at the target level for LRFD structural and geotechnical design."

In the absence of measured variability within each of the design inputs, it is difficult to know the actual margin of safety achieved through application of the global factor of safety (Fig. 5.1). Thus, application of a larger factor of safety does not necessarily provide a larger margin of safety in the presence of highly variable design inputs. This is the essence of the greatest challenge to ASD methods. Application of consistent safety factors across various design scenarios where input parameters may exhibit differing states of variability most assuredly results in variable levels of achieved reliability. Adjustment of the factor of safety based on professional judgment certainly provides some mitigation of the possible results of this uncertainty. However, professional judgment and regional experience are also variable among individual engineers and, therefore, must contribute additional variability to the achieved margin of safety.

Application of a Factor of Safety (FS) in the most basic form of Eq. 5.1.1 does not fully address another key challenge in execution of ASD for foundation design to achieve consistent reliability. Specification of a desired FS is not sufficient to derive a consistent level of safety above the nominal foundation capacity across various design assumptions. This particular aspect is illustrated through examination of the achieved factor of safety in Table 5.1 following computation of pier uplift capacity using several different design models (Kulhaway, 1984). The computations noted are for a 5ft diameter by 5ft deep straight sided drilled shaft in clay. The average side resistance is 750 psf with a potential tip suction of ½ atmosphere caused during undrained transient loading at the end of a pier in uplift.

Table 5.1 - ASD design capacity example

(Source (Kulhaway, 1984) p. 395)

Design Assumption	Design Equation	Q_{ud} based on FS = 3(kN)	Q_u/Q_{ud} ("actual" FS)
1	$Q_{ud} = (Q_{su} + Q_{tu} + W)/FS$	170.7	3.0
2	$Q_{ud} - W = (Q_{su} + Q_{tu})/FS$	214.2	2.4
3	$Q_{ud} = (Q_{su} + W)/FS$	108.9	4.7
4	$Q_{ud} - W = Q_{su}/FS$	152.4	3.4
5	$Q_{ud} = W/FS$	21.8	23.5

Note: Q_{su} = side resistance = 261.8 kN
Q_{tu} = tip resistance = 184.4 kN
W = weight of shaft = 65.3 kN
Q_u = available capacity = $Q_{su} + Q_{tu} + W$ = 511.6 kN
Q_{ud} = design uplift capacity
FS = factor of safety
1 kN = 0.225 k

This trait of the single factor of safety is problematic and requires the factor of safety be defined in relation to the intended design model to achieve the desired level of safety above the nominal capacity. The relationship between the design model and the achieved margin of safety is not exclusive to traditional design methods as RBD methods must also be calibrated for specific design models.

In contrast to RBD, ASD generally applies a consistent factor of safety across any number of design calculation methods. Since ASD does not explicitly recognize model variability, it is therefore inseparable from other uncertainties present in the design. Thus, continued refinements in the standard of practice, particularly increasingly accurate design models, have been neglected in terms of a refined FS where individual contributions of design elements to overall uncertainty cannot be addressed. This is true of either ultimate capacity based design models or deformation limit based design.

In an environment where increasingly sophisticated assessments of safety levels are conducted as the standard practice for structures, there is motivation to enhance the understanding of the corresponding margin of safety in foundations for a number of reasons. Generally, there is a desire to achieve an incrementally greater level of reliability in foundations relative to structures due to the higher cost of foundation repair/replacement relative to structures. The goal of increasing reliability among structures and foundations cannot be readily achieved with any certainty under the ASD format. As noted, increasing the global factor of safety does not necessarily yield a greater margin of safety where high variability design inputs exist. Without an assessment of the design system within a probabilistic format the effects of variability in the design model remain unknown, thus ASD requires a higher level of conservatism than might otherwise be required in a properly executed RBD format to provide assurance of a reliable design. This is undesirable from both a reliability and economic perspective.

"The relatively low number of transmission line foundation failures would suggest that this approach {ASD} has been successful if not an economic failure" (Peyrot & Dagher, 1984).

Substantial headway has been made in the assessment of reliability for structures, enhancing both safety and economics of the transmission system. Building structures on foundations that have been derived in an incompatible and conservative manner is a disservice to the work of the structural engineer and the geotechnical profession (Phoon, Kulhaway, & Grigoriu, 2000). For these reasons, there is an increasing effort to develop rational methods for evaluating reliability in foundation design with the ultimate goal of employing RBD design methods for transmission line foundation design. Existing RBD methodology and guideline documents are discussed here as well as in Section 6 of this document.

5.2 Limit State Design

The desire to improve the economy, reliability and compatibility of structure designs is the primary motivation for adoption of RBD for foundation design. The first incremental step toward rationalization of foundation design into a probabilistic framework is assessment of the limit states governing design. This method of design draws from structural engineering practice and is well suited for foundation design. The explicit assessment of the various failure states affecting foundation design is called Limit State Design (LSD) and is predicated on a design philosophy recognizing three basic design elements: identification of all potential failure modes (limit states), application of design checks to evaluate each limit state, and development of a design sufficient to demonstrate that each limit state is sufficiently improbable (Phoon, Kulhaway, & Grigoriu, 1995); (Phoon, Kulhaway, & Grigoriu, 2000).

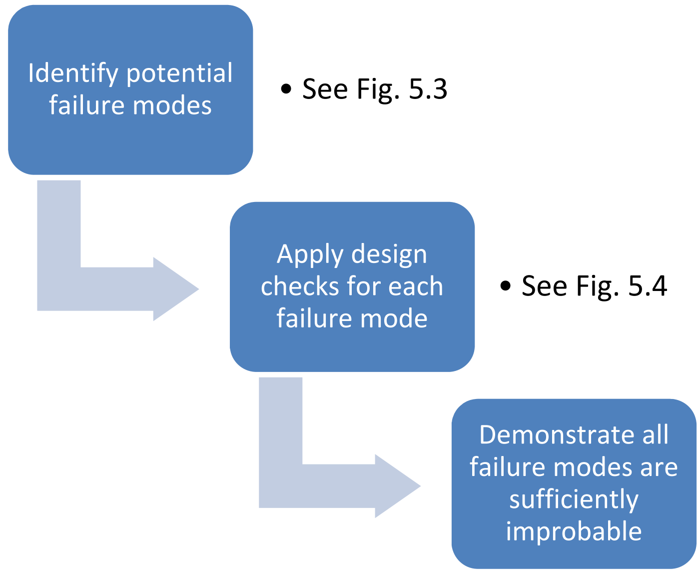


Figure 5.2 - Limit state design process

a. Identification of Limit States

Identification of limit states is not an entirely straightforward effort and is founded in professional judgment as is the traditional factor of safety. However, the key difference is in the use of judgment in the LSD design process to assess the subsurface mechanisms contributing to probable failure modes rather than selection of a non-site-specific factor of safety. The importance of this evaluation is well recognized and has been noted as equal in importance to the more elaborate probabilistic assessments generally associated with RBD (Phoon, Kulhaway, & Grigoriu, 2000); (Phoon, Kulhaway, & Grigoriu, 2000).

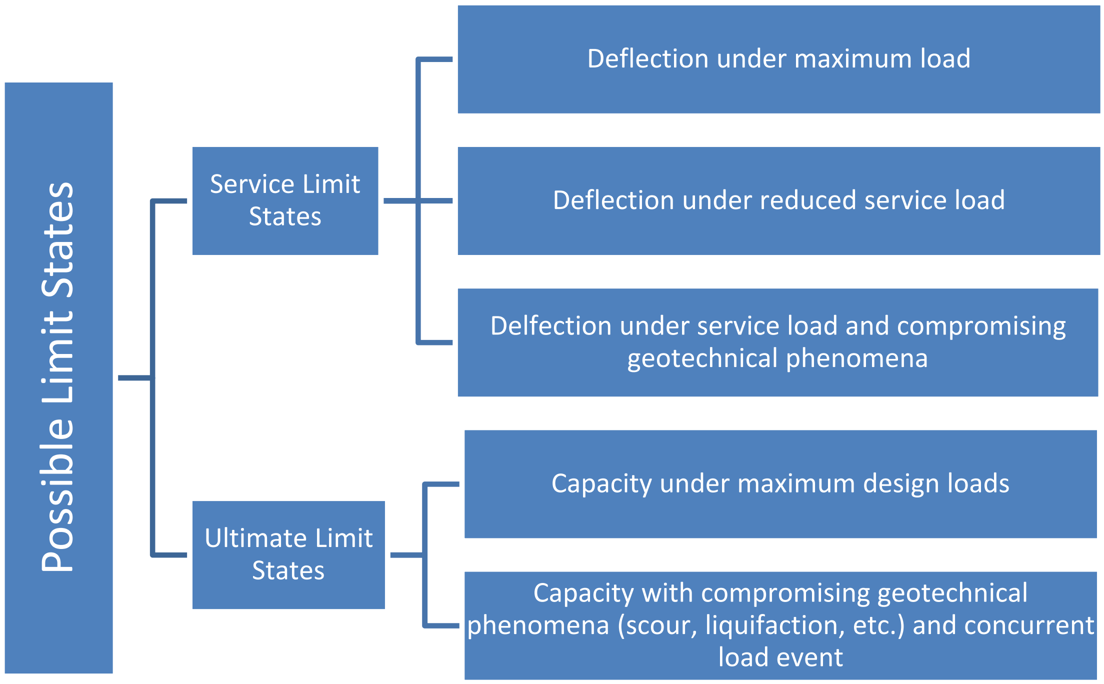


Figure 5.3 - Potential geotechnical limit states

In their most basic form, limit states can be grouped as service limit states and ultimate limit states. Ultimate limit states involve catastrophic failure or large deformation associated with failure of the foundation. Service limit states are generally limits on deformation imposed by the superstructure that are acceptable for the continuous use of the structure and foundation. Within these two basic groupings, there may be a number of subsets dictated by individual geotechnical phenomena (scour, liquefaction, etc.) for ultimate limit states and operational limitations on service limits (permissible deflections for long term serviceability, short term contingency, etc.) (Fig. 5.3).

b. Perform Checks on Limit States

Each limit state should represent a unique combination of performance criteria, geotechnical and loading conditions. Correspondingly, the design model applied to each limit state may be changed to suit the form of analysis required. Identification of compatible conditions and a design model is not a trivial endeavor. For the purposes of this discussion, the design model is considered the combination of applied loads, assumed compromising geotechnical conditions, geotechnical parameter measurements (SPT N-Value), correlations to strength parameters, laboratory or direct *in situ* measurements of strength, and the mathematical model incorporating all of these inputs to predict the foundation behavior (Fig 5.4).

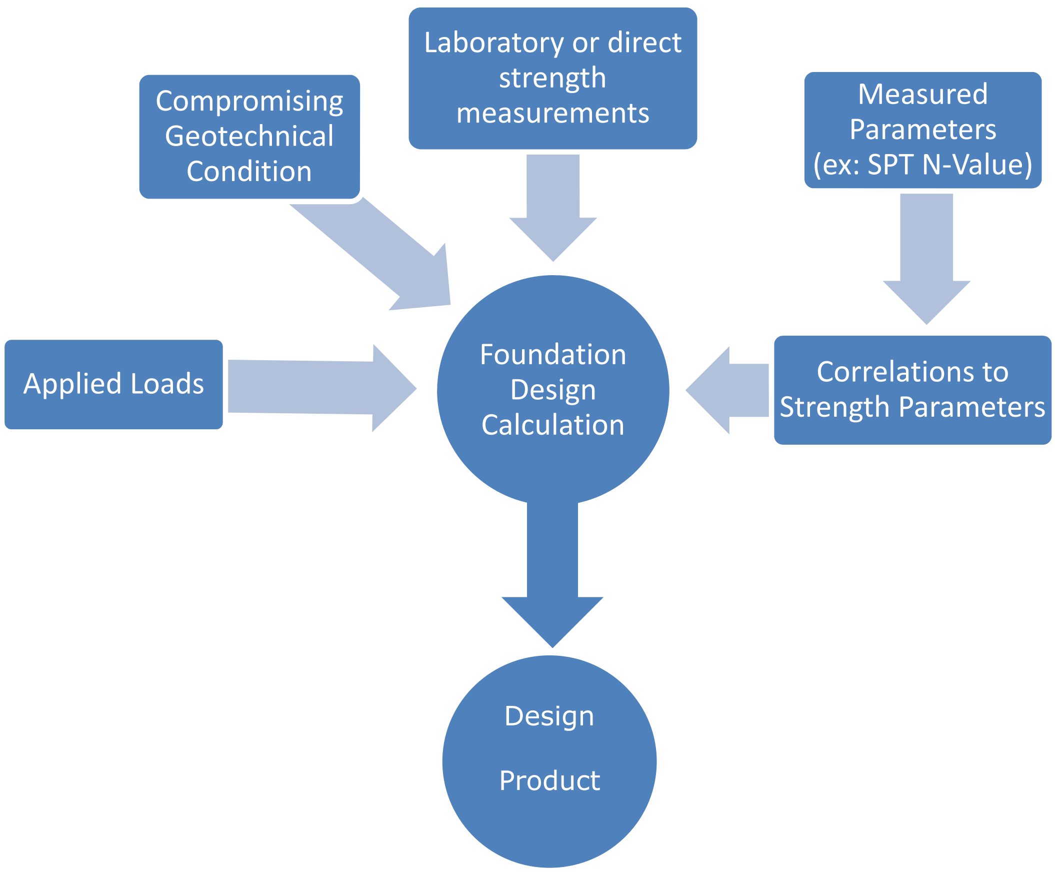


Figure 5.4 - Foundation design process

For the design model to be successful (accurate), each of the contributing components must develop in a compatible manner. This is exemplified by the relationship between applied loads and compromising geotechnical conditions e.g., transient geotechnical conditions such as pier scour, liquefaction, ice/debris loading, etc., causing a temporary change in foundation behavior. Generally, foundation engineers are not responsible for the calculation of applied loads, which are likely supplied by structure designer. As such, the loading conditions of interest for the structure designer may not be entirely compatible with the needs of the geotechnical engineer beyond calculation of ultimate load capacity and service limit deflections under typical geotechnical conditions.

Compromising geotechnical conditions are design considerations that may require attention from the geotechnical engineer in accordance with a particular limit state. Each of these geotechnical conditions may be attributed to a natural phenomenon with some known probability of occurrence and, in certain cases, an accompanying probable load event (e.g., ice loading due to stream flow under winter storm conditions). Where the potential for transient geotechnical conditions exist and they have low probability of occurrence, it is undesirable to evaluate foundation performance under the similarly improbable load event (e.g., liquifaction due to the maximum probable earthquake paired with loads for a 100-yr wind event). The aggregate probability of occurrence amongst the geotechnical and load events is an important consideration to avoid duplication of low probability concurrent events yielding overly conservative designs. Similarly if a compromising geotechnical condition can be attributed to a particular weather phenomenon, loading should be applied in a manner representative of the appropriate conditions.

Computation of geotechnical parameters and foundation behavior requires the same care. In general, geotechnical engineering is rife with empirical correlations to strength parameters and other computations related to foundation performance (DiGioia Gray and Associates, 2009). The reliance on empirical correlations is not problematic by itself. However, compatibility of the correlations to the design conditions under consideration undoubtedly is problematic. Empirical correlations to design properties are typically relatively simple mathematical models for converting easily obtained data (SPT N-Value) into useful strength properties (φ, c, E_p,), which are generally more costly and time consuming to obtain by direct measurement. The price to be paid for the simplicity and convenience of these models is limited flexibility. Generally correlations have been calibrated for a narrow range of soil types and conditions. Beyond these conditions, the accuracy of the model suffers.

As noted by others (Kulhaway, 1992); (1994); (Phoon, Kulhaway, & Grigoriu, 1995), there are consequential differences between structural and geotechnical engineering. For structural engineers, models representing material and structure behavior can generally be expected to be robust in their predictions across a broad range of material and structure configurations. Geotechnical engineers, in contrast must recognize the limitations of their computational models and the complexities of the soil environment at large. This requires selective use of models appropriate for the material at hand and remains a source of uncertainty in geotechnical calculations.

c. Demonstrate Low Probability of Limit State Failures

The methods employed for assessment of uncertainties within the context of limit state design are the main underpinning of modern reliability based design. Demonstration of the probability of reaching any particular limit state is best achieved within a probabilistic framework to yield a desired level of certainty. This is the focus of the remainder of this study.

However, non-probabilistic limit state design methods have been implemented and are largely recognized as partial factor of safety methods. Conceptually, these methods apply factors of safety to each component of the design model (load and resistance elements). Each factor varies in magnitude based on the level of uncertainty or variability associated with the parameter it represents. Parameters with high degrees of certainty would therefore accompany a partial factor of safety near a value of 1.0 while less certain parameters employ factors greater or lesser according to the appropriate need to reduce (resistance) or increase (loads) the influence of the parameter in the design model.

5.3 Reliability Based Design:

Reliability and economy in design are opposing, but equally important goals for foundation engineers. Perfect reliability can be obtained only at exceptional cost, thus a balance must be found to achieve satisfactory reliability at a satisfactory cost. Historically, the use of deterministic methods employing factor of safety methods have tended to yield conservative results at greater economic cost than may have been necessary (Peyrot & Dagher, 1984); (Phoon, Kulhaway, & Grigoriu, 2000); (Allen, 2005). This is an understandable evolution within the constraints of design methods that cannot fully specify the design risks at hand. Thus engineers will inevitably and justifiably err on the side of caution in consideration of the consequences of poor reliability compared with poor economy.

The broader concept of reliability based design stems from the desire to address design risk within a rational framework capable of yielding consistent reliability (low probability of failure) across varying design conditions (soil type, loading regime, foundation type). Across all areas of practice, development of such a rational framework has not been entirely straightforward given the complexity of the design uncertainties at hand.

Enhanced assessment of design reliability first came of interest in civil engineering for structural design during the second half of the twentieth century in recognition of the shortcomings of deterministic design methods. The results of this effort are seen in the release of RBD codes in the U.S. for concrete (ACI, 1983) and steel (AISC, 1986). This trend has persisted in structural engineering fields across the world and within the U.S. to the present.

For the purposes of achieving consistent reliability amongst structures and foundations, it is desirable to implement similar RBD methods for foundation design. However, geotechnical engineering has generally lagged in the adoption of RBD methods in recognition of some key elements which have generated resistance to assessment of design risk by probabilistic methods:

- Soil behavior is not easily represented by traditional probability distributions. Probabilistic representations of soil behavior are further complicated by heterogeneity on a project scale and broad variability on a regional scale (Simpson, Pappin, & Croft, 1981); (Boden, 1981); (Phoon, Kulhaway, & Grigoriu, 1995).
- Geotechnical investigations generally do not produce the amount of data required to perform probabilistic analyses for design on a project specific basis.
- Practitioners are uncomfortable or unwilling to perform the complex statistical evaluations traditionally associated with RBD methodologies (Beal, 1979), (Phoon, Kulhaway, & Grigoriu, 1995), (Griffiths, Fenton, & Tveten, 2002), (DiGioia Gray and Associates, 2009).

Several geotechnical RBD guide and code documents have been implemented to overcome the challenges noted through assessment of general soil variability and probabilistic calibration methods in various industries (transmission foundations (Phoon, Kulhaway, & Grigoriu, 1995); (DiGioia Gray and Associates, 2012); (CIGRE, 2008); (CSA, 2010) and highways (Allen, 2005). These documents are regarded as a simplified approach to RBD as the complex probabilistic analyses commonly associated with RBD methods are not performed by the design engineer. Rather, the reliability analyses have been carried out by the developers of the guideline document with resistance factors calibrated from assessment of generalized geotechnical databases, full scale load tests, etc. A conceptual description of RBD and the methods for calibration of resistance factors are discussed next.

Conceptual Basis of RBD:

It is well understood by practicing engineers that all parameters contributing to design calculations have some degree of associated uncertainty. This is to say, any design calculation carries some inherent risk that the resulting design may fail to achieve the desired performance characteristics. The goal of RBD is to quantify each uncertainty within the design calculation and derive a rational and calculable framework to achieve a consistent (low) probability of failure across varying design scenarios and methods.

In this context, 'failure' is a broad term in reference to any deficiency in foundation performance relative to the goals established during design. Thus failure should not be construed as limited to catastrophic structural failure and is more commonly in reference to excessive deformations that may or may not have tangible effects to the detriment of the superstructure. The negative connotation of the term 'failure' has led to adoption of the reliability index, 'β', as an equivalent measure of probabilistic performance, which is discussed later.

For the purpose of foundation design, broad classifications of design uncertainty generally sort design inputs into two groups, load and resistance (strength). Each of these model inputs derives uncertainty from multiple sources. Uncertainty in load values is substantially the result of variability in the natural phenomena imparting loads upon the superstructure (wind, ice, temperature and any combination thereof). While load producing events are certainly the greatest contributor to uncertainties in loads, further variability is introduced through the prediction of structure behavior in response to applied loading to generate foundation loads, quality of construction, material variability and so on. The extent to which each uncertainty contributing to the aggregate load uncertainty is accounted for should be determined by the sensitivity of the design model to such uncertainty. In the particular instance of transmission lines, especially those supported by single shaft tubular structures, structural modeling is relatively simple and the contributed error is generally ignored (DiGioia Gray and Associates, 2012). Accordingly, for transmission foundation RBD, the most effort toward quantifying uncertainty in loads is aimed at variability in load events, relying on work done in support of structural code development (ASCE, 2005); 2010). The end result is representation of foundation load as a random variable subject to behavior in accordance with a defined probability density function (Fig. 5.5)

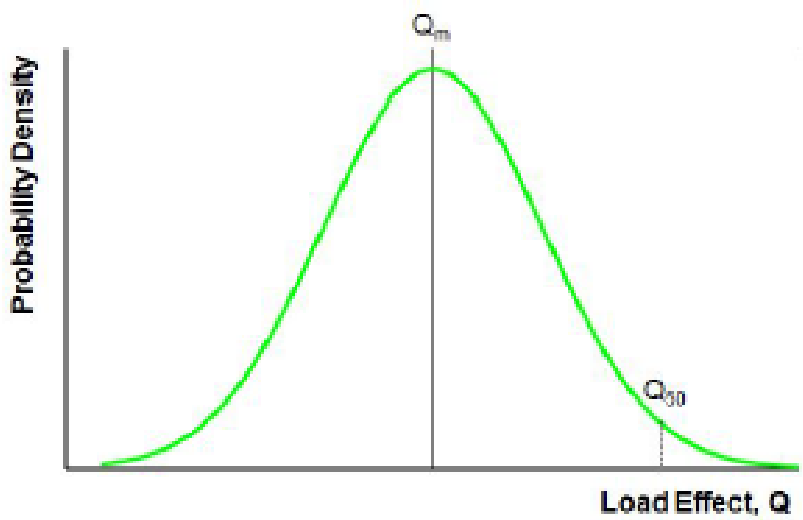


Figure 5.5 - Schematic representation of variability in foundation load

Image courtesy of (DiGioia Gray and Associates, 2012)

Resistance variability is similarly the result of collective uncertainties in a number of inputs contributing to the foundation's response to load. The greatest source of uncertainty in resistance is the strength variability of soils. However, the apparent variability of soil strength is the result of a number of conditions, including actual spatial variability of soil strength aggravated by variability in field and laboratory measurement techniques. Much effort has been exerted to quantify errors contributed by each of these factors, as this has been a focal point in the geotechnical engineering field for quite some time (Terzaghi, 1967; (Baecher, 1987); (Kulhaway, 1992), (1994); (Phoon, Kulhaway, & Grigoriu, 2000).

Design models in the form of correlations between measured values and strength parameters important to design, as well as design calculations operating upon the input parameters to yield the predicted foundation resistance, also contribute uncertainty. Uncertainty in design models should not be confused with variability however. Mathematical computations in design are, if nothing else, highly repeatable and therefore invariable. Uncertainty in design model computations arises from the question of how well or poorly the design model represents the actual behavior of the soil in question. Disparity between the model and the 'real world' is a question of compatibility and calibration and is an important consideration nonetheless.

Consideration of construction techniques and quality similarly contribute uncertainty to the prediction of resistance. Poor construction quality certainly contributes a risk of adverse performance to the degree that the constructed foundation deviates from that which was specified during design. However, this is one area where the opportunity exists to remove uncertainty generated during the geotechnical investigation and other design considerations. Particularly in the case of laterally loaded piers, excavations during construction reveal the nature of the material governing foundation performance relative to the assumptions made during design. Thus corrections may be applied if the two are substantially different. While quality assurance and control during construction can both enhance or reduce uncertainty of foundation performance, they are certainly the most controllable for the inputs affecting the design outcome.

These uncertainty considerations all contribute to the broad category of resistance uncertainty (Fig. 5.6). It is a significant task to move from the conceptual representation of resistance uncertainty to a defined and quantified assessment of resistance uncertainty. The methods for executing this are discussed later in this section.

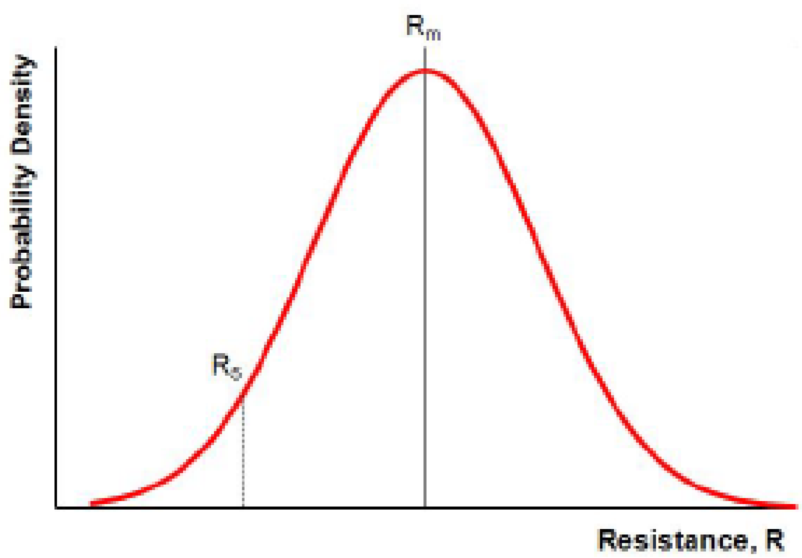


Figure 5.6 - Schematic representation of variability in foundation resistance
Image courtesy of (DiGioia Gray and Associates, 2012)

As with ASD, the goal of RBD is to provide a sufficient margin for safety between the anticipated foundation resistance and applied load to insure satisfactory performance will be achieved with high confidence. Within the RBD framework, the margin of safety is derived through explicit assessment of the variability inherent in the load and resistance components of the design equation to achieve the basic equality (Eq. 5.3.1).

$$R_5 = Q_{50} \qquad \text{Eq. 5.3.1}$$

Where:

R_5 = 5% Lower Exclusion Limit of Foundation Resistance

Q_{50} = 50 year Return Period Wind Event

Design based on Eq. 5.3.1 , represented graphically in Fig. 5.7, is used to achieve the desired low probability of failure in design.

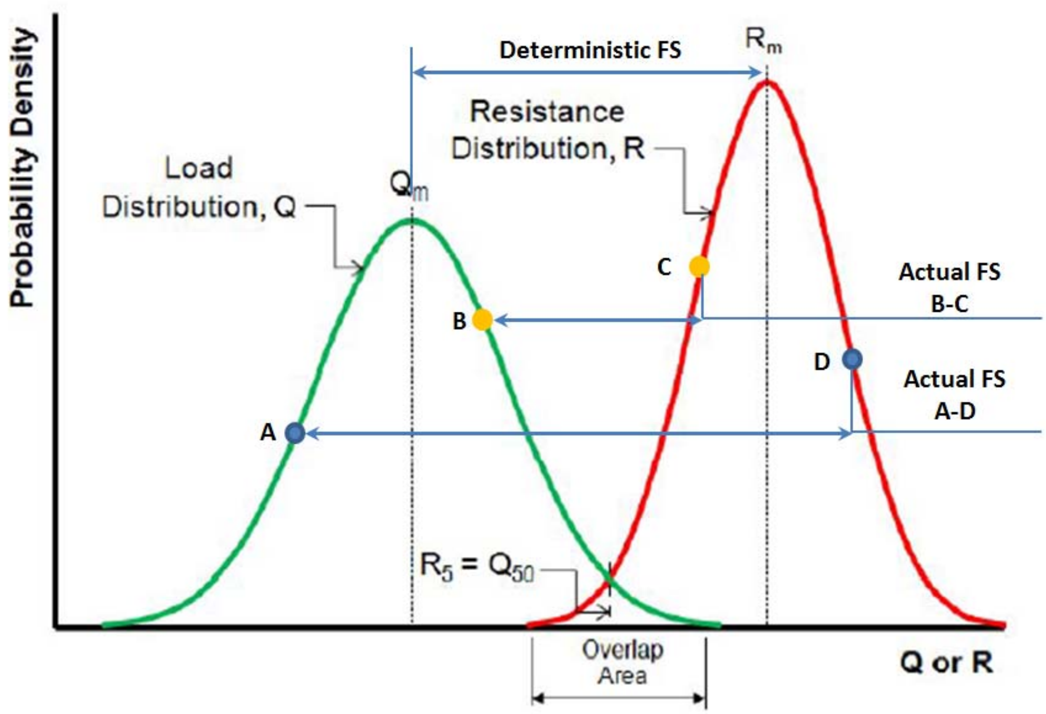


Figure 5.7 - Combination of load and resistance probability distributions
Adapted from (DiGioia Gray and Associates, 2012)

The underlying assumption of Fig. 5.7 that load and resistance vary independently of one another and failure can only occur when two low probability events coincide (e.g., a foundation with resistance near the 5% Lower Exclusion Limit (LEL) is loaded by a 50 year wind event). The probability of failure is represented graphically by the ‘overlap area,’ which is defined by the net area falling under both the resistance and load probability density curves. This represents the condition in which the foundation resistance is less than the load applied.

It should be expected that actual foundation resistance and loads may fall anywhere along the curves Q and R, and the achieved safety margin varies accordingly. Of course, the actual values will predominantly congregate near the points of greatest probability according to the representative probability density function. For illustration purposes, points A, B, C and D represent values for resistance and load that would be predicted to occur when using curves Q and R. If a lower than expected load corresponds with a higher than expected resistance, the actual margin of safety (Actual FS A-D) is greater than the mean margin of safety associated with traditional global factor of safety methods. Similarly, should a higher than expected load coincide with a weaker than expected foundation, the actual margin of safety (Actual FS B-C) is lower than the mean margin of safety. Therefore, the actual margin of safety is a constantly varying number with a mean value equal to the mean margin of safety (Fig. 5.8).

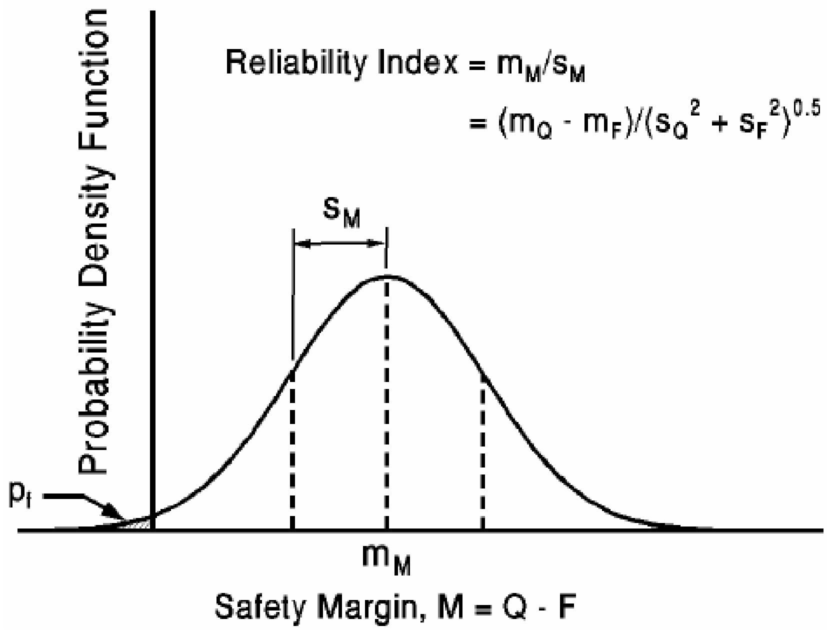


Figure 5.8 - Graphical relationship between β and P_f

Image courtesy of (Phoon, Kulhaway, & Grigoriu, 1995)

From this perspective, the probability of failure is represented by the area under the margin of safety probability density function with a negative value. An increase in the margin of safety shifts the curve in Fig. 5.8 to the right, reducing the probability of failure.

5.4 Characterization of Reliability:

The probability of failure, p_f, is a useful and intuitive value for practical understanding of the goal of RBD methods. However, it is a somewhat cumbersome number to handle, particularly when the low probabilities of failure desirable in most engineering applications are of interest. As has also been noted, the term 'failure' caries a negative connotation not indicative of the true nature of failure in engineering terms (Phoon, Kulhaway, & Grigoriu, 1995). As a matter of convenience, the reliability index, β, may be substituted for p_f as a means of characterizing design risk. The relationship between β and p_f is an inverse relationship represented by Eq. 5.4.1.

$$\beta = -\phi^{-1}(p_f) \qquad \text{Eq. 5.4.1}$$

where:

$-\phi^{-1}$ = Inverse standard normal probability density function

Values for β in typical engineering applications lie between 1 and 4, corresponding to values of p_f that range from 15% to 0.003% and are provided in Table 5.2. Although permanent structures generally seek β of 2 or greater depending on their importance.

Table 5.2 - Relationship between β and pf with expected performance Adapted from (U.S. Army Corps of Engineers, 1997)

Reliability Index, β	Probability of Failure, p_f	Expected Performance
1	0.159	Hazardous
1.2	0.115	
1.4	0.0808	
1.6	0.0548	Unsatisfactory
1.8	0.0359	
2	0.0228	Poor
2.2	0.0139	
2.4	0.0082	
2.6	0.0047	Below Average
2.8	0.0026	
3	0.0013	Above Average
3.2	0.00069	
3.4	0.00034	
3.6	0.00016	
3.8	0.00007	
4	0.00003	Good

For the special case where both load and resistance, Q and R, are represented by normal distributions, β is calculated as:

$\beta = m_M/s_M$ Eq. 5.4.2

$$= \frac{m_R - m_Q}{\sqrt{s_R^2 + s_Q^2}}$$ Eq. 5.4.3

Where:

m_M = Mean safety margin
s_M = Standard deviation of safety margin
m_R = Mean resistance
m_Q = Mean load
s_R = Standard deviation of resistance
s_Q = Standard deviation of load

Eq. 5.4.3 demonstrates that an increase in the mean margin of safety ($m_M = m_R - m_Q$) or a reduction in the standard deviation associated with Q or R will increase β, the equivalent of a reduction in p_f. However, this special case is of limited use for geotechnical engineering where high variance parameters modeled with normal distributions may yield negative values--an impossibility. It is therefore useful, and common, to use log-normal distributions to represent input parameters, limiting model values to positive numbers. Eq. 5.4.3 is adapted for the special case of to log-normal parameters by Eq. 5.4.4.

$$\beta = \frac{\ln\left[m_R/m_Q\sqrt{(1+COV_Q^2)/(1+COV_R^2))}\right]}{\sqrt{\ln[(1+COV_Q^2)(1+COV_R^2)]}}$$ Eq. 5.4.4

Where:

COV_Q = Coefficient of variation of load
COV_R = Coefficient of variation of resistance

This equation is often approximated as Eq. 5.4.5 (Phoon, 2004).

$$\beta = \frac{\ln(m_M)}{\sqrt{COV_Q^2 + COV_R^2}} \qquad \text{Eq. 5.4.5}$$

Estimation of reliability, either in terms of β or p_f is the essence of the goal for probabilistic design methods. It is also the result of much computational and engineering effort both in the selection of the appropriate reliability objective and in derivation of the actual value.

5.4.1 *Calibration Methods for Reliability Based Design:*

Within the larger effort to develop RBD guidelines for foundation design, much of the focus has been directed toward development of load and resistance factors within a defined framework (e.g., single factor, multiple factor (LRFD)) (Phoon, Kulhaway, & Grigoriu, 1995); (Paikowsky, 2004); (Allen, 2005). The process of calibration is characterized by two distinct steps: development of a framework within which to compute reliability, and calibration of reliability factors to achieve the desired probabilistic result.

Computation of reliability is achieved through assessment of variability imparted by each design input to derive the gross system reliability accounting for the influence of each input. A number of techniques have been employed, both in closed form solution formats such as the Mean Value First Order Second Moment Method (MVFOSM) and First-Order Reliability Method (FORM), as well as open form simulations, i.e., Monte Carlo simulations.

The methods for calibration of load and resistance factors are variable in their complexity, applicability to various design scenarios, and ultimately their place in the continued development of RBD methods. Early efforts tend to focus on calibration to match margins of safety achieved by global factor of safety methods (Eurocode, 1993). Others utilize probabilistic methods to assess foundation resistance on a broad scale (e.g., single resistance factor approach) (AASHTO, 2007), (DiGioia Gray and Associates, 2012). Alternatively, some have conducted probability analyses on individual design model inputs to implement multiple resistance factor approaches (Phoon, Kulhaway, & Grigoriu, 1995).

5.5 Reliability Computations

Calculation of predicted reliability for a particular design model is a two-step process in which the variability of the model inputs (friction angle, lateral modulus, etc.) is determined and subsequently the probabilistic behavior of the outcome is evaluated (i.e., probability of failure). Presumably, the variability of model inputs is known, having been determined through analysis of the available dataset. Evaluation of variability in the design outcome is less straightforward as this is dependent upon the variability derived from multiple inputs, each with differing influence on the aggregate probability of a particular outcome. Depending on the number of input parameters and their statistical behavior (variability, numerical and spatial variability, skewness, etc.), the viability of available analysis techniques changes. In the interest of reduced computational effort, closed form analysis methods, Mean Value First Order Second Moment (MVFOSM) and the First Order Reliability Method (FORM) are desirable and have been employed extensively (Barker, et al., 1991); (Phoon, Kulhaway, & Grigoriu, 1995); (Allen, 2005); (DiGioia Gray and Associates, 2012).

5.5.1 *Mean Value First Order Second Moment:*

Of the closed form analysis methods, Mean Value First Order Second Moment (MVFOSM) is the simplest. Conceptually, it is derived from a Taylor Series expansion of a design model expression $f(X_i)$. The function, f, is dependent upon some finite number of input variables, each defined by their first and second statistical moments (mean and variance), hence the term *second moment*. The design function is linearized through evaluation at the mean values of the input variables, signaled by the term *mean value.* The resulting mean and variance of the design function, f, is determined from a Taylor Series expansion of the base expression which is truncated at the first order terms, hence the term *first order.* This approach is employed by Barker, et al (1991) for initial calibrations of load and resistance factors for bridge foundations.

Expansion of the important identities and equations pertinent to the simplest form of MVFOSM are provided here. A more complete definition of the method is provided by Griffiths, Fenton & Tveten (2002).

The MVFOSM approach employs a series of statistical identities in which the random variable, X, is represented by a known Probability Density Function, $f_X(x)$. If, for example, g(X) is a function of the random variable X, the expected value of g(x) is defined by Eq. 5.5.1:

$$E[g(X)] = \int_{-\infty}^{\infty} \boldsymbol{g(x) f_X(x) dx} \qquad \text{Eq. 5.5.1}$$

The statistical moments are then defined:

First Moment: Mean

$$\boldsymbol{\mu_X = E[X] = \int_{-\infty}^{\infty} x f_X(x) dx} \qquad \text{Eq. 5.5.2}$$

Second Moment: Variance

$$V[X] = \quad \sigma_X^2 = E[(X - \mu)^2] = \int_{-\infty}^{\infty}(x - \mu)^2 f_X(x)dx \qquad \text{Eq. 5.5.3}$$

Where *f* (X,Y) is a function of two uncorrelated random variables, X and Y, the Taylor Series expansion about the mean values, μ_x and μ_y, and truncated at the linear terms is:

$$f(X, Y) = f(\mu_X, \mu_Y) + (X - \mu_X)\frac{\partial f}{\partial x} + (Y - \mu_Y)\frac{\partial f}{\partial y} \qquad \text{Eq. 5.5.4}$$

Hence, the expected value of *f* (X, Y) is:

$$E[f(X, Y)] = f(\mu_X, \mu_Y) \qquad \text{Eq. 5.5.5}$$

The remaining terms define variance:

$$V[f(X, Y)] = V[(X - \mu_X)\frac{\partial f}{\partial x} + (Y - \mu_Y)\frac{\partial f}{\partial y}] \qquad \text{Eq. 5.5.6}$$

Expansion of the second moment for two variables is defined by Eq. 5.5.7 where the generic version for *n* variables is provided by Eq. 5.5.8:

$$V[f(X, Y)] = \left(\frac{\partial f}{\partial x}\right)^2 V[X] + \left(\frac{\partial f}{\partial y}\right)^2 V[Y] \qquad \text{Eq. 5.5.7}$$

and for *n* variables:

$$V[f(X_1, X_2, \ldots X_n)] = \sum_{i=1}^{n}\left(\frac{\partial f}{\partial x_i}\right)^2 V[X_i] \qquad \text{Eq. 5.5.8}$$

The linearization of the design function and neglect of higher order terms are limitations of the MVFOSM method. Similarly, the method lacks a means to incorporate spatial variability observed in design inputs. Both of these limitations may contribute significant error to MVFOSM computations (Griffiths, Fenton, & Tveten, 2002). Since geotechnical parameters generally exhibit higher order variability and do not follow normal distributions,, these unmet assumptions have led the profession away from MVFOSM to more advanced techniques.

5.5.2 *First Order Reliability Method:*

The FORM offers more robust assessments of reliability behavior where the approximations of MVFOSM may contribute excessive inaccuracy. The FORM has been successfully employed for structural design methods (Ravindra & Galambos, 1978) (Ellingwood & Galambos, 1980) and to foundations (Paikowsky, 2004) (Allen, 2005), including specific application to transmission line foundations ((Phoon, Kulhaway, & Grigoriu, 1995) (Phoon, Kulhaway, & Grigoriu, 2003) (DiGioia Gray and Associates, 2012)).

The FORM, requires knowledge of the first and second statistical moments (mean and variance) of the load and resistance variables as well as their probability distribution function's shape (normal, lognormal, beta, etc.). Unlike MVFOSM, the FORM is an iterative method in which the probability of failure is formulated in a geometric evaluation of the minimum distance (β) between the failure envelope defined by the limit state P (Q, F) = 0 and a joint Probability Density Function (PDF), $p_{Q,F}(q,f)$ (Figs. 5.5.1 & 5.5.2) where Q and F are random variables representing load and resistance, respectively. The coordinate yielding the minimum reliability (β) is the termed the "design point."

Defined in Cartesian space, the performance function (Eq. 5.5.9) defines a linear boundary between the safe and failure domains. Where the performance function is greater than zero, the design is safe with resistance greater than load; the opposite is true when the function is less than zero. The joint probability density function formed by the resistance and load variables, represented by contour lines in Fig. 5.9

is a two dimensional surface. The area of the surface within the failure domain is equal to the probability of failure.

$$P\,(Q, F) = Q - F \qquad \text{Eq. 5.5.9}$$

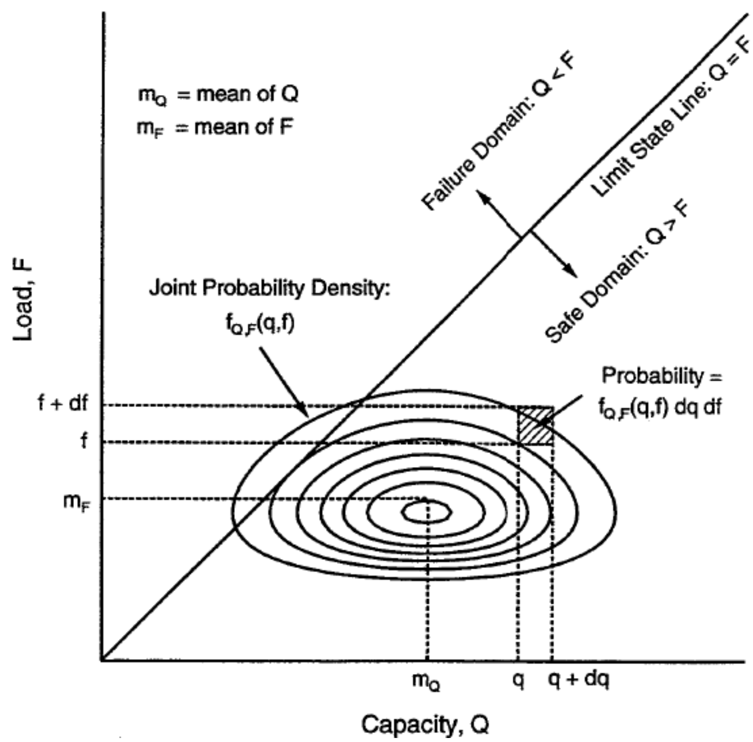


Figure 5.9 - Probability of failure defined in original Cartesian space

Image courtesy of (Phoon, Kulhaway, & Grigoriu, 1995)

The joint probability distribution, if Q and F are statistically independent, is defined by Eq. 5.5.10. Summation of the probability of occurrence for each pair of Q and F within the failure domain yields the probability of failure.

$$p_{Q,F}\,(q,f) = P_Q(q)\; f_F(f) \qquad \text{Eq. 5.5.10}$$

Beyond theoretical applications, the actual failure domain may be highly non-linear, lending significant complexity to reliability computations within Cartesian space. Computation of the failure domain surface area may not be possible in closed form.

Similarly, as the number of design variables increases, integration of the failure volume becomes exceedingly difficult. For these reasons, the common implementation of FORM includes transformation of the original independent random variables into standard normal random variables as defined by their mean and standard deviations (Fig. 5.10). This transformation is helpful due to the comparative ease in integration of standard normal distributions. The joint probabilities of bivariate standard normal probability density functions, mean of zero and standard deviation of one, are represented by a double symmetric contour surface in transformed space.

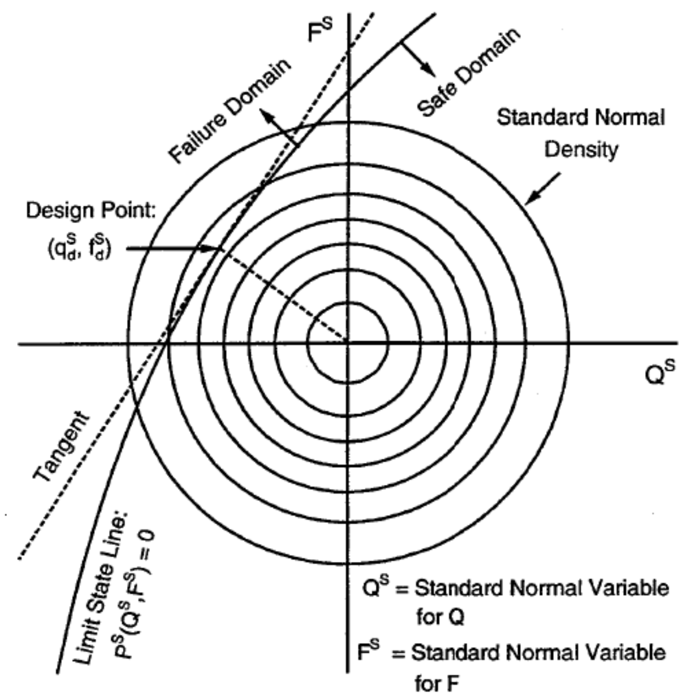


Figure 5.10 - First Order Reliability Method

Image courtesy of (Phoon, Kulhaway, & Grigoriu, 1995)

Transformation to standard normal space similarly perturbs the limit state line to a non-linear curve. The retained non-linearity of the limit state function is the source of a common simplification within FORM where the limit state is approximated by the tangent to the limit state curve at the point nearest to the origin (design point). This simplification successfully captures the portion of the failure volume where the joint probability function provides the greatest contribution to the probability of failure -at the design point. As the tangent limit state line deviates from the actual curved line on the periphery of the joint probability function, the relative contribution to the probability of failure is low and so is the corresponding error in the computed probability of failure (Phoon, Kulhaway, & Grigoriu, 1995). Determination of the actual design point, (q^s_d, f^s_d), is an iterative process. Upon convergence, the reliability index can be found as:

$$\boldsymbol{\beta}_{FORM} = \sqrt{(\boldsymbol{q}_d^S)^2 + (\boldsymbol{f}_d^S)^2} \qquad \text{Eq. 5.5.11}$$

Thus, computation of reliability within the FORM becomes an effort to determine the location of the design point, which may be carried out in accordance with the recommendations of Rackwitz and Fiessler (1978):

1. Make an initial assumption on the location of the design point in the Cartesian space:

   $q_d = m_Q$ Eq. 5.5.12

   $f_d = m_F$ Eq. 5.5.13

   Where: m_Q = mean of the resistance variable

   m_F = mean of the load variable

2. Compute the first and second moments of the standard normal probability distribution function of resistance:

$$\boldsymbol{m_{QN}} = \boldsymbol{q_d} - \phi^{-1}[\boldsymbol{F_Q(q_d)}]\boldsymbol{s_{QN}} \qquad \text{Eq. 5.5.14}$$

$$\boldsymbol{s_{QN}} = \boldsymbol{\Psi}\frac{\{\phi^{-1}[F_Q(q_d)]\}}{f_Q(q_d)} \qquad \text{Eq. 5.5.15}$$

Where: Ψ (·) = standard normal probability density function

ϕ^{-1} = Inverse standard normal probability density function

$\boldsymbol{m_{QN}}$ = mean of equivalent normal distribution for resistance

$\boldsymbol{s_{QN}}$ = standard deviation of equivalent normal distribution for resistance

3. Compute the first and second moments of the standard normal probability distribution function of load:

$$\boldsymbol{m_{FN}} = \boldsymbol{f_d} - \phi^{-1}[\boldsymbol{F_F(f_d)}]\boldsymbol{s_{FN}} \qquad \text{Eq. 5.5.16}$$

$$\boldsymbol{s_{FN}} = \boldsymbol{\Psi}\frac{\{\phi^{-1}[F_F(f_d)]\}}{f_F(f_d)} \qquad \text{Eq. 5.5.17}$$

Where: $\boldsymbol{m_{FN}}$ = mean of equivalent normal distribution for load

$\boldsymbol{s_{FN}}$ = standard deviation of equivalent normal distribution for load

4. Transform the random variables (Q, F) to standard normal space:

$$\boldsymbol{Q^S} = (\boldsymbol{Q} - \boldsymbol{m_{QN}})/\boldsymbol{s_{QN}} \qquad \text{Eq. 5.5.18}$$

$$\boldsymbol{F^S} = (\boldsymbol{F} - \boldsymbol{m_{FN}})/\boldsymbol{s_{FN}} \qquad \text{Eq. 5.5.19}$$

Where: $\boldsymbol{Q^S}$ = random variable for resistance in standard normal space

$\boldsymbol{F^S}$ = random variable for load in standard normal space

5. Define the performance function P in terms of standard normal variables Q^S and F^S.

   $\boldsymbol{P(Q, F) = Q - F}$

   $= (s_{QN}\ Q^S + m_{QN}) - (s_{FN}\ F^S + m_{FN})$

   $= P^S\ (Q^S, F^S)$ Eq. 5.5.20

   Where: $\boldsymbol{P^S}$ = transformed performance function in standard normal space

6. Compute the trial location of the design point (q_d, f_d) in standard normal space.

   $\boldsymbol{q_d^S}$ = $(q_d - m_{QN})/s_{QN}$ Eq. 5.5.21

   $\boldsymbol{f_d^S}$ = $(f_d - m_{FN})/s_{FN}$ Eq. 5.5.22

7. Determine the partial derivatives $\boldsymbol{\partial P^S/\partial Q^S}$ and $\boldsymbol{\partial P^S/\partial F^S}$ at $\boldsymbol{q_d^S}$ and $\boldsymbol{f_d^S}$.

   For the performance function defined in Eq. 5.5.20 the result is noted below as an example, but differing performance functions will take different forms.

   $\boldsymbol{\partial P^S/\partial Q^S = s_{QN}}$ Eq. 5.5.23

   $\boldsymbol{\partial P^S/\partial F^S = s_{FN}}$ Eq. 5.5.24

8. Compute a new trial design point.

   $$\boldsymbol{q_{di}^S = \frac{\left[q_d^S \cdot \partial P^S/\partial Q^S + f_d^S \cdot \partial P^S/\partial F^S - P^S\left(q_d^S, f_d^S\right)\right] \partial P^S/\partial Q^S}{\left(\partial P^S/\partial Q^S\right)^2 + \left(\partial P^S/\partial F^S\right)^2}} \quad \text{Eq. 5.5.25}$$

   $$\boldsymbol{f_{di}^S = \frac{\left[q_d^S \cdot \partial P^S/\partial Q^S + f_d^S \cdot \partial P^S/\partial F^S - P^S\left(q_d^S, f_d^S\right)\right] \partial P^S/\partial F^S}{\left(\partial P^S/\partial Q^S\right)^2 + \left(\partial P^S/\partial F^S\right)^2}} \quad \text{Eq. 5.5.26}$$

9. Iterate until a stable design point is found.

Determination of computed reliability using FORM provides a means to evaluate relatively complex design problems in an efficient manner. The iterative method and computational intensity are best suited for implementation in a computer program.

5.5.3 *Monte Carlo Simulation:*

For many geotechnical engineering problems, the limit state function, number of input variables and their associated probability density functions are sufficiently complex that it becomes difficult to evaluate reliability by FORM or incompatibilities exist with the simplifying equations of MVFOSM. Monte Carlo computer simulations offer a flexible means to estimate β where complexity limits the use of closed form methods. Monte Carlo methods have been employed as a method to validate reliability factors derived from alternate analysis methods (Allen, 2005).

Each design input is characterized by a representative probability density function (e.g., normal, lognormal, beta, etc.) with defined fitting parameters (mean, standard deviation, etc.). Random number generators adhering to the representative probability density function for each input are employed to derive a result for the limit state equation for each set of random inputs. The result of each simulation becomes an entry in a results database. After a large number of computer simulations are run, it becomes possible to employ curve fitting methods to determine the representative probability density function of the limit state results (Fig. 5.11). Thus β is estimated in a manner that, at times, provides greater ease and accuracy than could otherwise be achieved with MVFOSM or FORM.

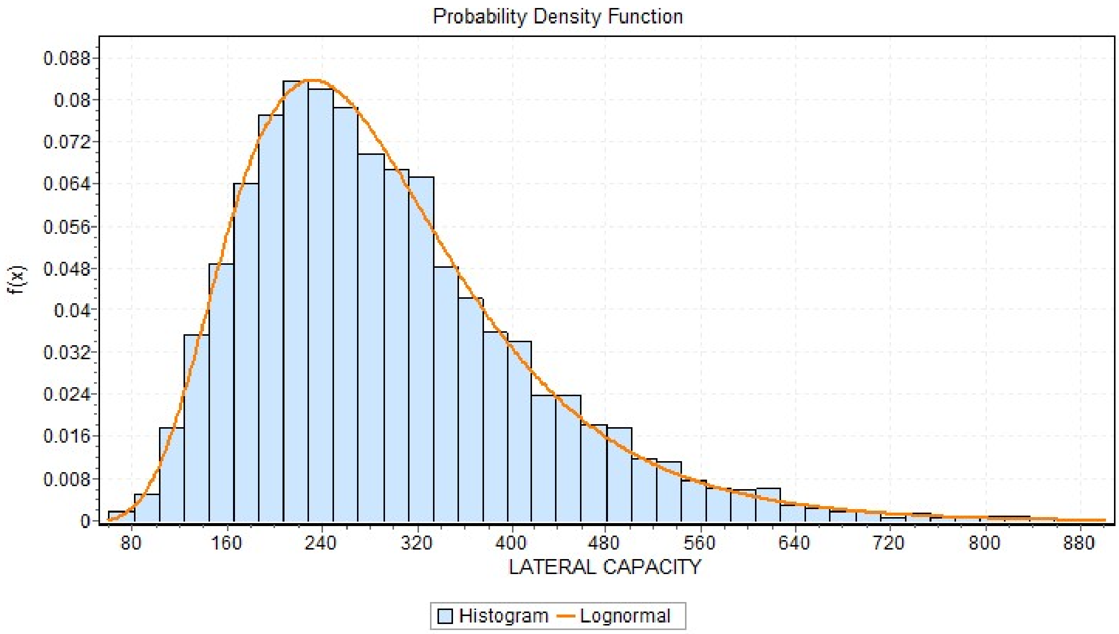


Figure 5.11 - Monte Carlo Simulation results PDF

for lateral pier capacity after 2,000 simulation runs

Monte Carlo simulations become useful when limit state equations are highly non-linear or iterative analyses are required to derive results. Further, with Monte Carlo simulations there is enhanced flexibility to consider multiple probability density functions and variable dependencies (variables that vary independently or as linked parameters). Just as closed form methods employing simplifying equations may introduce error, the curve fitting methods used to describe input and limit state variability in Monte Carlo simulations are sources of error that may unintentionally influence reliability predictions.

5.6 Reliability Calibrations:

Reliability calibration is the exercise in which an individual design or design model is adjusted to achieve $p_f \approx p_T$ in which p_f is the design probability of failure and p_T is the target probability of failure. For individual design solutions, calibration is achieved through adjustment of foundation dimensions. Design models are calibrated through assignment of resistance factors to yield the desired target reliability from the design equation. Reliability computations are carried out either by computational methods such as MVFOSM or FORM or alternatively by simulation methods (i.e., Monte Carlo simulations). For individual designs, these computations can be onerous, and generally RBD is carried out at the code level. However, large or atypical projects may warrant RBD on an individual design basis.

Selection of an appropriate p_T may be done through any number of methods. Ideally, a cost optimization study in which construction and failure costs are evaluated against the target probability of failure will yield the best economy. In theory, low construction cost (i.e., lower reliability design) will yield higher failure costs due to an increased failure rate. Alternatively, higher construction cost will minimize failure costs. Theoretically, there is a p_T value that yields the lowest combined lifecycle total construction and failure cost (Fig. 5.12). However, real world determination of failure costs is difficult to predict, given uncertainties about system damage after a component failure and especially where loss of human lives is possible (Phoon, Kulhaway, & Grigoriu, 2000).

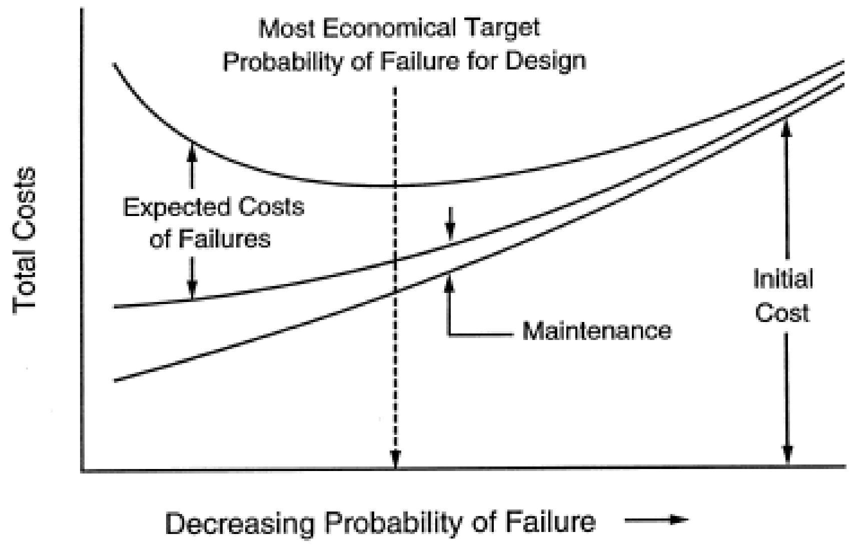


Figure 5.12 - Cost benefit analysis of pT

Image courtesy of (Phoon, Kulhaway, & Grigoriu, 2000)

Selection of p_T may also come from an assessment of historic failure rates for comparable industry segments Fig. 5.13. Comparison of measured rates of failure with those derived from design calculations has been shown as a potential source of inaccuracy, with actual failure rates generally higher than those calculated in design. This disparity is attributed to construction methodologies and workmanship errors that increase uncertainties beyond those normally considered during design. Research indicates that 10-20% of failures observed in civil structures are attributable to inadequate assessments of load and resistance (CIRIA, 1977). This observation has led others to increase design assessments of p_f by one order of magnitude to better protect against uncertainties introduced during construction and other elements outside the purview of the design engineer (Phoon, Kulhaway, & Grigoriu, 2003).

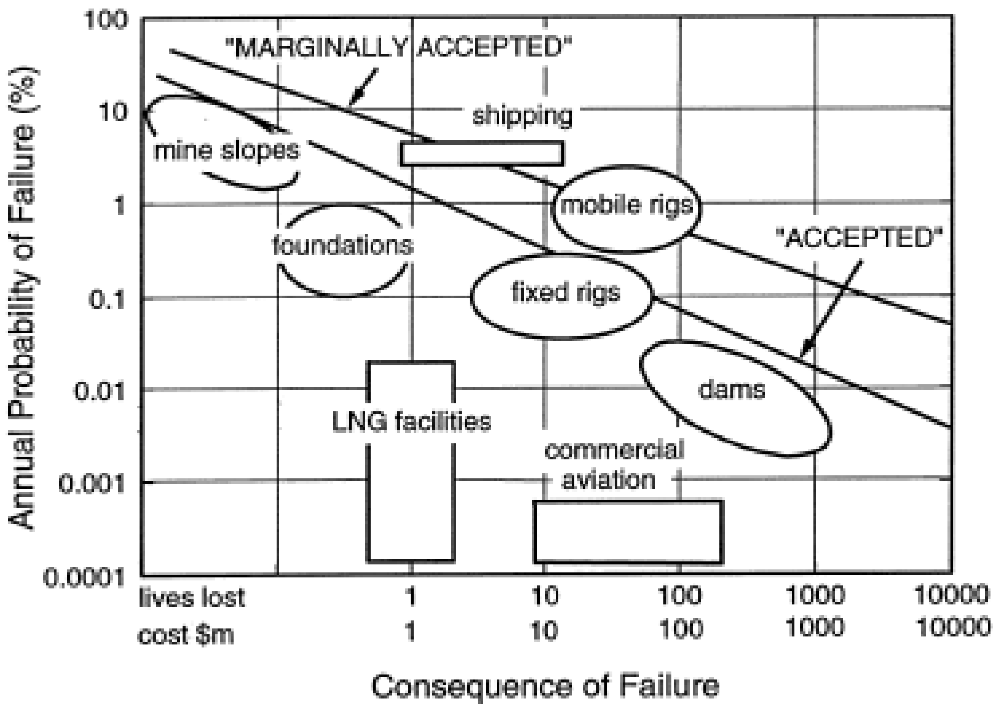


Figure 5.13 - Failure rates by industry

(CIRIA, 1977)

For the purposes of code development, continuity among evolving design methods is an important consideration that has yielded the most common approach to p_T calibrations for RBD methodologies. Early implementation of RBD methodologies in situations where past practice has employed ASD methods has generally relied on some form of calibration to achieve compatibility with the previous methodology (Barker, et al., 1991) (Phoon, Kulhaway, & Grigoriu, 1995) (AASHTO, 2007). Methods of calibration to match ASD vary depending on the availability of statistical data to support reliability computations, and include (a) calibration by fitting, and (b) calibration by reliability theory.

5.6.1 *Calibration by Fitting*

Calibration by fitting does not include a statistical assessment of reliability and is largely an exercise to restructure existing ASD calculation methods into a format compatible with future implementation of RBD. Within the designated performance function, load and resistance factors are calibrated to achieve a factor of safety equivalent to that of the preceding ASD performance function. However, an evaluation of the level of reliability implicit in the target factor of safety is not a component of this approach. Calibration by fitting stops short of the primary objective of RBD, consistent reliability across designs, and is recognized as an incremental step toward a larger goal.

Calibration by fitting offers two advantages to code developers: continuity with past design methods in terms of the results derived from both approaches and a method to fill in statistical gaps with the collective experience of the profession. In the fervor to adopt statistically rigorous design techniques, continuity between new and former design methods is an essential quality. Radical changes in design methodology are counterproductive when trying to gain acceptance from practitioners who have extensive successful experience utilizing an existing methodology. Comparable results across successive design methods offer practitioners the ability to scrutinize their results with the benefit of past experience. Significant differences among design results will generally yield skepticism toward the newer methodology and threaten or at least slow its acceptance.

Historically, calibration by fitting has been employed as a supplemental technique for code development (Barker, et al., 1991) (Phoon, Kulhaway, & Grigoriu, 1995) (Phoon, Kulhaway, & Grigoriu, 2003). The extent of data available to code developers on differing design scenarios (e.g., drilled piers in uplift vs. laterally loaded drilled piers) can depend on the prevalence of a particular scenario in practice. Design situations that are rare may not offer the amount of data necessary for rigorous statistical analysis at the time of development for a particular code document. Calibration by fitting offers code developers a means to draw on industry experience to fill gaps in data and common design scenarios can benefit from more extensive statistical treatment.

5.6.2 *Calibration by Reliability Theory*

Calibration by reliability theory is the expression of the driving mathematical principles behind the move toward RBD. Where the extent of data is suitable, calibration by reliability theory offers similar ties to previous practice as calibration by fitting, but does so by matching the p_f implicit in a range of design scenarios in lieu of matching the FS. The result is consistent reliability across the domain of interest.

Existing inconsistencies in p_f across the domain of interest complicate the task of calibration by reliability theory and a coherent analysis framework is required to insure appropriate results are derived. A generalized approach adopted by a number of authors and organizations is outlined below (CIRIA, 1977) (Ellingwood & Galambos, 1980) (Phoon, Kulhaway, & Grigoriu, 1995) (Phoon, Kulhaway, & Grigoriu, 2003):

a) Conduct a parametric study to determine the variation of reliability level for each parameter in the design problem. Parameters of interest will vary depending on the design scenario, but will generally include soil strength parameters, their associated statistical moments and physical design aspects such as foundation depth and diameter.

b) Subdivide the analysis space into a series of smaller calibration domains (Fig. 5.14). The size of the smaller calibration domains depends on the sensitivity of the design outcome to the parameter of interest. Parameters that heavily influence results should have a correspondingly smaller calibration domain compared to parameters with less influence.

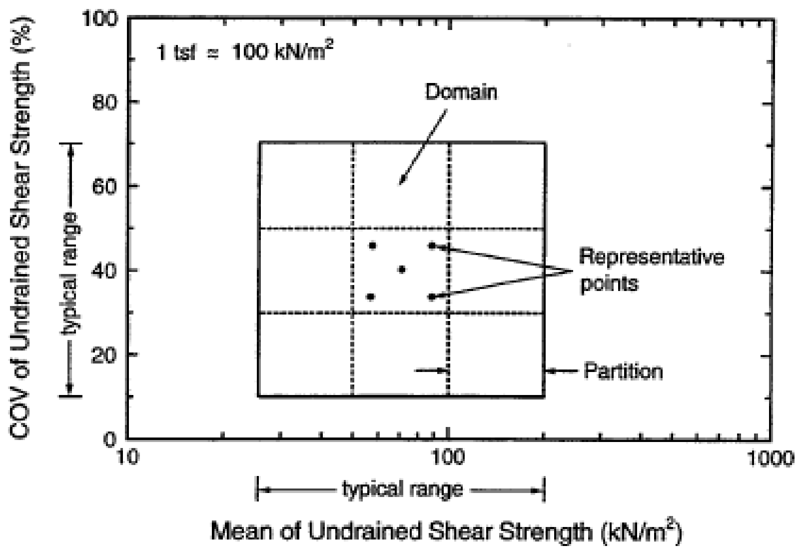


Figure 5.14 - Subdivision of an analysis domain for RBD
(Phoon, Kulhaway, & Grigoriu, 1995)

c) Select representative points from each analysis domain with each point representing a set of design parameters.

d) Derive foundation designs for each set of design parameters across the range of analysis domains for the existing standard of practice (e.g., ASD) and the proposed RBD format employing a set of trial resistance factors. Evaluate the level of reliability achieved by each foundation design through application of a reliability algorithm (MVFOSM, FORM, and Monte Carlo Simulation).

e) Efficacy of each iteration is evaluated based on minimization of an objective function (Eq. 5.6.1):

$$H(\psi_\varphi, \psi_c, \psi_E,\ldots) = \sum_{i=1}^{n}(\beta_i - \beta_T)^2 \qquad \text{Eq. 5.6.1}$$

Where:

$\psi_\varphi, \psi_c, \psi_E,\ldots$ = Resistance factors on design model inputs

Generally $0 \leq (\psi_\varphi, \psi_c, \psi_E,\ldots) \leq 1$

βi = Reliability index for i^{th} point in domain

β_T = Target reliability index

n = Number of points in calibration domain

f) Adjust resistance factors and iterate steps 4 and 5 until the objective function is minimized, which is an indication that some degree of uniformity in the level of reliability within the domain has been achieved. Uniformity of the reliability level is evaluated by Eq. 5.6.2.

$$\Delta\beta = (H/n)^{0.5} \qquad \text{Eq. 5.6.2}$$

$\Delta\beta$ = Average deviation from target reliability index within the calibration domain.

g) Repeat steps 3 to 6 for each calibration domain.

The results of the calibration process will yield a set of resistance factors for each calibration domain. If the selection of domain size was appropriate, the resistance factors should vary between domains and the reliability index uniformity should be small. Limited variability of resistance factors across the calibration domains is an indication that the calibration domains may be consolidated to reduce the number of resistance factors. Alternatively, a lack of reliability uniformity within each calibration domain is an indication that further subdivision could be warranted (Figs. 5.15 & 5.16).

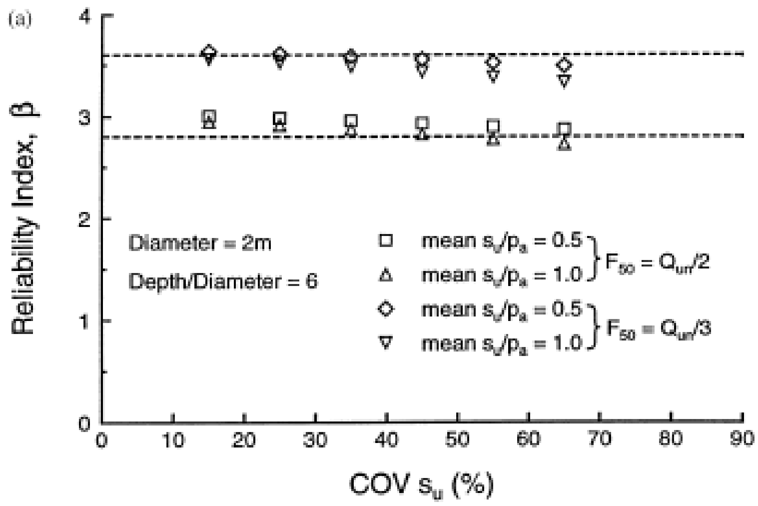


Figure 5.15 - Non-uniformity in reliability indices from ASD sample designs (Phoon, Kulhaway, & Grigoriu, 2000)

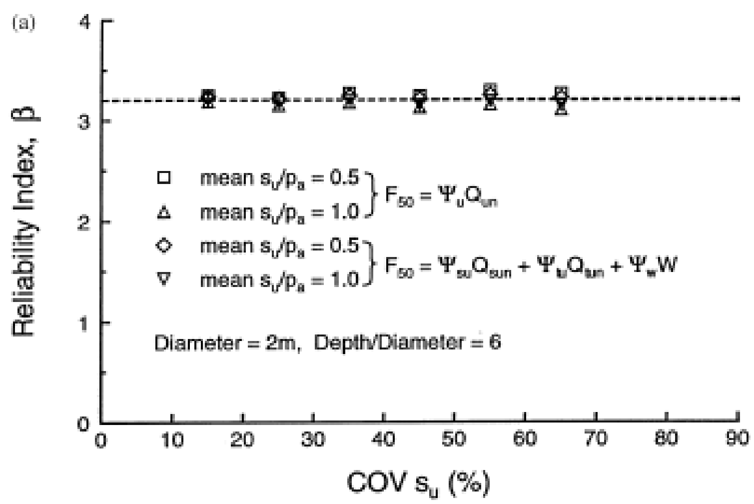


Figure 5.16 - Uniformity in reliability indices from RBD after calibration (Phoon, Kulhaway, & Grigoriu, 2000)

Often calibration of RBD methodologies requires a mixture of fitting and reliability theory computations. The amount of data required to perform a purely statistical calibration is large. Particularly near the extreme ends of the analysis domain, the data required for a statistical calibration may not exist. Thus, code developers have relied upon both methods to derived reliability factors across the entire analysis domain (Barker, et al., 1991; Phoon, Kulhaway, & Grigoriu, 2000;

Phoon, Kulhaway, & Grigoriu, 2003). As available datasets have developed over time, further enhancements to reliability computations have been made to reflect new information (AASHTO, 2007; Paikowsky, 2004). The ability to incorporate new data into a standing statistical framework as a basis for continued refinement is one of the primary benefits of RBD over deterministic design methods.

6 EXISTING RBD DESIGN CODES

To provide a general indication of how different RBD methodologies address site specific variability and stratification as applied to the specific instance of laterally loaded drilled pier foundations, three documents are examined:

a) Eurocode 7: Geotechnical Design, EN-1997-1 (British Standards, 2004)
b) Drilled Shafts: Construction Procedures and LRFD Design Methods, FHWA-NHI-10-016 (Brown, Turner, & Castelli, 2010)
c) Transmission Structure Foundation Design Guide (DiGioia Gray and Associates, 2012)

The selected documents are not a comprehensive representation of RBD documents available to date. Rather, they represent a cross section of available documents that are widely accepted, that address lateral loading of drilled pier foundations, and that provide different industry perspectives on RBD analyses. The computational effort exerted toward development of reliability indices for laterally loaded drilled piers within each document reflects the importance this loading condition caries within each document's target industry.

As a transmission industry specific document, *Transmission Structure Foundation Design Guide* explicitly addresses high eccentricity laterally loaded drilled piers, having derived reliability analyses from full scale load testing. *Eurocode 7* is an international standard adopted and modified by European nations for general civil construction works. A general code document, *Eurocode 7*, provides a specific framework for reliability analyses of laterally loaded piers either with code provided reliability factors or through a recommended reliability analysis performed by the design engineer. *Drilled Shafts: Construction Procedures and LRFD Design Methods* (abbreviates as FHWA) is largely intended for the design of drilled pier bridge foundations and therefore emphasis is placed on axial loads. However, in contrast to other bridge foundation design code documents, FHWA also addresses laterally loaded piers. Resistance factors for lateral loading are derived from unspecified computations conducted by the code authors, supplemented by engineering judgment. Target levels of reliability and the conditions under which reliability is considered vary by document.

6.1 Eurocode 7: Geotechnical Design, EN-1997-1

Eurocode 7 is the geotechnical component of a larger family of Eurocode documents published by the European Committee for Standardization (CEN) on behalf of the of the European Union (EU). The geotechnical code EN-1997-1 is the emphasis of this study, however, it draws upon and references a series of documents in the Eurocode family (Table 6.1).

Generally, each EU country adopts a national annex of the Eurocode standards in which general practices, resistance and load factors are assigned to reflect the reliability standards of the nation. For the purposes of this study, the United Kingdom British Standard BS EN1997-1:2004 is the source of all analysis techniques and reliability factors.

Table 6.1 - Eurocode Family of Geotechnical Code Documents

Document Number	Document Title
EN 1990:2002	Eurocode: Basis of Structural Design
EN 1991	Eurocode 1: Actions on Structures
EN 1997-1	Eurocode 7: Geotechnical Design - Part 1: General Rules
EN 1997-2	Eurocode 7: Geotechnical Design – Part 2: Ground Investigation and Testing
EN 1536:1999	Eurocode: Execution of Special Geotechnical Work: Bored Piles
EN 1537:1999	Eurocode: Execution of Special Geotechnical Work: Ground Anchors
EN 12063:1999	Eurocode: Execution of Special Geotechnical Work: Sheet Pile Walls
EN 12699:2000	Eurocode: Execution of Special Geotechnical Work: Displacement Piles
EN 14199	Eurocode: Execution of Special Geotechnical Work: Micropiles

The Eurocode document family is intended as a comprehensive reliability based approach to general civil engineering design and construction works. In theory, the chief advantage of the Eurocode framework is compatibility in reliability computations across each design discipline (e.g., structure and foundation design). As a minimum standard of practice EN-1990:2002 prescribes a multiple partial factor design framework with established target levels of reliability (Table 6.2). Latitude is granted to individual countries and design engineers with regard to the complexity of the reliability computations employed in the calibration of partial factors as discussed in Section 5 of this document and Figure 6.1.

Table 6.2 - EN-1990:2002 Target Reliability

Limit State	Target Reliability Index (β)	
	1 Year	50 Years
Ultimate	4.7	3.8
Serviceability (irreversible)	2.9	1.5

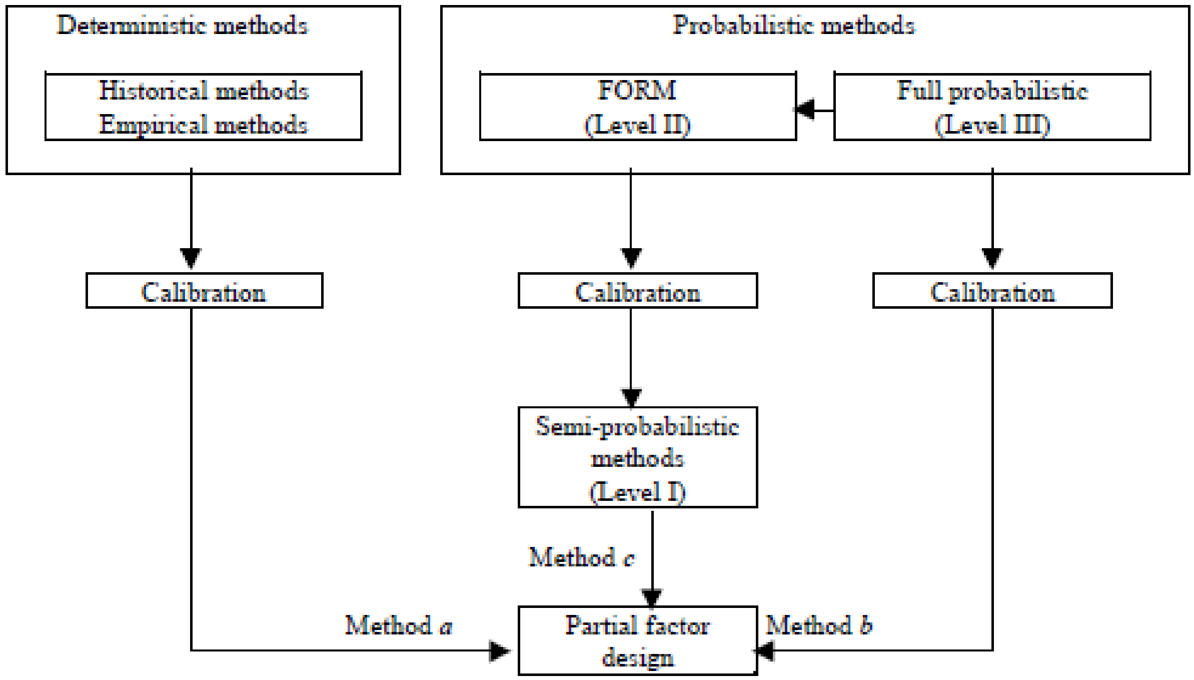


Figure 6.1 - EN-1990:2002 Reliability Calibration Models

Image from Eurocode 7

Eurocode notes calibration to existing deterministic models (Method a) is the primary source of the load and resistance factors provided therein. Method c, probabilistic calibration by FORM, is similarly noted as the approach employed for further development of the Eurocodes. Throughout the code documents it is not clear which components are derived from deterministic or probabilistic models with regard to the United Kingdom Annex. Other EU countries may provide more extensive documentation of the methods employed to calibrate specific components of their respective annexes. Full probabilistic evaluations, Method b, are not discussed as a primary approach for calibration of the Eurocodes and are largely intended as an avenue of enhanced analysis for use in specialty applications as warranted.

6.1.1 *Application of Eurocode 7 – Drilled Pier Capacity*

Eurocode 7 generally permits greater flexibility in design equation formats than codes developed in the United States. With regard to foundation resistance, designers may select one of three design equations, each employing a different partial factor format based on the method of calculation from which geotechnical resistance is derived. Equation 6.1.1 (Eurocode 7 eq. 2.7a) applies a design model in which foundation design resistance is derived from a mechanistic model that includes factors on individual soil strength parameters (e.g., shear strength, friction angle). Alternatively, Equation 6.1.2 (Eurocode 7 Eq. 2.7b) represents application of a mechanistic model in which nominal soil strength parameters yield a nominal foundation resistance value and a gross resistance factor is applied to the result. This method is applied to each component of foundation resistance (e.g., side resistance, end bearing). Equation 6.1.3 (Eurocode 7 Eq. 2.7c) is a combination of both approaches in which factors are applied to individual soil strength parameters as well as to gross foundation resistance.

$$R_d = R(\gamma_F\ F_{rep};\ X_k/\gamma_M;\ a_d) \qquad \text{Eq. 6.1.1}$$

$$R_d = R(\gamma_F\ F_{rep};\ X_k;\ a_d)\ /\ \gamma_R \qquad \text{Eq. 6.1.2}$$

$$R_d = R(\gamma_F\ F_{rep};\ X_k/\gamma_M;\ a_d)\ /\ \gamma_R \qquad \text{Eq. 6.1.3}$$

Where:

R_d = Design foundation resistance

γ_F = Partial factor on actions (loads)

F_{rep} = Representative value of an action (load)

X_k = Characteristic value of material property

γ_M = Partial factor on a material property

ad = Nominal foundation dimension

γ_R = Partial factor on gross foundation resistance

Note 1: The code specifies γ_F may be taken as 1.0 where the design procedure includes factored loads.

The code generally notes the characteristic (nominal) value of design parameters as a cautious estimate of the value affecting the limit state of interest. Where statistical analyses are performed, the characteristic value is similarly defined as a property having a prescribed low probability of occurrence. For the purpose of estimating material properties where statistics are employed, the characteristic value is calculated as the mean value computed at the low bound of a 95% confidence interval.

Under the requirements of the UK annex of Eurocode 7, "Design Approach 1" is prescribed as the method through which load and resistance factors are employed to derived foundation size and/or capacity. "Design Approach 1" incorporates two partial factor combinations:

Combination 1: A1 "+" M1 "+" R1

Combination 2: A2 "+" M2 "+" R1

Each combination applies a different set of factors to load and resistance components with Combination 1 applying larger factors to load than resistance and vice versa under Combination 2. The governing combination dictates the design outcome of the foundation. The partial factors applicable to the design of drilled pier foundations are provided in Tables 6.3 to 6.5.

Table 6.3 - EN-1997:2004 Partial Factors on Actions (Loads) (γF)

Action		Symbol	Set	
			A1	A2
Permanent	Unfavorable	γ_G	1.35	1.0
	Favorable		1.0	1.0
Variable	Unfavorable	γ_Q	1.5	1.3
	Favorable		0	0

Table 6.4 - EN-1997:2004 Partial Factors on Soil Parameters (γM)

Soil Parameter	Symbol	Set	
		M1	M2
Angle of Shearing Resistance [1]	$\gamma_{\varphi'}$	1.0	1.25
Effective Cohesion	$\gamma_{c'}$	1.0	1.25
Undrained Shear Strength	γ_{cu}	1.0	1.4
Unconfined Strength	γ_{qu}	1.0	1.4
Weight Density	γ_{γ}	1.0	1.0
[1] This factor is applied to tanφ			

Table 6.5 – EN-1997:2004 Partial Factors on Drilled Pier Resistance (γR)

Resistance	Symbol	Set			
		R1	R2	R3	R4
Base	γ_O	1.1	1.1	1.0	1.45
Shaft (compression)	γ_S	1.0	1.1	1.0	1.3
Total/combined (compression)	γ_t	1.1	1.1	1.0	1.4
Shaft in Tension	$\gamma_{s;t}$	1.25	1.15	1.1	1.6

6.1.2 *Eurocode 7 – Service Limit Applications*

In accordance with sub-clause 2.4.8(2) of BS EN 1997-1:2004, the partial factors noted in Eq.'s 6.1.1 through 6.1.3 are taken as 1.0. The characteristic value for each design parameter remains at the 5% fractile estimate on the mean value. Of the three codes and guidelines examined herein, Eurocode 7 is the only document to provide a firm definition of the nominal values for use in design, and by extension is the only document to define a probabilistic framework for service limit computations.

6.2 FHWA,2010 - Drilled Shafts: Construction Procedures and LRFD Design Methods, FHWA-NHI-10-016

The FHWA LRFD drilled shaft design manual is an update to the previous edition (O'Neil & Reese, 1999) that employed traditional ASD methods. The move to LRFD is, in part, motivated by the continued development of the AASHTO LRFD bridge design specification originally published in 1992 and currently in its 6th edition (AASHTO, 2013). The FHWA manual employs the same design format as AASHTO 2007, with a number of modifications and enhancements. Key enhancements affecting foundation design practice and of particular importance for this analysis are the introduction of procedures for the design of laterally loaded piers and expansion of existing axial loading design models. Existing AASHTO design practices employ a resistance factor of 1.0 to laterally loaded piers, which is in conflict with the FHWA recommended resistance factor of 0.67.

FHWA draws on multiple sources for derivation of resistance factors applicable to different design scenarios. The majority of the resistance factors implemented in FHWA are derived from AASHTO 2007 which, by extension, are derived from:

- (Allen, 2005)
    - NCHRP Report 343 (Barker, et al., 1991)
    - NCHRP Report 507 (Paikowsky, 2004)

As with AASHTO, the FHWA 2010 is an industry specific document intended for bridge foundations where superstructures are designed in accordance with compatible AASHTO and FHWA design equations and reliability goals.

Application of FHWA design equations and resistance factors beyond the transportation industry is challenging for a number of reasons:

- FHWA resistance factors are calibrated to match load factors intended for bridge design. These load factors are not necessarily comparable with those of the NESC or other codes important for transmission line design.
- Drilled pier foundations for bridge structures often exhibit relatively large L/B ratios and lateral behavior is correspondingly calibrated according to p-y computation methods, assuming linear elastic pile behavior (O'Neil & Reese, 1999). Transmission line foundations generally have L/B ratios less than 8 and behave as rigid bodies under lateral loading. Application of p-y calculations methods which include pier flexure may contribute to inaccurate estimates on lateral movement for rigid piers lending some incompatibility to the lateral resistance factors (Kandaris, DiGioia, & Heim, Evaluation of Performance Criteria for Short Laterally Loaded Drilled Shafts, 2012).

6.2.1 *Application of FHWA 2010*

As in Eurocode 7, the methods employed for calibration of resistance factors in FHWA vary with the extent of data applicable to each design scenario and incorporate all of the methods noted previously. Where reliability methods are used to determine resistance factors, a target reliability index of $\beta = 3.0$ is utilized.

The FHWA design equation is a multiple partial factor format in which individual components of nominal foundation resistance are reduced by a corresponding resistance factor and loads increased by a load factor (Eq. 6.2.1 and Tables 6.2.1 and 6.2.26).

$$\sum \eta_i \gamma_i Q_i \leq \sum \square_i R_i \qquad \text{Eq. 6.2.1}$$

Where:

η_i	=	Load modifier for ductility, redundancy, and importance
γ_i	=	Load factor applied to force effect *i*
Q_i	=	Force effect *i*
$\square_i$	=	Resistance factor for resistance component *i*
R_i	=	Nominal value of resistance component *i*

FHWA does not specify the definition of the term 'nominal', thus an interpretation is made based on the primary source document, AASHTO 2007. Under AASHTO 2007, nominal is defined within the construct of allowable stress design as the ultimate capacity of an element or, where deflection-limited criteria are considered and load tests are performed, the capacity accompanied by the maximum allowable deflections. This definition of nominal capacity excludes consideration of the deformation required to mobilize the various components of foundation resistance (AASHTO, 2007).

Appropriate selection of strength parameters for use with the resistance factors of Table 6.6 is not provided by FHWA and AASHTO 2007 is referenced. In accordance with AASHTO, 2007, flexibility is granted to the design engineer to introduce engineering judgment based on an assessment of the project database relative to past experience. In general, the resistance factors presented are calibrated based on average soil properties where variability adheres to accepted values (Table 6.8) (Duncan, 2000) (Sabatini, Bachus, Mayne, Schneider, & Zettler, 2002). Depending on the variability encountered, progressive levels of conservatism are recommended ranging from a conservative interpretation of the mean to low bound values (AASHTO, 2007). The decision to use high variability data, in lieu of collecting additional data to better-define the mean, is determined based upon the sensitivity of the design outcome to the parameter of interest.

Table 6.6 - FHWA Load Factors (γ)
(Brown, Turner, & Castelli, 2010)

Load Combination Limit State	*PL*	*LL*	*WA*	*WS*	*WL*	*FR*	*TCS*	*TG*	*SE*	Use one of these at a time			
										EQ	*IC*	*CT*	*CV*
Strength I	γ_p	1.75	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
Strength II	γ_p	1.35	1.00	-	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
Strength III	γ_p	-	1.00	1.40	-	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
Strength IV	γ_p	-	1.00	-	-	1.00	0.50/1.20	-	-	-	-	-	-
Strength V	γ_p	1.35	1.00	0.40	1.00	1.00	0.50/1.20	γ_{TG}	γ_{SE}	-	-	-	-
Extreme Event I	γ_p	γ_{EQ}	1.00	-	-	1.00	-	-	-	1.00	-	-	-
Extreme Event II	γ_p	0.50	1.00	-	-	1.00	-	-	-	-	1.00	1.00	1.00
Service I	1.00	1.00	1.00	0.30	1.00	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
Service II	1.00	1.30	1.00	-	-	1.00	1.00/1.20	-	-	-	-	-	-
Service III	1.00	0.80	1.00	-	-	1.00	1.00/1.20	γ_{TG}	γ_{SE}	-	-	-	-
Service IV	1.00	-	1.00	0.70	-	1.00	1.00/1.20	-	1.00	-	-	-	-
Fatigue	-	0.75	-	-	-	-	-	-	-	-	-	-	-

PL	permanent load	*WL*	wind on live load	*EQ*	earthquake
LL	live load	*FR*	friction	*IC*	ice load
WA	water load and stream pressure	*TG*	temperature gradient	*CT*	vehicular collision force
WS	wind load on structure	*SE*	settlement	*CV*	vessel collision force
		TCS	uniform temperature, creep, and shrinkage		

Table 6.7 - FHWA Resistance Factors (φ)
(Brown, Turner, & Castelli, 2010)

<table>
<tr><th>Limit State</th><th>Component of Resistance</th><th>Geomaterial</th><th>Equation, Method, or Chapter Reference</th><th>Resistance Factor, φ</th></tr>
<tr><td rowspan="3">Strength I through Strength V

Geotechnical Lateral Resistance</td><td>Pushover of individual elastic shaft; head free to rotate</td><td>All geomaterials</td><td>p-y method pushover analysis; Ch. 12</td><td>0.67[1]</td></tr>
<tr><td>Pushover of single row, retaining wall or abutment; head free to rotate</td><td>All geomaterials</td><td>p-y pushover analysis</td><td>0.67[1]</td></tr>
<tr><td>Pushover of elastic shaft within multiple-row group, with moment connection to cap</td><td>All geomaterials</td><td>p-y pushover analysis</td><td>0.80[1]</td></tr>
<tr><td rowspan="11">Strength I through Strength V

Geotechnical Axial Resistance</td><td rowspan="4">Side resistance in compression/uplift</td><td>Cohesionless soil</td><td>Beta method
(Eqs. 13-5 to 13-15)[2]</td><td>0.55 / 0.45</td></tr>
<tr><td>Cohesive soil</td><td>Alpha method
(Eq. 13-17)</td><td>0.45 / 0.35</td></tr>
<tr><td>Rock</td><td>Eq. 13-20[2]</td><td>0.55 / 0.45[3]</td></tr>
<tr><td>Cohesive IGM</td><td>Modified alpha method
(Eq. 13-28)</td><td>0.60 / 0.50[1]</td></tr>
<tr><td rowspan="3">Base resistance</td><td>Cohesionless soil</td><td>N-value (Eq. 13-16)</td><td>0.50</td></tr>
<tr><td>Cohesive soil</td><td>Bearing capacity eq.
(Eq. 13-18)</td><td>0.40</td></tr>
<tr><td>Rock and Cohesive IGM</td><td>1. Eq. 13-22
2. CGS, 1985 (Eq. 13-23)</td><td>0.50</td></tr>
<tr><td>Static compressive resistance from load tests</td><td colspan="2">All geomaterials</td><td>≤ 0.7[4]</td></tr>
<tr><td>Static uplift resistance from load tests</td><td colspan="2">All geomaterials</td><td>0.60</td></tr>
<tr><td>Group block failure</td><td colspan="2">Cohesive soil</td><td>0.55</td></tr>
<tr><td>Group uplift resistance</td><td colspan="2">Cohesive and cohesionless soil</td><td>0.45</td></tr>
<tr><td rowspan="3">Strength I through Strength V;
Structural Resistance of R/C</td><td colspan="3">Axial compression</td><td>0.75</td></tr>
<tr><td colspan="3">Combined axial and flexure</td><td>0.75 to 0.90</td></tr>
<tr><td colspan="3">Shear</td><td>0.90</td></tr>
<tr><td>Service I</td><td colspan="2">All cases, all geomaterials</td><td>Ch. 13, Appendix B</td><td>1.00</td></tr>
<tr><td rowspan="3">Extreme Event I and Extreme Event II</td><td>Axial geotechnical uplift resistance</td><td>All geomaterials</td><td>Methods cited above for Strength Limit States</td><td>0.80</td></tr>
<tr><td>Geotechnical lateral resistance</td><td>All geomaterials</td><td>p-y method pushover analysis; Ch. 12</td><td>0.80[1]</td></tr>
<tr><td>All other cases</td><td>All geomaterials</td><td>Methods cited above for Strength Limit States</td><td>1.00</td></tr>
</table>

[1] Currently not addressed in AASHTO (2007)
[2] Design equation differs from AASHTO (2007)
[3] Resistance factor different from AASHTO
[4] See AASHTO Table 10.5.5.2.3-1

Table 6.8 - COV Values for Common Soil Properties
(Duncan, 2000)

Measured or interpreted parameter value	**Coefficient of Variation, V (%)**
Unit weight, γ	3 to 7 %
Buoyant unit weight, γ_b	0 to 10 %
Effective stress friction angle, ϕ'	2 to 13 %
Undrained shear strength, s_u	13 to 40 %
Undrained strength ratio (s_u/σ_v')	5 to 15 %
Compression index, C_c	10 to 37 %
Preconsolidation stress, σ_p'	10 to 35 %
Hydraulic conductivity of saturated clay, k	68 to 90 %
Hydraulic conductivity of partly-saturated clay, k	130 to 240 %
Coefficient of consolidation, c_v	33 to 68 %
Standard penetration blowcount, N	15 to 45 %
Electric cone penetration test, q_c	5 to 15 %
Mechanical cone penetration test, q_c	15 to 37 %
Vane shear test undrained strength, s_{uVST}	10 to 20 %

The lateral load resistance factor of 0.67 is noted as a preliminary value in contrast to the AASHTO value of 1.0. Brown et al (2010)indicate that 0.67 is a value derived from engineering judgment, and in the absence of a reliability-based calibration study.

6.2.2 *FHWA, 2010 - Service Limit Applications*

FHWA service limit analyses are carried out with soil properties established under the same procedure used for limit state design. In accordance with Tables 6.2.2 and 6.2.3, both load and resistance factors are generally reduced/increased to a value of 1.0. The structure wind load factor is reduced to a value of 0.30 as a means to reduce the service load to a value lower than the nominal 55mph wind event employed for limit state design. For service limit computations under extreme loading events, the resistance factor is reduced to 0.80.

The resistance factor of 1.0 for service limit design is noted as a preliminary value under continuous assessment. In general, laterally loaded transmission line structures (tubular steel poles) can withstand deflections in excess of those acceptable for bridge superstructures. Further adjustment of this value may limit the applicability of FHWA for the purposes of transmission line foundation design.

6.3 EPRI, 2012 - Transmission Structure Foundation Design Guide

The EPRI Transmission Structure Foundation Guide is a transmission line industry specific state-of-practice document, developed with the intention to provide a manageable and consistent framework for LRFD transmission line foundation design. EPRI 2012 follows previous EPRI work (Phoon, Kulhaway, & Grigoriu, 1995) which employs more complex multiple resistance factor design methods. This most recent work is an LRFD procedure calibrated using full-scale load test data in comparison to EPRI foundation design software (MFAD, HFAD and TFAD). Reliability computations contained therein are resolved to a single resistance factor applied to nominal design resistance.

Derivation of resistance factors is carried out by a semi-empirical calibration method in which full-scale load test data is compared to theoretical computations. Resistance factors are selected to adjust design model resistance values to represent measured values with a defined level of confidence. For compatibility, EPRI 2012 calibrations are developed on the basis of a 5% Lower Exclusion Limit (LEL) in relation to a 50 year Return Period (RP) load event in accordance with ASCE Manual 74 (ASCE, 2010). Selection of the 5% LEL is derived from independent FORM analyses (Ghannoum, 1983a) (Ghannoum, 1983b) (Dagher, Kulendran, Peyrot, Maamouri, & Lu, 1993), which demonstrate, where component resistance values employ low exclusion limits (5% - 10%), the annual probability of failure is approximated by Eq 6.3.1 is relatively independent of the resistance or load COV:

$$P_f \approx \frac{1}{2RP} \qquad \text{Eq. 6.3.1}$$

Where:

P_f = Annual Probability of Failure

RP = Load Event Return Period

Thus, in consideration of the typical ASCE 74 50-year return period event, a foundation derived from 5% LEL resistance criteria would theoretically achieve an annual $P_f = 0.01$ corresponding to $\beta = 2.3$. The calibrated resistance factor remains invariant through execution of the prescribed design method. Designers may elect to increase foundation reliability by increasing the return period under consideration.

Each resistance factor recommended by EPRI 2012 is derived from full-scale load tests and employs statistical analysis of the *m* ratio (Eq. 6.3.2) (Bazan-Zurita, Jarenprasert, Bazan-Arias, & DiGioia, 2010).

$$m = \frac{\textit{Test Resistance}}{\textit{Nominal Resistance}} \qquad \text{Eq. 6.3.2}$$

The resistance factor, φ_5, derived for a 5% LEL is then:

For normal distributions: $\varphi_5 = m_m\,(1\text{-}0.01645V_m)$ Eq. 6.3.3

For Lognormal distributions: $\varphi_5 = m_m\,(1\text{-}0.01k_5V_m)$ Eq. 6.3.4

Where:

m_m = Mean of *m*-values for all tests (model bias)

V_m = Coefficient of variation for all *m*-values

k_5 = $1.645 - 0.00925V_m$

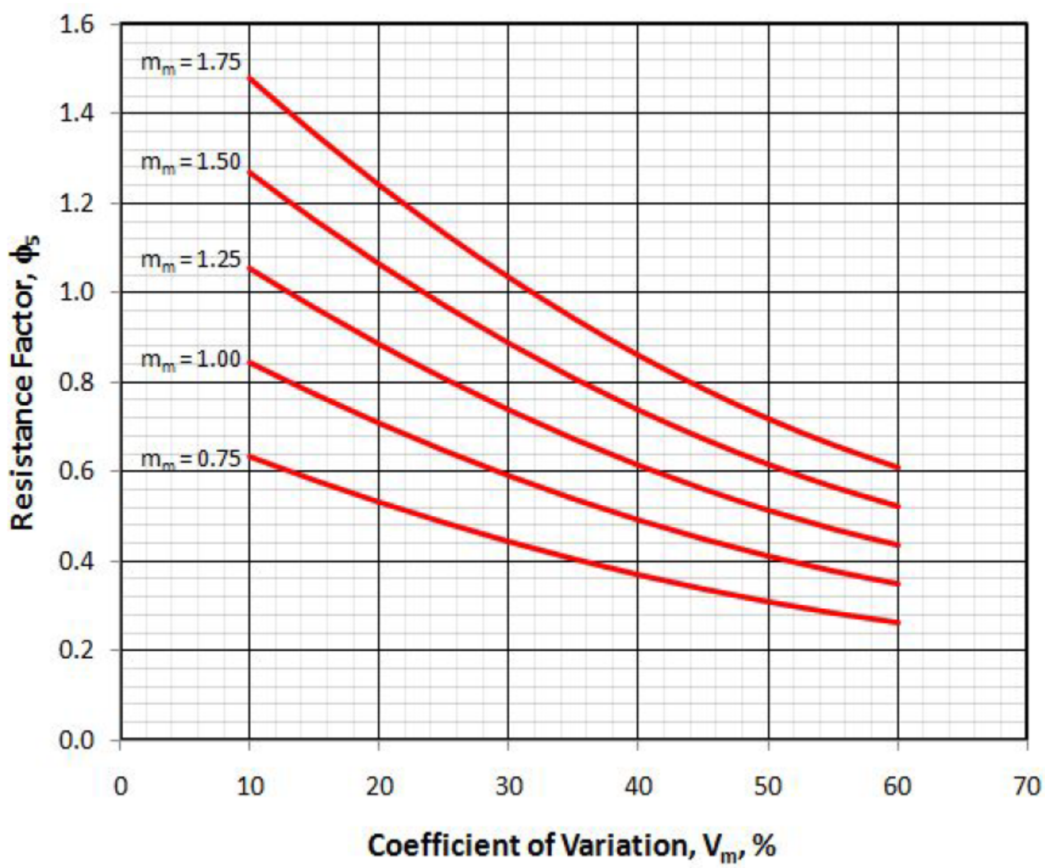


Figure 6.2 - Relationship between φ5, Vm, mm
(DiGioia Gray and Associates, 2012)

The single resistance factor format calibrated to represent load test data resolves to a linear regression analysis of capacity data through the *m*-value (Fig 6.3). The selected resistance factor, φ_5, is the correlation constant that generates a 95% level of certainty load test capacity will exceed the model capacity.

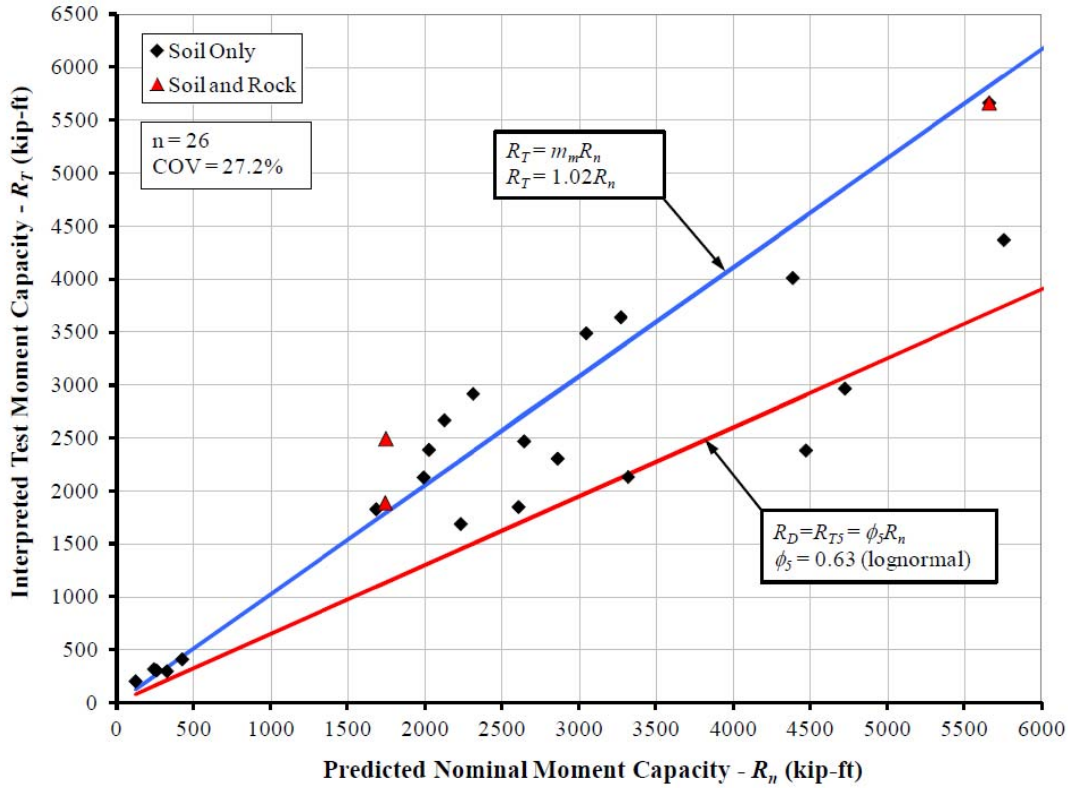


Figure 6.3 - Calibration of moment capacity according to m-value (DiGioia Gray and Associates, 2012)

6.3.1 *Application of EPRI, 2012*

There are two advantages to using the EPRI single resistance factor approach. First, the design format is simple to apply in its formulation (Eq. 6.3.5) and using mean value soil parameters makes derivation of design inputs straightforward. Second, calibration of the resistance factor to the 50-year RP event is convenient from a reliability perspective, allowing designers to manipulate load factors to achieve the desired level of reliability.

$\varphi_5 R_n \geq$ Effect of (Dead Load + γQ_{50}) Eq. 6.3.5

Where:

R_n = Nominal foundation resistance

γ = Load factor to convert to a RP other than 50 years

Q_{50} = 50 year RP load event

Where EPRI design software is used, the values for φ_5 are embedded values directly incorporated in the design capacity output. For the purposes of this research the factor of importance is lateral moment capacity $\varphi_5 = 0.63$, corresponding to a 50-year RP and an approximate $P_f = 0.01$. Where a different level of reliability is desired, designers may elect to adjust γ in accordance with Tables 6.9 and 6.10.

Table 6.9 - ASCE Manual 74 (2010) return period wind load factors

Return Period (years)	Extreme Wind Load Factor
25	0.85
50	1.00
100	1.15
200	1.30
400	1.45

Table 6.10 - ASCE Manual 74 (2010) return period ice load factors

Return Period (years)	Ice Thickness Factor	Concurrent Wind Load Factor
25	0.80	1.00
50	1.00	1.00
100	1.25	1.00
200	1.50	1.00
400	1.85	1.00

Although the lateral resistance factors recommend by EPRI 2012 and FHWA 2010 are very similar, 0.63 and 0.67 respectively, each value is derived in relation to differing design models. EPRI analyses are specific to short rigid piers with L/B < 10 where the MFAD rigid body model is applicable. The FHWA factor is recommended within the context of a p-y based model for flexible piers and is also an empirical value pending further analysis to derive a reliability-based value.

6.3.2 *EPRI, 2012 – Service Limit Applications*

Service limit design is not addressed explicitly in EPRI 2012. As an appendix to the guideline document, the results of a series of industry survey questions regarding foundation performance criteria are presented. The survey responses indicate that the application of performance criteria vary widely among EPRI members. Recommendations for performance criteria or modifications to the recommended load or resistance factors are not provided.

Subsequent analysis of EPRI load test data by others has yielded recommendations for service limit considerations to maintain movement within the elastic deformation range of the foundation response to load (Kandaris, DiGioia, & Heim, 2012). Key observations from that work are:

- Design practices which restrict foundation movement to the elastic range of the load –deflection curve generally governs foundation performance requirements over deflection limits imposed by the superstructure.
- A rotation limit of 1 degree is recommended to maintain elastic motion of the foundation.

- 1 degree of rotation typically corresponds to a lateral movement at the top of the pier equal to 3.5% - 4% of the foundation diameter.
- Specification of deflection criteria should include adjustments corresponding to foundation diameter.

7 ABEL-PINAL CENTRAL 500KV TRANSMISSION LINE

The Abel-Pinal Central (ABL-PC) Transmission Line was constructed in 2009-2010 and is a segment of a larger Extra High Voltage (EHV) transmission project known as the Southeast Valley Project. The Southeast Valley Project is a transmission loop extending from the eastern border of Mesa, AZ along the southern perimeter of the Phoenix, AZ and terminating near the Palo Verde Nuclear Generating station 15 miles west of Buckeye, AZ. The project was funded as a joint venture among several Arizona utility companies, with design and construction managed by the Salt River Project (SRP) based in Tempe, AZ.

The geotechnical investigation and subsequent foundation design conducted for ABL-PC by SRP was carried out with the intent of implementing statistical methods for establishing soil stratification and strength parameters for foundation design. Foremost in this process is the selection of boring locations based upon geologic reconnaissance work performed in advance of the geotechnical field investigation. Boring locations are allocated to mimic the proportion of structures located within each geologic region. This allows for a strategic selection of boring locations based upon the importance of a given stratum to the overall project. The greater number of borings in prevalent strata generates larger datasets capable of supporting enhanced statistical analysis in regions where design optimization will have the maximum cost impact. This methodology is extended to the selection of locations for advanced investigation methods such as pressuremeter testing or CPT where appropriate. Once field data has been collected according to geologic region it becomes possible to develop design soil zones. Within each soil zone a statistical analysis of the associated test data may be used to derive foundation design parameters for use along the project route.

The methods used to implement these methodologies on the Southeast Valley Project are discussed below. In addition, Section 8.2.1 of this document provides a detailed description of four soil strata used as a basis for production of Monte Carlo simulations in support of this research work.

7.1 Project Description:

The BDA 500/230kV segment is an extension of the existing Browning-Dinosaur line intended to connect the existing Dinosaur Receiving Station located approximately 7 miles east of Queen Creek, AZ with the future Abel Receiving Station located approximately 10.5 miles to the south (Fig 4). The ABL-PC segment begins at Abel Receiving Station located approximately 12 miles southeast of Queen Creek, AZ and extends an additional 29.25 miles southwest to the Pinal Central Receiving Station located 9 miles east of Casa Grande, AZ (Fig. 7.1). In total, the construction of both segments required the installation of 155 double and single circuit structures founded on drilled shaft reinforced concrete piers. Pier diameters ranged from 6 to 11 feet with depths ranging from 16 to 33 feet.

Figure 7.1 - Route Map: Pinal Central-Abel 500/230kV

The ABL-PC segment is located within the Basin and Range Physiographic Province characterized by broad, elongated alluvial plains drained by the Gila and Salt Rivers. The northern 3 miles of ABL-PC resides in the Salt River Valley Sub-Basin with the remainder of the line route located within the northern portion of the Eloy Sub-Basin drained by the Gila River (Geologic Consultants, 2006).

The Salt River Valley Sub-Basin consists of late Tertiary to recent age stream channel deposits characterized by non-cemented poorly graded sands and gravels, Gila River flood plain deposits characterized by low density clayey silts and silty clays underlain by gravels and sands, and basin fill deposits consisting of interbedded layers of fine grained sands with low to high plasticity clays and silts. The basin fill deposits encountered are predominantly used as agricultural land subject to frequent grading and localized saturated conditions in upper strata due to irrigation (SRP, 2009).

7.2 Field Reconnaissance:

In advance of boring site selection, a geologic survey of the line route was conducted by Geologic Consultants, Inc. of Phoenix, AZ. The objective of the geologic reconnaissance and research was the development of geologic strip maps for the entire line route, which enable an objective evaluation of potential subsurface boring locations.

When selecting geotechnical boring locations, it is assumed that areas with similar geology will exhibit similar soil properties. Proposed structure locations are overlain on the geologic strip maps developed during the geologic reconnaissance and a simple tally of structures located in each geologic zone is used to apportion borings along the line route (Fig. 7.2). Geologic zones with a given percentage of the total structures receive an equivalent percentage of the allotment of total borings.

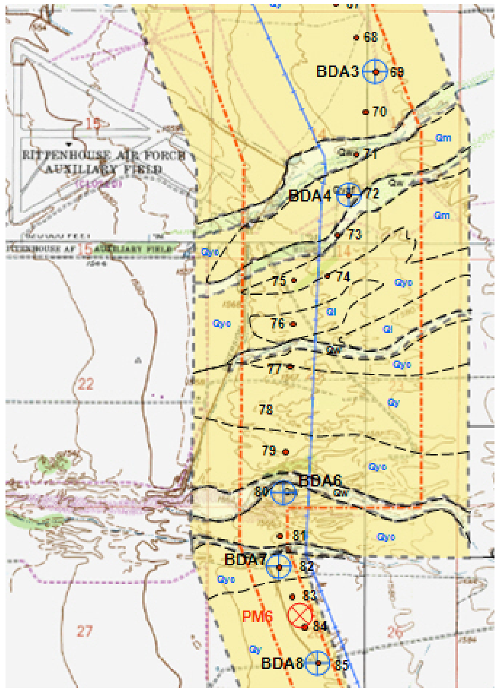


Figure 7.2 - Sample of line route overlain on geologic data.
Blue targets represent boring locations and red targets represent pressuremeter test locations.

Determination of the total number of subsurface borings to utilize for the geotechnical investigation is based on several factors. Foremost is the production of sufficiently representative data within geologic zones spanning a large percentage of the line route. The number of borings required to meet this goal is weighed against schedule and budget constraints as well as logistical challenges such as site access restrictions due to land ownership, harsh terrain or presence of archeological sites.

Twenty-three borings were planned for the ABL-PC project, twenty-two of which were completed, with one excluded due to a lack of archeological clearances. Ten, fifteen and eight borings at the nearby Dinosaur, Abel and Pinal Central Receiving Stations, respectively, supplemented the transmission line investigation.

Typical borings were performed with a nominal 7” diameter single flight hollow stem auger advanced to a maximum depth of 31.5 feet below grade. Where soil conditions permitted, drive ring samples were taken at 2.5-foot intervals to 10 feet in depth and 5-foot intervals thereafter to the full depth of each boring. In granular soil conditions, standard penetration resistance tests (SPT) were conducted at the same depth intervals noted for drive ring samples.

Two borings along the ABL-PC route were located in the main flow channel of the Gila River. At these locations, exploratory borings were performed with a nominal 12-inch diameter percussion hammer system advanced to depths of 85 and 95 feet below existing grade. SPT samples were taken in these borings as well following the same sampling scheme as previously noted for standard auger borings.

7.3 Optimization Reconnaissance:

For the most abundant soil strata encountered along the line route, an additional investment was made in more advanced testing techniques beyond standard exploratory borings.

Pressuremeter testing was performed to better characterize the soil modulus of softer soils along the line route. Most notably, the largest basin fill deposit along the line route extends from Pinal Central Receiving Station to fourteen miles northeast along the line route. This deposit is characterized by weak interbedded clay-silt-sand layers to a depth of approximately twelve feet, with increased strength to depth. Given the large extent of this formation, there was a strong case for the use of pressuremeter data to obtain a direct measurement of soil lateral deformation modulus, an important parameter for laterally loaded foundation design. The pressuremeter data retrieved paired with blow count data in the same strata provides a site-specific correlation for corrected blow count to lateral modulus.

An alluvial fan deposit extending 4.5 miles north of the Gila River T is characterized by earthen fills over much harder rock-like material at depth. To better identify the depth of interface between earthen fills and rock-like material, a seismic refraction survey was conducted at 7 locations within the formation. Seismic refraction surveys were conducted using lines of 12 geophones spaced at 10-foot intervals providing 110 feet of subsurface coverage to an approximate depth of 30 feet.

7.4 Design Soil Zones:

For the purposes of efficient foundation design, soil design zones are established along the line route. Each design zone is defined as a theoretical soil profile developed as a conservative representation of data retrieved from a specific region of the project. The assignment of strength parameters within each soil layer is of great importance and discussed in greater detail in Section 7.4.1. The soil layer dimensions and strength parameters derived for each design zone are used to develop foundation designs for each structure residing within the zone.

The defining aspect of the statistically-based methods used on the ABL-PC project and this study to develop design soil stratification and strength parameters on the Southeast Valley Project relates to the identification of soil design zones and layers from analysis of variations in blow count with depth. The used process is iterative and utilizes a mixture of elements derived from the initial geologic reconnaissance, geotechnical field investigation, subsequent laboratory testing, and ultimately engineering judgment, to arrive at a comprehensive evaluation of soil stratification over the length of the project.

7.4.1 *Soil Stratification*

Just as geotechnical boring locations are determined to conform to common geologic regions, all data recovered from borings within a common geologic setting is, initially, assumed to reside in a common design zone unless significant differences in material type are observed in the field data. This initial grouping represents the first attempt at evaluation of commonality within the dataset in the horizontal direction. Validation of the horizontal grouping according to geologic region is achieved by evaluating the corrected SPT blow count with depth. In the absence of lab data, the SPT blow count is one of the most widely used values in geotechnical engineering for correlations to useful design parameters such as horizontal modulus, internal friction angle and so forth. Thus, its selection as a basic tool for assessment of soil strength characterization paired with material type is well founded and practical due to the abundance of SPT data recovered over the course of the field investigation.

Visual assessment of corrected SPT blow count data plotted with depth, in tandem with calculation of the associated COV within each soil layer, provides an effective means of quickly developing design soil zones. This is easily achieved through the use of commonly available spreadsheet programs and statistical analysis tools contained therein. The assessment of soil data groupings by evaluation of the COV is common practice and recommended by the AASHTO LRFD Bridge Design Specification (AASHTO, 2007). Figs. 7.3 through 7.5 provide an example of stratification assessment process applied to the PC-ABL transmission line. Potential layers are identified visually by identifying abrupt changes in the corrected SPT blow count with depth. With each subsequent layer introduction, an assessment of the achieved COV provides an indication of the quality of data and the appropriateness of the subdivision. The goal is generally to achieve a COV of 35% or less as recommended by AASHTO.

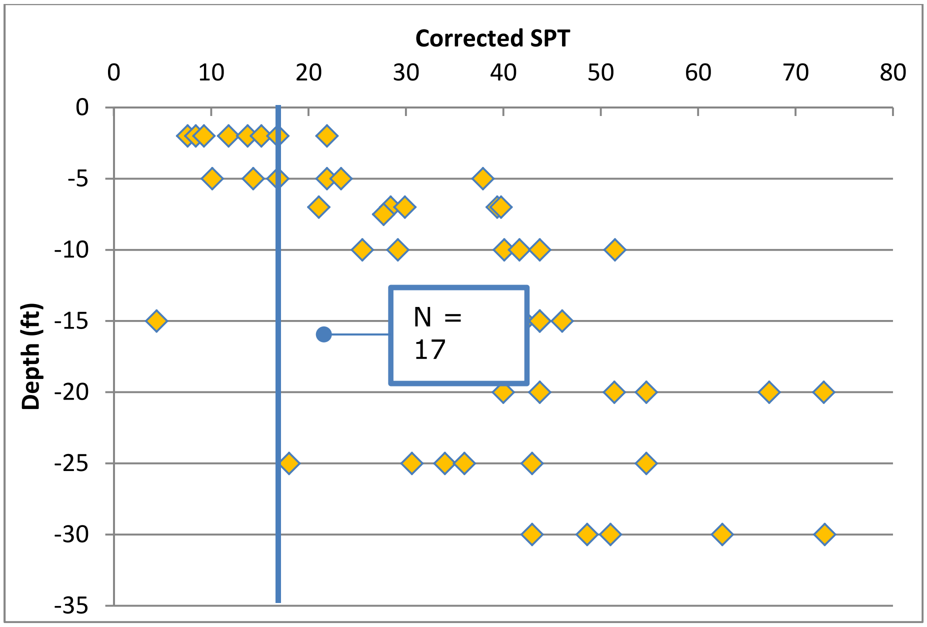


Figure 7.3 - Single layer system for Zone 2A data

0-30ft COV = 50%

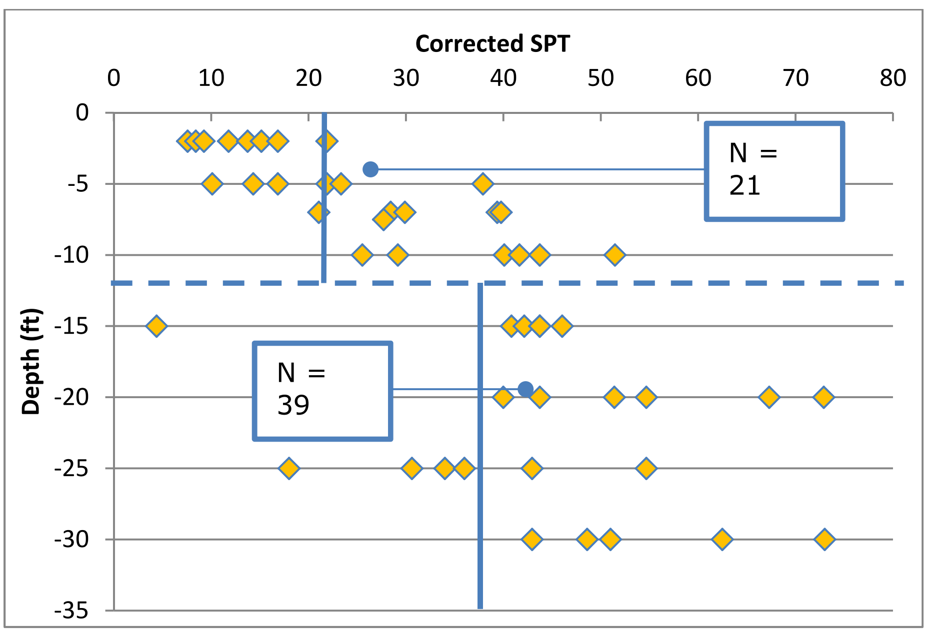


Figure 7.4 - Two layer system for Zone 2A data,

0-12ft COV = 50%, 12-30ft COV = 35%

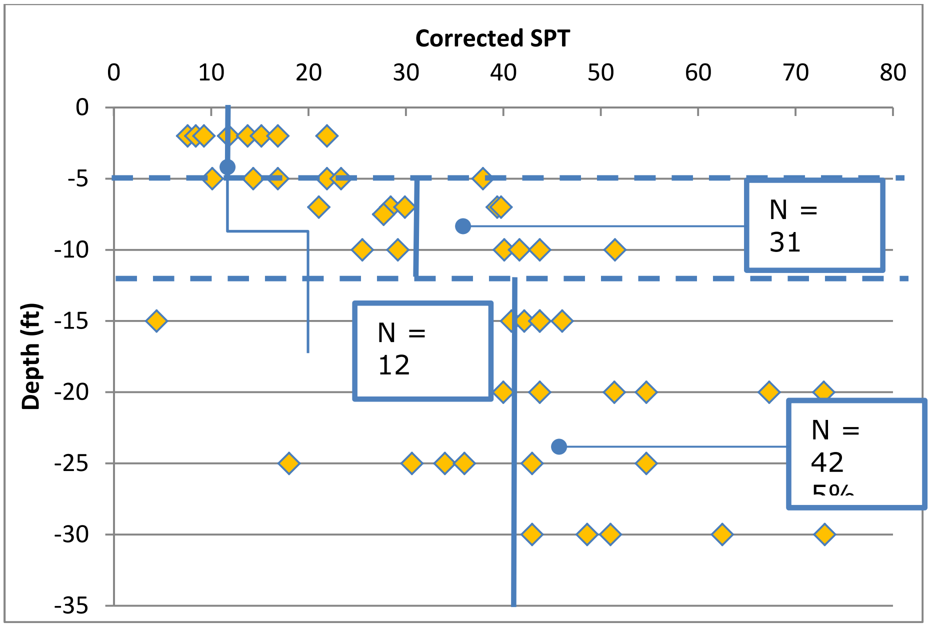


Figure 7.5 - Three layer system for Zone 2A data

1-5ft COV = 36%, 5-12ft COV = 25%, 12-30ft COV = 28%

N values reported for each layer in Figs. 7.4 through 7.5 represent the 5% Lower Exclusion Limit (LEL) on the mean value, representing the theoretical design value in accordance with the methodology applied to PC-ABL. The use of the 5% LEL on the mean value for service limit design on the PC-ABL project is derived from the recommendations of Eurocode 7 Geotechnical Design guide EN1997-1:2004, which is discussed in Section 6 of this document (British Standards, 2004). The 5% LEL on the mean value is calculated by Eq. 7.4.1.

Low Bound 90% CI = $\bar{x} - t_{\alpha/2,n-1}s/\sqrt{n}$ Eq. 7.4.1

$\bar{x}$ = Mean SPT blow count value

$t_{\alpha/2,n-1}$ = Two tailed t-distribution

α = (100 - % CI)/100

n = Number of data points

When a dataset is small, design values are assigned as the conservative (minimum) value between that calculated for the prescribed confidence interval or the mean value minus one standard deviation.

7.5 Lab Testing

Laboratory testing of samples recovered from the field investigation were performed at SRP's materials testing facility in Tempe, AZ. Laboratory testing included 59 unsaturated direct shears, 25 Atterburg limit tests as well as 125 in-situ moisture content and in situ density measurements. Gradation analyses were performed on samples from the Gila River flow channel for use in scour analyses.

Unsaturated direct shear tests were performed on drive ring samples recovered from the field investigation at surcharge loads of 1, 2, and 3ksf. Internal friction angle data was evaluated based upon a linear regression analysis of values acquired from direct shear testing. In the absence of direct shear data, friction angle values were obtained based on published correlations to blow count (Hantanaka & Uchida, 1996; Schmertmann, 1975; Shioi & Fukui, 1982). The correlated dataset was evaluated at the same 5% LEL as the SPT data by Eq. 7.4.1. Similarly, density and moisture content lab data were evaluated using a 90% confidence interval for the purposes of developing foundation design parameters.

All laboratory data is grouped according to the soil design zones established by analysis of the SPT data. The abridged field and laboratory data pertinent to this research is provided in Appendix A.

7.6 Foundation Design

Foundation designs were carried out using EPRI's laterally loaded drilled shaft foundation design program, MFAD (GAI Consultants, Inc., 1982). The MFAD model is based on a four-spring model used to simulate the modes of resistance imparted by the surrounding soils within the constraints of a foundation behaving as a rigid body, as shown in Fig 7.6.

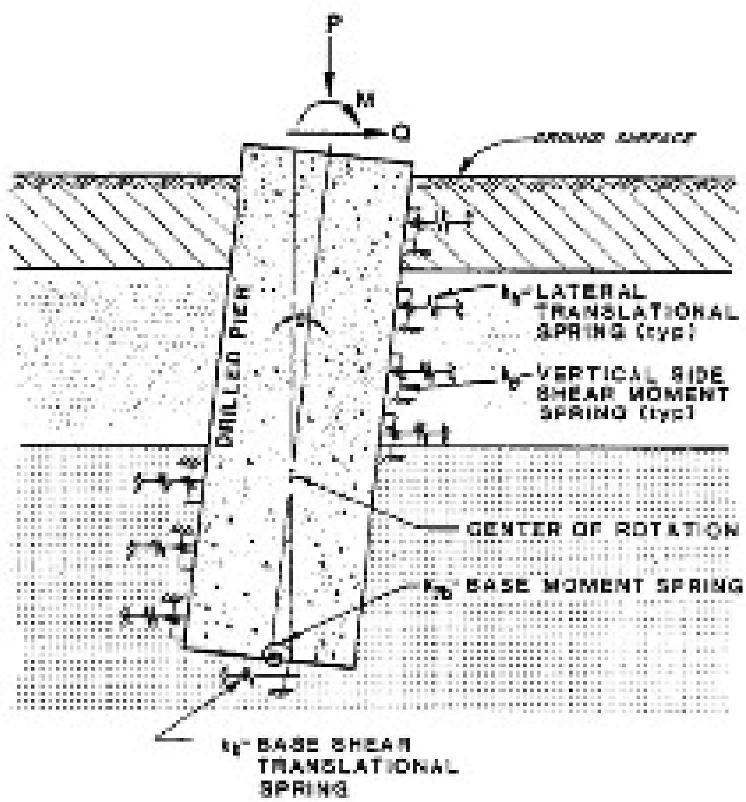


Figure 7.6 - MFAD spring model

(DiGioia Gray and Associates, 2012)

Pier analyses were carried out based upon a maximum deflection equal to 4% of the pier diameter, 50% of which may be non-recoverable. The maximum permissible rotation at the top of the pier was limited to 1° of which 0.5° may be non-recoverable.

Typically the structure baseplate dimensions govern the diameter of transmission pole foundations; therefore the only variable to affect foundation capacity is embedment depth. Foundation designs were carried out in a two-step process; an initial MFAD run to determine embedment depth and a second run to determine internal foundation forces for the purposes of concrete design. During the initial MFAD run, depth is determined based upon ultimate loads supplied by the structure designers with an importance factor of 1.25 applied to dead-end structures, 1.15 for angle structures and 1.05 for tangent structures. The subsequent MFAD runs utilize the ultimate structure loads reduced by a factor of 1.65, as discussed in Section 3 of this document, to approximate the unfactored structure loads and develop internal shear and moment forces for concrete reinforcing design according to ACI load factors.

8 THEORETICAL FOUNDATION PERFORMANCE MODEL

The RBD foundation design documents discussed herein (Section 6) represent a cross section of the existing documents that provide explicit recommendations for the design of laterally loaded drilled pier foundations. They are variable in their target industry, the formulation of the governing design equations, the rigor with which load and resistance factors are derived and the desired level of reliability.

Within the field of geotechnical engineering and efforts to derive RBD methodologies therein, much emphasis has been placed on the quantification of uncertainty contributed by inherent soil variability, load variability, measurement techniques, correlations, design models and the like. However, only limited focus has been placed on the effect that stratification has on reliability outcomes. This, at least to some degree, may be attributable to the difficult in quantifying spatial variability in the calibration formats commonly used in RBD calibrations (MVOSM and FORM), which cannot accommodate the high dimensional analysis to do so (Cao et al., 2013). This stands to reason--stratification plays an important role in the achieved reliability of the design product. However, from a code development perspective, assessment of uncertainty in stratification is a challenging notion. Stratification is solely a site-specific consideration and is not readily quantifiable in a generic framework.

To the extent stratification is understood on a site-specific basis, the variability of strata dimensions (depth and thickness) is less subject to statistical variation as it is to variations in deposition, erosion and other mechanisms that are not easily represented in a numerical model. A number of researchers have identified methods to employ soil variability to evaluate strata dimensions (Phoon, Kulhaway, & Grigoriu, 1995) (Heim, Kandaris, & Houston, 2011). In general, these methods iterate through potential soil layers while monitoring changes in the COV of a chosen geotechnical metric (commonly SPT blow count). The layer configuration that yields the lowest COV across each layer is considered the design stratification. These methods are useful in the derivation of statistically viable soil properties with depth and are compatible with RBD methodologies.

However, for the instance of laterally loaded drilled piers embedded in multiple soil layers, the magnitude of support derived from each layer depends upon its strength and depth relative to the other layers. Where there is disparity in strength amongst layers, stronger layers will generally attract more load than weaker strata. Therefore, from a reliability perspective, any uncertainties in the strength or dimensions of the strongest layer is of greater importance to the design problem than that of the weaker layers.

The role stratification plays in reliability outcomes is of interest, particularly as the field of geotechnical engineering moves toward RBD methodologies with the goal of consistent reliability across variable design environments. To ascertain the role that stratification plays in the performance of RBD derived foundation designs, a Monte Carlo simulation model is employed to illuminate the reliability performance of a series of foundation designs. The Monte Carlo approach is employed as a robust computation method capable of incorporating spatial variability.

The model examines the performance of foundations derived from the Eurocode, FHWA and EPRI design guides. Two alternative approaches are considered using site-specific soil strength characteristics evaluated at the 5% LEL of the mean value and at the 10% LEL of sample. Neither approach employs strength factors. Instead, the design process relies on interpretation of the soil strength dataset to derive appropriate foundation designs for both ultimate capacity and service limit states. The approach employing the 5% LEL on the mean value is representative of the design approach utilized on the ABL-PC project, whereas the 10% LEL method is representative of a low bound evaluation of strength parameters, as is commonly used in practice.

Foundation designs are accumulated based on nominal soil strength parameters computed in accordance with each design approach (Section 8.2.3) and sourced from the ABL-PC 500kV Transmission Line Project database. The methods used to develop model inputs, the simulation model and the modeling outcomes are discussed herein.

8.1 Foundation Performance Simulation Procedure

8.1.1 *Modeling Procedure*

The intent of the theoretical foundation performance model is to evaluate the probability that a series of foundation designs (Table 8.2.10) will satisfy performance requirements when soil variability is considered. The model is constructed on the basis of a Monte Carlo simulation. Within the simulation, foundation diameter (Table 8.2.8) as well as soil layer dimensions (Table 8.2.1) are held constant while soil strength parameters within each soil layer are permitted to vary according to their identified probability density functions (Table 8.2.2) . Foundation loads are permitted to vary as well.

Strength parameters for each soil layer are calculated automatically from correlations to a randomized N_{60} SPT blow count value (Section 8.1.2). The randomized blow count value is computed on the basis of the probability density function assigned to each soil layer (Table 8.2.2). By virtue of the modeling procedure, soil strength parameters within each layer are considered linked parameters and vary in unison according to the randomly selected probability value. Thus, an N_{60} value corresponding to a 50% probability of exceedance will yield computations of strength values representing the same level of probability.

For each randomized set of strength parameters, the theoretical foundation design (Table 8.2.10) under consideration is analyzed to evaluate ultimate capacity and deflection performance. This analysis is repeated over a large number of permutations, each with a recalculated set of randomized strength parameters. The results of each analysis are assembled in a foundation performance database that is examined to evaluate the probabilistic performance of each theoretical foundation. Curve fitting methods are used to assign a probability density function (PDF) to each set of foundation performance metrics, which enables computation of the probability that each theoretical foundation will satisfy the performance requirements, having accounted for soil variability.

8.1.2 *Soil Strength Correlations*

To facilitate rapid computation of soil strength parameters corresponding to a randomized variable, a series of correlations to the corrected SPT blow count are used. Correlation equations are employed on a material specific basis and are either derived from published sources or, when available, from site-specific correlations developed for the ABL-PC project.

The use of correlations and linked strength parameters in lieu of PDF's assigned to individual strength parameters is a simplification used to enable Monte Carlo simulation using a limited dataset. Data on specific strength parameters on a layer basis is either absent or insufficient to construct meaningful PDF's. However, SPT blow count data is available in relatively large quantities for each soil layer of interest, and therefore is the best measure of soil variability available within the dataset.

8.1.2.1 *Friction Angle, ϕ*

Each of the four soil strata considered in the simulation model are characterized as sandy clay materials. The average of four published correlations for clean sands (Schmertmann, 1975) (Eq. 8.1.2), for natural sands (Shioi & Fukui, 1982) (Eq. 8.1.3 and 8.1.4) and (Hantanaka & Uchida, 1996) (Eq. 8.1.5) to friction angle is used in the Monte Carlo simulation program.

Field measured blow counts are converted to the standard penetration number N60 in accordance with (Seed, Tokimatsu, Harder, & Chung, 1985) (Skempton, 1986) (Eq. 8.1.1)

$$N_{60} = (80 \times N_c)/60 \qquad \text{Eq. 8.1.1}$$

Where:

For cohesive materials (Ring sampler):

$$N_c = 0.89N \quad (N \leq 6.6)$$

$$N_c = 0.842N \quad (6.6 \leq N \leq 27)$$

$$N_c = 0.729N \quad (27 \leq N)$$

For granular materials (Ring sampler):

$$N_c = 0.55N$$

For cohesive and granular materials (Split spoon sampler):

$N_c = N$

$N =$ Field measured blow count with autohammer

$D =$ Average layer depth (ft)

$\gamma_t =$ Total unit weight (pcf)

After (Schmertmann, 1975) the SPT correlation for peak friction angle φ_p sands is computed in accordance with Eq. 8.1.2:

$$\Phi_p = \left[\tan^{-1}\left(\frac{N_{60}}{(12.2+20.3(D\gamma_t))/2117}\right)^{0.34}\right]\left[\frac{180}{\pi}\right] \qquad \text{Eq. 8.1.2}$$

Where:

Φ_p = Peak angle of internal friction

After (Shioi & Fukui, 1982) for natural sandy soils friction angle is computed in accordance with Japanese national standards for:

Roadway bridges:

$$\Phi_p = 27 + 0.36 \times N_{70} \qquad \text{Eq. 8.1.3}$$

Design standard for structures:

$$\Phi_p = 15 + (18 \times (N_1)_{70})^{0.5} \qquad \text{Eq. 8.1.4}$$

Where:

$N_{70} = 1.36 \text{ x } Nc$

$(N_1)_{70} = N_{70} \text{ x } (\frac{\mathbf{2000}\ \boldsymbol{x}\ \boldsymbol{D}}{\boldsymbol{\gamma}\mathbf{t}})^{0.5}$

After (Hantanaka & Uchida, 1996) for natural sand deposits

$$\Phi_p = 20 + (15.4 \text{ x } (N_1)_{60})^{0.5} \qquad \text{Eq. 8.1.5}$$

Where:

$(N_1)_{60} = N_{60} \text{ x } (\frac{\mathbf{2000}\ \boldsymbol{x}\ \boldsymbol{D}}{\boldsymbol{\gamma}\mathbf{t}})^{0.5}$

In all cases the peak angle of internal friction is reduced to the residual value:

$$\Phi = \left(\frac{\mathbf{180}}{\boldsymbol{\pi}}\right) \text{ x } \mathbf{tan^{-1}}\left(\mathbf{0.67\ x\ \ tan(\Phi p\ x}\ \left(\frac{\boldsymbol{\pi}}{\mathbf{180}}\right)\right) \qquad \text{Eq. 8.1.6}$$

8.1.2.2 *Cohesion*, *c*

Cohesion values are computed from correlations to direct shear data taken from the ABL-PC dataset (Fig. 8.1). Correlation equations are computed on the basis of direct shear data recovered for each soil zone of interest (Section 8.2). Within each soil zone, discrete direct shear results are grouped according to similar blow count values, generally those within five blows/ft of one another. Cohesion is computed on the basis of the direct shear results for each grouping and is assigned a corresponding N value equal to the average within each grouping. Cohesion correlation equations are derived by curve fitting computed on the basis of the discrete (c,N) values for each group. Unlike friction angle, published correlations to blow count are generally recognized as incompatible with desert southwest soils where cementation and granular particles exist. Assignment of soil boring data to design soil zones is performed in accordance with the procedure noted in Section 7.4 of this document.

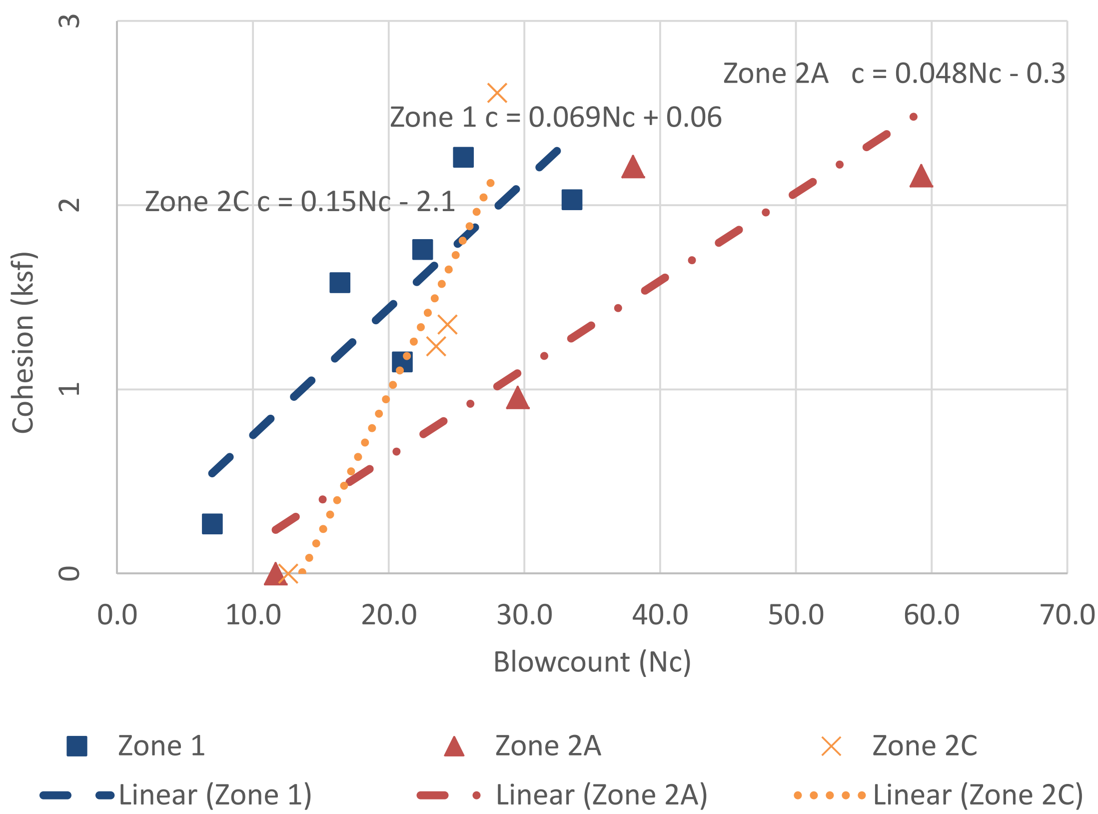


Figure 8.1 - Cohesion correlation curves from ABL-PC direct shear data

Zone 1:

$$c = 0.069N_c + 0.06 \qquad \text{Eq. 8.1.7}$$

Zone 2A:

$$c = 0.48N_c + 0.3 \qquad \text{Eq. 8.1.8}$$

Zone 2C:

Layer 1: $c = 0.15N_c - 2.1$ Eq. 8.1.9

Layer 2: $c = 0.5$

Zone 3C:

Layers 1 & 2: $c = 0.5$ Eq. 8.1.9

Layer 3: $c = 0$

8.1.2.3 *Lateral Deformation Modulus, E_p*

Lateral deformation modulus values for Zone 1 are derived from pressuremeter data acquired on the ABL-PC project. The remaining lateral modulus values are computed from unpublished pressuremeter correlation curves developed by the Salt River Project for the Phoenix metropolitan area. Correlation curves are selected for cemented-cohesive, cohesive and cohessionless soils as noted.

Zone 1:

Layer 1: $E_p = 0.19N_c - 0.75$ Eq. 8.1.10

Layer 2: $E_p = 0.11N_c^{1.08}$ cohesionless Eq. 8.1.11

Zone 2A:

Layer 1: $E_p = 0.011N_c^{1.7}$ cohesive Eq. 8.1.12

Layers 2 & 3: $E_p = 0.68N_c^{0.66}$ cohesive-cemented Eq. 8.1.13

Zone 2C:

Layers 1 & 2:

$E_p = (0.11N_c^{1.08} + 0.011N_c^{1.7}) / 2$ low plasticity gravel Eq. 8.1.14

Zone 3C:

Layers 1 & 2:

$E_p = (0.11N_c^{1.08} + 0.011N_c^{1.7}) / 2$ low plasticity gravel Eq. 8.1.15

Layer 3: $E_p = 0.11N_c^{1.08}$ cohesionless Eq. 8.1.16

8.1.3 *Load Variability*

To derive a theoretical reliability index, β, it becomes necessary to account for variability in loads applied to the foundation under consideration. Within the theoretical model, nominal foundation loads are permitted to vary under a series of simplifying assumptions relevant to the method for deriving nominal foundation loads and the probability density function governing the variability of the load.

8.1.3.1 *Design Load Margin of Safety*

The NESC designates load factors for various components of nominal live loads applied to transmission structures. The primary load components governing structure and foundation design are conductor tension and transverse wind loads with overload factors of 1.65 and 2.5 respectively. To evaluate foundation reliability incorporating load variability it becomes necessary to compute nominal foundation loads. Tangent structures are generally only subject to wind loads, thus, nominal foundation loads may be calculated by reducing design loads by a factor of 2.5. Dead-end and angle structures are subject to both conductor tension loads and wind loads, each increased by their respective load factors. The margin above nominal loads for a particular dead-end/angle structure is highly dependent upon span length, line angle, conductor configurations and other factors. For the purposes of this analysis, design loads are reduced by the conductor tension load factor of 1.65, a conservative value.

8.1.3.2 *Load Variability*

Load variability is simulated in accordance with the underlying assumptions of ASCE (2005) and the supporting wind speed database, Simiu, Changery & Filliben (1979). Both documents show wind events generally adhere to a Gumbel Extreme Value Type 1 distribution (Eq.'s 8.1.17 – 8.1.19) with a COV ranging from 15% to 30%.

$$F_v(v) = \text{Prob}(V < v)$$

$$= \exp[\ -\exp\{\ -(v-g_2)/g_1\}] \qquad \text{Eq. 8.1.17}$$

$$g_1 = 0.780\ s_v$$

$$g_2 = m_v - 0.5772 g_1$$

Where:

V = maximum annual wind speed

v = a possible value of V

s_v = sample standard deviation

m_v = sample mean

g_1 = scale parameter

g_2 = location parameter

Wind pressure is a squared product of wind speed (Eq. 8.1.18), thus foundation loads correspondingly increase exponentially with wind speed. Analysis of regional maximum annual wind speed data for Prescott, Tucson and Yuma, AZ indicates an input COV of 15% is appropriate. Application of Eq. 8.1.18 to regional wind speed variability data is used to develop Gumbel Type 1 nominal foundation load distributions (Table 8.1).

$$W_p = 0.00256 \times V^2 \times C_d \quad \text{Eq. 8.1.18}$$

Where:

Wp = Wind pressure (psf)

V = Wind velocity (mph)

C_d = Shape factor

Table 8.1 - Foundation load probability density function parameters

Foundation Type	g1	g2	COV
5DCA30-145-2	1785	6598	30%
5DCT-160	647	2391	30%

8.1.4 *Run Quantities*

Monte Carlo simulations generally employ a high number of discrete simulation runs to amass the required quantity of data to perform useful statistical analyses of the results. As with natural phenomena, low probability events (results) may occur randomly and distort the analysis of a small dataset. However, as datasets grow, low probability events become recognizable and the overall probability density function begins to emerge. With increasing number of simulation runs, measures of a specified PDF’s goodness of fit to the dataset will asymptotically approach a baseline value. Successive simulations runs beyond the baseline value consume unnecessary computing time in light of diminishing returns in model accuracy.

To ascertain the number of simulations runs required to derive a viable dataset, a series of representative analyses is conducted to evaluate a three (3) soil layer system. The simulation is evaluated over an increasing quantity of discrete simulation results and the goodness of fit to a lognormal PDF is observed. The Kolmogorov-Smirnov (KS) coefficient and probability of exceeding rotation criteria are plotted versus the quantity of runs to determine the value of interest (Fig 8.2 for the KS coefficient and 8.3 for the probability of exceeding rotation criteria). It can be seen from each figure, the calculation value approaches a stable value as the quantity of simulation runs exceeds ~2,000. Thus, all analyses are conducted on the basis of 3,000 simulation runs.

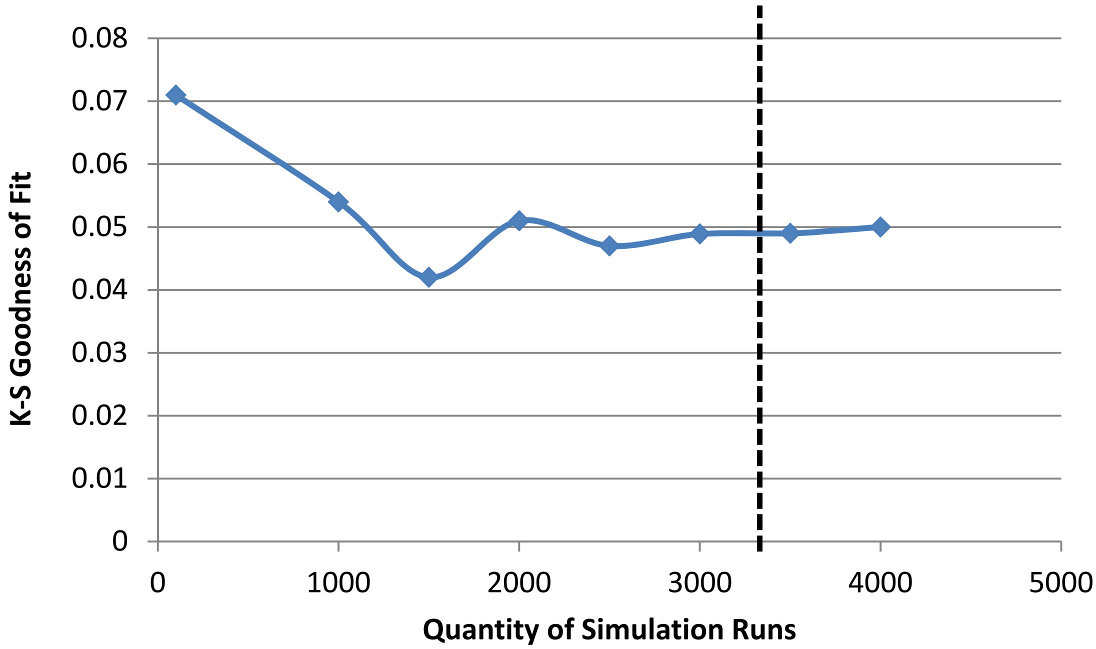


Figure 8.2 – Run quantity analysis for three-layer soil system based on kolmogorv-smirnov goodness of fit

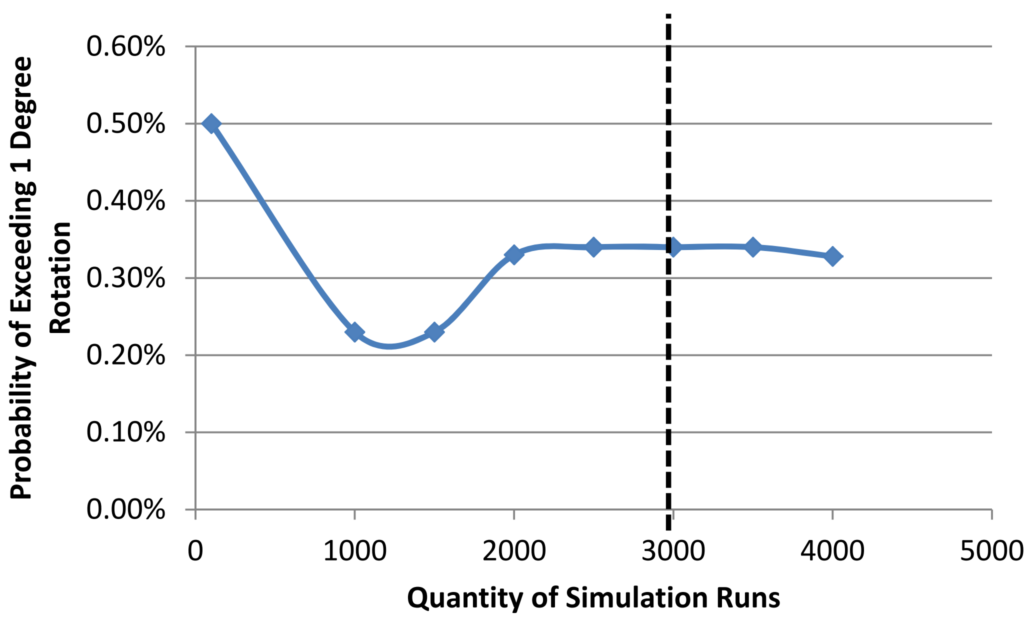


Figure 8.3 – Run quantity analysis for three-layer soil system based on probability of exceeding rotation criteria

8.2 Model Input Data

To the extent possible, the simulation model is developed to represent the dataset recovered from the ABL-PC 500kV Transmission Line Project (Section 7). Four (4) differing soil strata are represented in the simulation model, each selected to introduce variations in soil conditions, data quality and quantity. For the purposes of this research, the statistical variation of soil strength is of greater interest than discrete measurement values taken from the ABL-PC project. As a simplifying measure, statistical variations in soil strength are computed according to observed variability in the SPT blow count (N_{60}) for each soil layer identified by the project. Soil strength parameters (φ, c, E_p) for each discrete simulation run are computed on the basis of the SPT blow count correlations identified (Section 8.1.2). Thus, foundation performance generated by the model is derived from theoretical soil strata (strength parameters derived from simplifying correlations) that follow the statistical variability observed from field SPT data.

8.2.1 *Soil Data Source*

Soil behavior for the purposes of the reliability simulation is sourced from the ABL-PC geotechnical database. Actual data taken from the database comes in the form of soil classification, stratification, N_{60} values and the associated statistical variations therein. The strata selected for simulation are limited to the four design soil profiles identified as Soil Zones 1, 2A, 2C and 3C identified by the analysis procedure discussed in Section 7.4 of this document and described below. The zones are selected from the larger project database to provide a representative variation in soil profiles, based on the dataset in terms of material type, the quantity and quality of data available within each zone.

Zone 1:

Zone 1 is the largest soil zone encountered on the ABL-PC project covering approximately 30% of the corridor. Zone 1 has the most well developed database, incorporating information from 10 boring locations. Identified as a basin fill deposit, this zone is characterized by highly variable interbedded layers of low to high plasticity sandy/silty clays, non-plastic to low plasticity silty and clayey sands and medium to high plasticity cemented sandy clays. Soils generally increase in density and cementation below 13 ft. For the purposes of foundation design, Zone 1 is represented as a two-layer system with nominal strength parameters as noted (Table 8.2.1).

Zone 2A:

Zone 2A represents a less extensive portion of the ABL-PC project covering approximately 15% of the corridor. Zone 2A has correspondingly smaller database, with information from 5 boring locations. Identified as an alluvial fan/plain deposit, this zone is characterized by fine-grained clays and sandy clays underlain by medium to high plasticity fine-grained soils with strong cementation to depth. For the purposes of foundation design, Zone 2A is represented as a three-layer system with nominal strength parameters as noted (Table 8.2.1).

Zone 2C:

Zone 2C represents a small portion of the ABL-PC project covering approximately 7% of the corridor and is represented by 4 boring locations. As a sub-designation of Zone 2, it has similar origins as Zone 2A as an alluvial fan/plain deposit. However, 2C is characterized by larger particle sizes in the form of coarse-grained clayey/silty sands underlain by low plasticity gravels to depth. For the purposes of foundation design, Zone 2c is represented as a two-layer system with nominal strength parameters as noted (Table 8.2).

Zone 3C:

Zone 3C is similar in size to 2C, covering approximately 7% of the corridor and is represented by 4 boring locations. Zone 3 and its associated sub zones are characterized as young alluvial fan deposits. 3C generally exhibits clayey sands and sandy clays over cemented sand, gravel and cobble deposits to depth. For the purposes of foundation design, Zone 3C is represented as a three-layer system with nominal strength parameters as noted (Table 8.2).

Table 8.2 - ABL-PC Soil stratification and strength parameters

(Actual project design values)

Design Zone	Depth Below Grade (ft)	Strength Properties					
		N (bl/ft)	γ_t (pcf)	Φ (°)	c (ksf)	Ep (ksi)	COV
1	0-12.5	12	110	25	1.0	0.8	0.40
	12.5-30.0	23	115	30	1.0	2.7	0.30
2A	0-7.0	11	110	40	0.9	0.7	0.36
	7.0-13.0	28	118	43	1.0	3.5	0.25
	13.0-30.0	43	108	40	1.5	6.0	0.28
2C	0-13.0	15	108	38	0.6	1.3	0.36
	13.0-30.0	23	113	40	0.5	4.0	0.44
3C	0-7.0	11	104	35	0.45	0.8	0.48
	7.0-17.0	26	120	36	0.5	4.5	0.43
	17.0-30.0	50	127	40	0.0	5.0	0.41

8.2.2 *Soil Data Variability*

The theoretical foundation performance model is a Monte Carlo simulation analysis that incorporates soils data recovered from the ABL-PC project database. To ascertain the reliability performance of theoretical foundation designs, soil variability observed during the project geotechnical investigation is incorporated. For the purposes of this analysis, corrected SPT blow count (N_{60}) is employed as the parameter for representation of the in situ soil strength variability. Observed soil variability is simulated through a curve fitting process in which blow count data is analyzed and assigned a representative probability density function. The probability function is then carried forward into the reliability simulation as a mathematical representation of in situ soil variability.

Soil variability is commonly represented according to a lognormal probability density function that is useful from a probability perspective due to the prohibition of negative strength parameters (a physical impossibility). However, lognormal distributions asymptotically approach infinite strength values as well, which may tend to skew representations of the observed N_{60} distributions. To better capture the finite range of probable blow count values, a Beta distribution is fit to each soil layer, allowing for assignment of minimum and maximum blow count values according to the observed dataset. Basic parameters pertaining to each dataset are presented, Table 8.3, with PDF fitting results shown by Figure 8.4 for Soil Zone 1, Layer 1. Results for the remaining strata are provided in Appendix A.

Table 8.3 – ABL-PC Soil stratification; variability statistics on N60 values

Design Zone	Depth Below Grade (ft)	Variability Parameters					
		Sample Size	COV (%)	Beta Distribution Parameters			
				Min.	Max.	α	β
1	0-12.5	34	40	4.4	35	1.7	2.8
	12.5-30.0	36	30	2.6	51	3.4	3.2
2A	0-7.0	13	36	5.0	31	2.0	3.4
	7.0-13.0	13	25	8.9	61	4.5	4.5
	13.0-30.0	22	28	7.3	87	4.1	4.0
2C	0-13.0	21	36	6.0	43	2.1	3.3
	13.0-30.0	15	44	13.2	78	1.4	2.9
3C	0-7.0	11	48	7.0	45	1.5	3.0
	7.0-17.0	10	43	11.4	80	1.6	3.1
	17.0-30.0	8	41	43	165	0.9	2.3

Beta distribution fitting procedures employ an estimation of minimum and maximum values. Each value is established using the most extreme of either values observed in the dataset or the mean value plus and minus three standard deviations (>99% confidence level). This method of fitting is an approximation and is based on the assumption that the data follows a Gaussian distribution. This is an inherent source of error for the purposes of fitting to an existing dataset. However, given the desire to observe foundation behavior on the basis of theoretical soil variability, this simplifying measure is left in place.

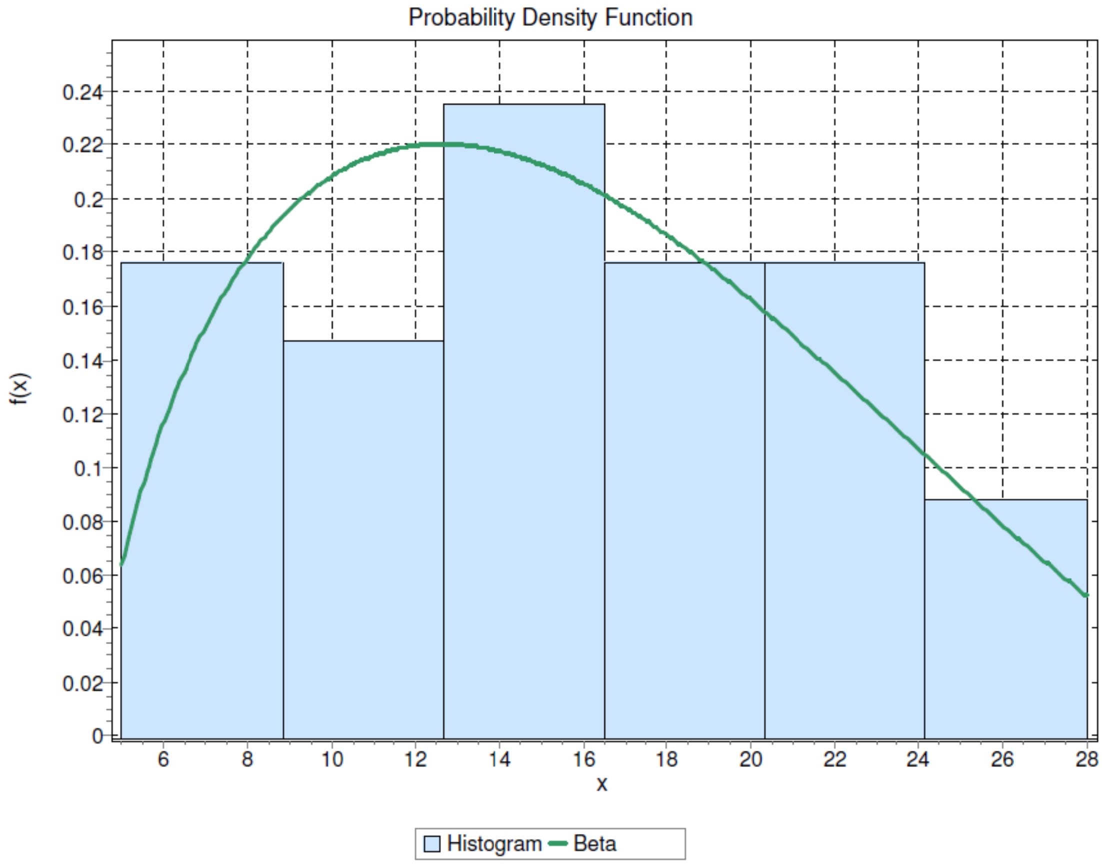


Figure 8.4 –Beta distribution PDF/ blow count histogram

Zone 1, Layer 1

8.2.3 *Theoretical Soil Strength Parameters*

Nominal soil strength properties are derived in accordance with the methods identified for each design method of interest. Where specified by the design method, probabilistically derived values (lower exclusion limit values) are computed on the basis a normal distribution parameters established for each soil zone and layer unless otherwise specified. The basis of nominal values specified by each design approach is summarized in Table 8.4.

Table 8.4 - Nominal value selection parameters by design method

Design Method	Nominal Value for Ultimate Capacity	Nominal Value for Service Limits
FHWA	Mean	Mean
EPRI	Mean	Not Specified
Eurocode	5% LEL on mean value (Factored)	5% LEL on mean value
Site Specific Variability	5% LEL on mean value	5% LEL on mean value
	10% LEL (Beta Dist)	10% LEL (Beta Dist)

In accordance with each design method, a series of design soil profiles are derived in accordance with the statistical metrics specified Tables 8.5 to 8.8. For the purposes of the simulation, soil layer thicknesses and depths are selected in accordance with those specified by the ABL-PC design profiles. The stratification defined for the ABL-PC project was defined employing statistical methods discussed in Section 7 of this document and is compatible with the goals of the simulation procedure at large.

Individual soil strength parameters are calculated from correlations to the N_{60} blow count in lieu of computing each value based on the equivalent statistical metric from the ABL-PC database. This is done as a simplification in recognition of the limited data available for various parameters and the associated uncertainty in computing their corresponding PDF. With the exception of unit weight, each strength parameter is statistically linked with the N_{60} value, thus a low bound N_{60} value will similarly result in low bound values for friction angle, cohesion and lateral modulus. Due to the limited influence on design outcomes and relative certainty in its measurement, unit weight is held constant in accordance with the design values used on the ABL-PC project. A complete description of the correlations used in the simulation model is provided in Section 8.2.3. Of the design documents referenced, Eurocode is the only method that employs strength factors applied directly to soil strength components. Soil parameters for the Eurocode ultimate capacity case are derived in accordance with the factors noted in Section 6.1 of this document.

Table 8.5 - Theoretical Zone 1 nominal soil strength parameters

Statistical Metric	Depth Below Grade (ft)	Strength Properties				
		N_{60} (bl/ft)	γ_t (pcf)	Φ (°)	c (ksf)	Ep (ksi)
Mean	0-12.5	15.9	110	31.5	1.16	2.32
	12.5-30.0	26.6	115	31.9	1.90	3.68
5% LEL of Mean	0-12.5	14.1	110	30.3	1.03	2.05
	12.5-30.0	24.4	115	31.0	1.74	3.35
Eurocode Factored	0-12.5	14.1	110	26.1	0.93	1.76
	12.5-30.0	24.4	115	26.5	1.52	2.63
10% LEL (Beta Dist)	0-12.5	7.9	110	25.8	0.60	0.76
	12.5-30.0	16.0	115	27.1	1.14	2.07

Table 8.6 - Theoretical Zone 2A nominal soil strength parameters

Statistical Metric	Depth Below Grade (ft)	Strength Properties				
		N_{60} (bl/ft)	γ_t (pcf)	Φ (°)	c (ksf)	Ep (ksi)
Mean	0-7.0	14.7	107	33.0	0.38	1.11
	7.0-13.0	35.1	116	38.6	1.35	7.08
	13.0-30.0	47.3	105	39.6	1.94	8.62
5% LEL of Mean	0-7.0	12.1	107	31.0	0.26	0.79
	7.0-13.0	30.8	116	36.7	1.15	6.49
	13.0-30.0	42.4	105	37.8	1.70	8.02
Eurocode Factored	0-7.0	12.1	107	25.7	0.21	0.57
	7.0-13.0	30.8	116	30.8	0.92	4.64
	13.0-30.0	42.4	105	31.9	1.36	5.73
10% LEL (Beta Dist)	0-7.0	8.0	107	27.8	0.08	0.41
	7.0-13.0	23.5	116	33.8	0.84	5.55
	13.0-30.0	29.6	105	33.2	1.10	6.36

Table 8.7 - Theoretical Zone 2C nominal soil strength parameters

Statistical Metric	Depth Below Grade (ft)	Strength Properties				
		N_{60} (bl/ft)	γ_t (pcf)	Φ (°)	c (ksf)	Ep (ksi)
Mean	0-13.0	20.7	107	34.2	1.08	2.40
	13.0-30.0	33.7	107	34.6	0.50	4.69
5% LEL of Mean	0-13.0	17.9	107	32.6	0.65	1.97
	13.0-30.0	26.9	107	32.0	0.50	3.44
Eurocode Factored	0-13.0	17.9	107	27.1	0.52	1.41
	13.0-30.0	26.9	107	26.6	0.40	2.46
10% LEL (Beta Dist)	0-13.0	13.8	107	28.5	0	1.10
	13.0-30.0	16.2	107	28.3	0.50	2.01

Table 8.8 - Theoretical Zone 3C nominal soil strength parameters

Statistical Metric	Depth Below Grade (ft)	Strength Properties				
		N_{60} (bl/ft)	γ_t (pcf)	Φ (°)	c (ksf)	Ep (ksi)
Mean	0-7.0	18.5	104	35.7	0.50	2.06
	7.0-17.0	34.7	120	37.3	0.50	4.89
	17.0-30.0	73.5	127	46.4	0.00	11.04
5% LEL of Mean	0-7.0	13.5	104	32.3	0.50	1.37
	7.0-17.0	26.0	120	33.6	0.50	3.28
	17.0-30.0	53.1	127	39.9	0.00	7.76
Eurocode Factored	0-7.0	13.5	104	26.8	0.40	0.98
	7.0-17.0	26.0	120	28.0	0.40	2.34
	17.0-30.0	53.1	127	33.8	0.00	5.54
10% LEL (Beta Dist)	0-7.0	10.3	104	29.7	0.50	0.96
	7.0-17.0	18.2	120	30.0	0.50	2.03
	17.0-30.0	47.0	127	37.9	0.00	6.80

8.2.4 *Foundation Design Procedure*

To evaluate theoretical foundation performance, a series of foundation designs was derived in accordance with the design procedures identified herein. Each foundation design was developed in accordance with actual NESC factored structure loads recovered from the ABL-PC project database. Theoretical foundation designs within Zones 1, 2A and 3C utilize the loads and pier diameter for a 5DCA30-145-2 which is a 145ft tall single-shaft tubular steel pole structure supporting two circuits in a dead-end configuration about a 30 degree line angle. Alternatively, theoretical foundation designs developed for Zone 2C apply the loads and pier diameter for a 5DCT-160 which is a 160ft tall single-shaft tubular steel pole structure supporting two circuits in a tangent configuration about a nominal 1 degree line angle. The structures considered for each design zone are selected to provide loading of sufficient magnitude to warrant foundation designs with depths in excess of the nominal 2B depth, thus providing results that are informative from a reliability-based design perspective. Foundation diameters are established on the basis of a single reinforcing cage configuration in which the structure anchor bolts act as longitudinal reinforcing and extend to the full depth of the foundation. Correspondingly, foundation diameters are determined in accordance with the anchor bolt circle diameters associated with each structure and the only dimensional variable for foundation design is depth below grade. Nominal pier diameters and structure loading within each design zone is provide in a Table 8.9.

Table 8.9 - Structure load and diameter criteria by soil zone

Design Zone	Structure Type	Pier Diameter (ft)	Reveal Height (ft)	Shear Load (kips)	Moment Load (Ft-kips)	Axial Load (kips)
1	5DCA30-145-2	8	2	118	12589	95
2A	5DCA30-145-2	8	2	118	12589	95
2C	5DCT-160	7	2	59	6912	80
3C	5DCA30-145-2	8	2	118	12589	95

Theoretical foundation designs are carried out in accordance with the soil parameters and stratification identified in Section 8.1.3. The foundation design methodology is that employed on the ABL-PC project, based in rigid pier design as established by the MFAD computer program (Section 4.4). Foundation performance criteria are similarly derived from the ABL-PC project methodology (Table 8.10).

Table 8.10 - Allowable foundation movements

Structure Type	Total Disp. (in)	Total Rotation (°)	Non-recoverable Disp. (in)	Non-recoverable Rotation (°)
5DCA30-145-2	3.84	1	1.92	0.5
5DCT-160	3.36	1	1.68	0.5

8.2.4.1 *Service Limit Design*

For laterally loaded transmission structure foundations, service limit criteria (such as rotation limits) generally govern design rather than ultimate capacity. Properly defined deflection limits are often linked to movements within the elastic portion of the load-deflection response of the foundation, beyond which movement predictions become less certain (Kandaris et al., 2012). It is desirable to achieve some level of margin against foundation movement within the plastic range of the load-deformation response. However, procedures for doing so are generally not specified.

Each design document provides different guidance about service limit design. EPRI does not provide guidance on design where service limits govern. If service limits are not satisfied under the factored design strength case, the designer may choose any number of approaches when using the EPRI guideline. Two likely choices are:

a) Incrementally increase the foundation depth until service limits are satisfied.
b) Determine the nominal capacity at which service limits are satisfied and increase the foundation depth to achieve a desired margin of safety.

The EPRI design method is predicated on the use of mean soil strength parameters, thus design scenarios where either service limits are satisfied under the factored design strength case, or option 'a' is implemented; the margin of safety against plastic foundation deformations is likely smaller than that against exceeding ultimate foundation capacity. Option 'b' (defined above) is the only approach that provides a defined margin of safety against excessive foundation movement in the absence of a reliability based design procedure for service limit design.

FHWA employs mean soil strength parameters. However, it explicitly specifies a resistance factor of 1.0 for lateral loading where service limits are of interest under nominal loading conditions. This approach is equivalent to option 'a' and, as prescribed, does not provide a margin of safety against plastic foundation deformations.

To evaluate the implications of each approach from a reliability perspective, theoretical foundation designs are derived for each option. The margin of safety assigned to service limits is that of ultimate capacity for each document, 0.63 and 0.67, for EPRI and FHWA respectively. Service limit design in accordance with Eurocode applies a strength factor of 1.0. However, nominal soil strength parameters are selected at the 5% LEL of the mean value.

The theoretical foundation dimensions derived for each design scenario are presented in Table 8.11. MFAD results for each design are provided in Appendix B.

Table 8.11 - Theoretical foundation dimensions by soil zone

Design Zone	Design Method	Diameter (ft)	Depth (ft)
1	EPRI /FHWA Ultimate*	8	18
	EPRI/FHWA Service*	8	22
	Eurocode Ultimate	8	18
	Eurocode Service Limit	8	19
	5% LEL Mean Soil	8	19
	10% LEL Beta Soil	8	26
2A	EPRI /FHWA Ultimate*	8	16
	EPRI/FHWA Service*	8	19
	Eurocode Ultimate	8	18
	Eurocode Service Limit	8	17
	5% LEL Mean Soil	8	17
	10% LEL Beta Soil	8	19
2C	EPRI /FHWA Ultimate*	7	14
	EPRI/FHWA Service*	7	18
	Eurocode Ultimate	7	18
	Eurocode Service Limit	7	16
	5% LEL Mean Soil	7	16
	10% LEL Beta Soil	7	26
3C	EPRI /FHWA Ultimate*	8	20
	EPRI/FHWA Service*	8	22
	Eurocode Ultimate	8	22
	Eurocode Service Limit	8	21
	5% LEL Mean Soil	8	21
	10% LEL Beta Soil	8	23

* Ultimate cases utilize a resistance factor of 1.0 to evaluate foundation dimensions that satisfy deflection limits, whereas service limit cases increase foundation capacity of the unfactored case by 1.6 (1/resistance factor) to achieved a ‘factored’ service limit design.

8.2.4.2 *Consideration of Homogeneous Soil Profile as a Special Case*

To understand the influence assumptions on input soil variability have on reliability outcomes, a series of homogeneous soil layer analyses are conducted. Each homogeneous case assumes a particular design soil layer from the ABL-PC database extends to a depth well below the extent of a theoretical foundation design developed for the homogeneous soil layer. Through a series of three Monte Carlo simulations for each theoretical foundation design, the assumed probability density function governing the variability of soil strength parameters is manipulated. Each simulation utilizes either a normal, lognormal and beta distribution with parameters computed from the soil layer dataset of interest.

Two theoretical foundation designs are selected representing differing soil conditions; Layer 1 from Zone 1, as a relatively soft cohesive layer, and layer 2 from Zone 3C, as a much stiffer and less cohesive layer. Theoretical foundation designs for each homogeneous case employ soil parameters computed on the basis of a 10% LEL value according to a Beta distribution (Tables 8.5 and 8.8) with load, anchor bolt and service limits corresponding to a 5DCA30-145-2 structure (Tables 8.9 and 8.10). The resulting pier depths are 31ft and 23ft for Zones 1 and 3C respectively. Soil variability parameters derived for each homogeneous layer are provided in Table 8.12.

Table 8.12 – Homogeneous soil layer PDF parameters

Layer	Beta Dist.				Lognormal Dist.		Normal Dist.	
	α	β	Min.	Max.	μ	σ	μ	σ
Zone 1, Layer 1	1.7	2.8	4.4	35	2.7	0.4	15.9	6.4
Zone 3C, Layer 2	1.6	3.1	11.4	80	3.5	0.4	34.7	15

Simulation results for each set of input PDF model are evaluated in consideration of a constant load input. Thus, the results are only indicative of the influence the input PDF has on resulting rotation performance CDF (Fig. 8.5).

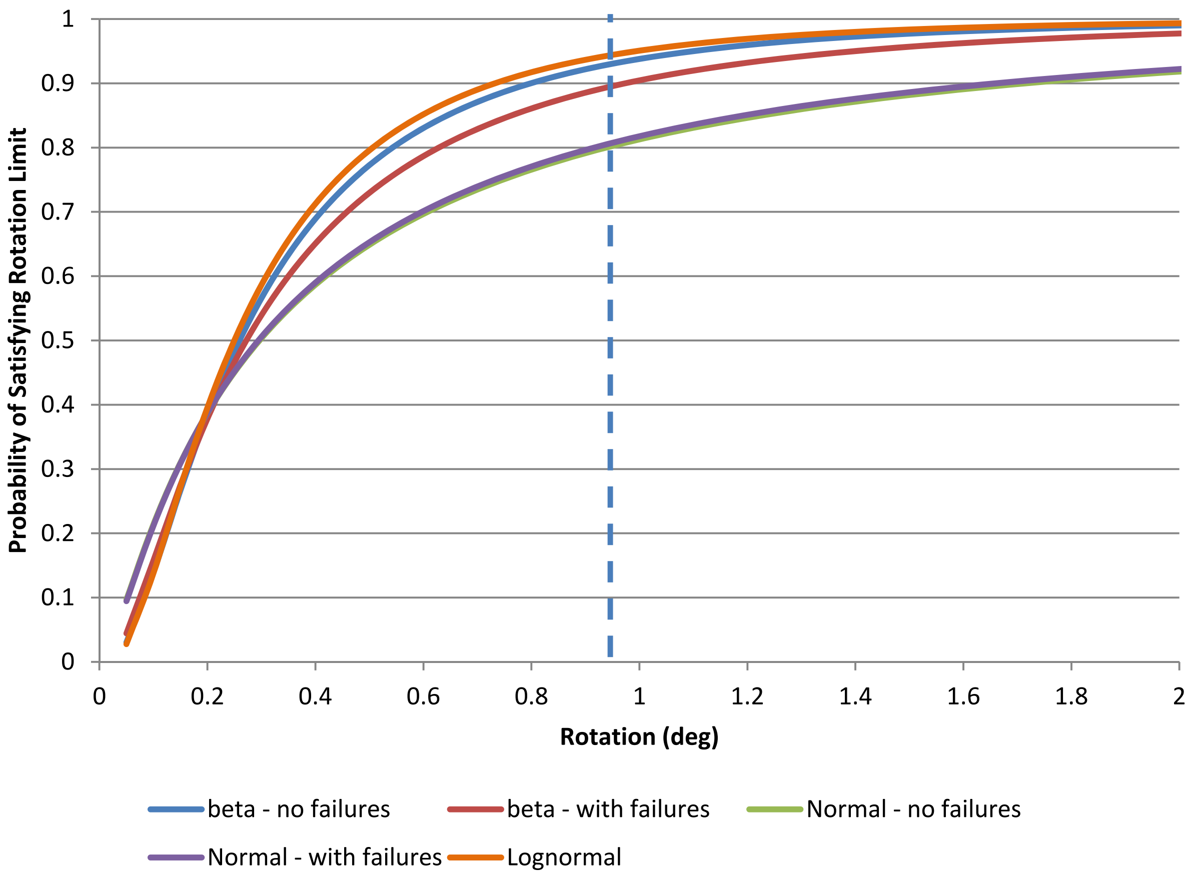


Figure 8.5 – Rotation performance CDF for variable input soil PDF's

Rotation performance results are evaluated for each input soil variability PDF at the performance threshold of 1° at the top of the pier (Table 8.13). When high deflections are calculated, the MFAD analysis model generates text results in lieu of numerical results indicating highly non-linear rotation performance. The normal input PDF in particular, results in a higher number of non-numerical rotation results due to unrealistically low input parameters generated by the normal distribution. For the purpose of establishing estimates on rotation performance, non-numerical rotation values are replaced with linearly extrapolated values to generate the foundation performance PDF and CDF as needed (Fig. 8.5). These extrapolated values account for 6% of the sample population with a normal input distribution, <1 % for the beta distribution and 0% of the lognormal distribution results.

Table 8.13 - Monte Carlo Simulation results by input PDF

Design Zone	Calculated Probability of Exceeding 1 Degree Rotation by Input Distribution				
	Beta (Failures Excluded)	Beta (Failures Included)	Normal (Failures Excluded)	Normal (Failures Included)	Lognormal
1	6.3%	9.6%	18.8%	18.3%	5.0%
3C	19.0%		27.3%		16.2%

The results indicate the normal input PDF generally yields invalid results due to unrealistic soil strength inputs. The beta and lognormal distributions generate comparable performance results. A beta input distribution is applied for all simulations represented herein. The beta and lognormal distributions similarly yield a foundation performance output PDF which is well represented by a lognormal distribution. All subsequent performance analyses are carried out based upon a lognormal performance output distribution.

8.3 Foundation Performance Analysis Results

Each theoretical foundation design is evaluated by the performance model considering variable soil and load inputs. The performance metrics of interest are lateral moment capacity and rotation evaluated from the perspective of computed levels of reliability. Assessment of moment capacity relative to the variable load input allows computation of the reliability index, β (Section 5.4). Where applicable, the computed reliability indices for each theoretical foundation design is compared to the stated reliability index goal of the code document from which it was derived. Foundation performance is evaluated in consideration of the probability the design under consideration will exceed the prescribed limit of 1 degree of rotation, referred to herein as the probability of exceedance, P_e.

8.3.1 *Homogeneous Profile Analysis*

To ascertain the influence soil layering has on foundation performance, a special homogeneous case is considered. The homogeneous layer of Zone 1 (Section 8.2.4.2) is divided into two independent layers within the Monte Carlo simulation model. In terms of nominal design, the two layers are represented by identical soil strength parameters computed on the basis of the 5% LEL of the mean value for the layer of interest. Each layer is derived from identical parameters and therefore, from a nominal design perspective, function as a single homogenous layer yielding a foundation 20ft in depth (see Appendix B for MFAD results). However, the layers are permitted to vary independently of one another in the simulation model as if they are discrete soil layers (Fig. 8.6).

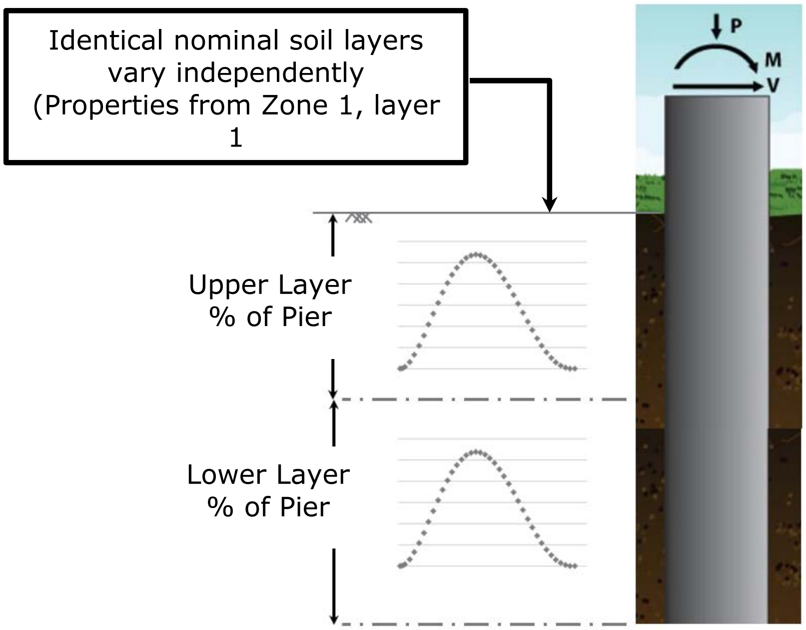


Figure 8.6 - Statistically independent homogeneous layer model

This analysis permits an assessment of the influence the presence of discrete layers has on foundation performance outcomes without the complicating influence of differences in discrete layer strength parameters. The results indicate the presence of independently varying layers increase the reliability of a foundation (Fig. 8.7). Intuitively, this phenomenon has merit based on the theoretical reduction in the likelihood low bound soil strength conditions will occur in both soil layers simultaneously relative to a single homogenous layer. However, the results are only indicative of the transition from a single layer to a two layer system. Presumably, with additional layers the reduced likelihood of coincident low bound values is tempered by reductions in the role each layer plays in the foundation performance. Therefore, it is unclear how computed reliability is impacted with increasing numbers of layers, but it seems likely the influence of each additional layer diminishes.

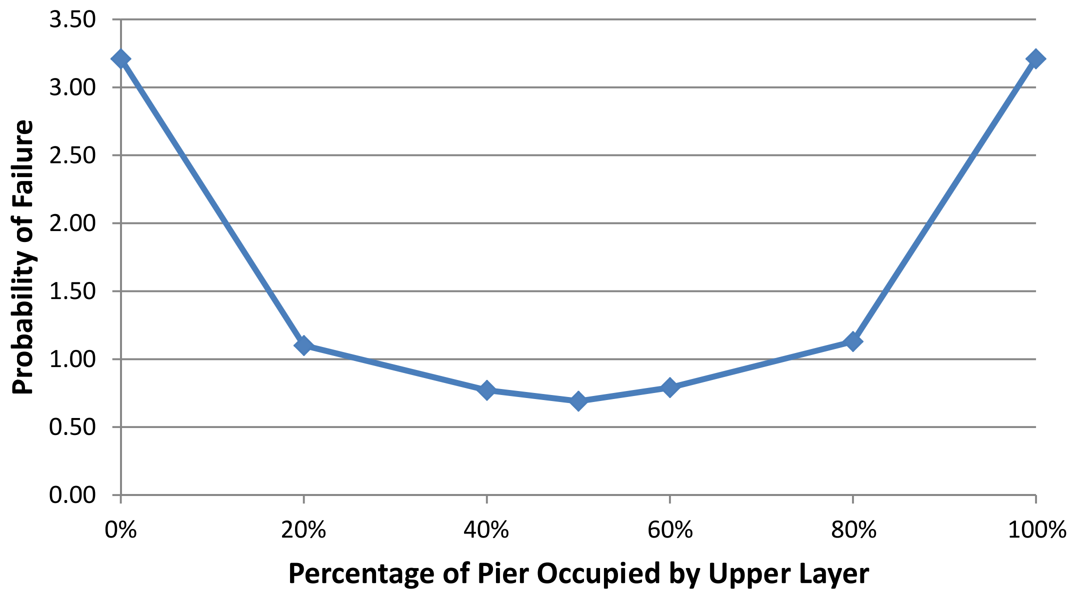


Figure 8.7 – Reliability dependency on layering considering ultimate capacity

Of course, only if soil properties actually differ layer to layer would the computed reliability with additional layering be more representative of field conditions; simply subdividing a given soil layer into finer increments should not affect actual reliability. The question of impact of selected layering on computed reliability requires some additional study.

8.3.2 *ABL-PC Theoretical Foundation Performance Results*

Simulations results derived for each design methodology are discussed individually herein and summarized below (Table 8.14). In general, the results indicate the computed foundation reliability and service limit performance for the theoretical foundations is comparable to industry accepted levels of reliability (Section 5.4). However, some variations in computed reliability are observed for the same design methodology across the soil profiles considered. Notably, Zone 2C consistently yields reliability indices below those of Zones 1, 2A and 3C. This trend is consistent across all design methodologies with the exception of design based upon 10% LEL soil parameters. Due to the high level of variability rotation results observed in the Zone 2C dataset, a number of computation results exceed the elastic range of the MFAD model which generates an error value in lieu of a numerical performance value which can be employed to compute values of P_e. Thus, analysis of the output results without some further evaluation of the non-numeric error results would skew computed values of P_e. Therefore, values of P_e computed for Zone 2C are derived from the actual quantities observed in the dataset which exceed the rotation threshold of 1° instead of fitting a PDF to the numerical portion of the dataset.

The ultimate capacity performance metrics computed are based upon ultimate moment capacity in comparison to nominal moment loads. Computation of reliability performance relative to lateral shear or axial loads is not included as these load components generally contribute to, but do not govern foundation size. Similarly, P_e is computed on the basis of exceeding 1 degree of foundation rotation under the variable load condition employed by the model. Calculation of the probability of exceeding lateral movement is excluded due to the equivalent nature of pier rotation performance in comparison to lateral deflection for rigid piers.

Table 8.14 – Summary of reliability and service limit performance results

	Design Method	Fdn Depth (ft)	Mean FS	Ultimate Capacity β	Ultimate Capacity P_f (%)	Rotation P_e (%)
Zone 1	EPRI/FHWA Ultimate	18	3.70	2.51	0.70	16.90
	EPRI/FHWA Service	22	6.06	3.73	0.02	1.60
	Eurocode Ultimate	18	3.70	2.51	0.70	16.90
	Eurocode Service	19	4.20	2.85	0.23	10.40
	0.05 LEL Mean Soil	19	4.20	2.85	0.23	10.40
	0.1 LEL Beta Soil	26	8.60	4.94	0.00	0.05
Zone 2A	EPRI/FHWA Ultimate	16	3.80	2.64	0.44	13.63
	EPRI/FHWA Service	19	6.24	3.52	0.03	1.57
	Eurocode Ultimate	18	5.47	3.26	0.07	1.44
	Eurocode Service	17	4.56	2.93	0.23	5.51
	0.05 LEL Mean Soil	17	4.56	2.93	0.23	5.51
	0.1 LEL Beta Soil	19	6.24	3.52	0.03	1.57
Zone 2C	EPRI/FHWA Ultimate	14	5.84	1.77	10.50	*25.60**
	EPRI/FHWA Service	18	8.78	2.79	0.89	*4.80**
	Eurocode Ultimate	18	8.78	2.79	0.89	*4.80**
	Eurocode Service	16	7.21	2.29	3.48	*14.10**
	0.05 LEL Mean Soil	16	7.21	2.29	3.48	*14.10**
	0.1 LEL Beta Soil	26	17.45	4.94	0.00	*0.00**
Zone 3C	EPRI/FHWA Ultimate	20	4.59	2.54	0.26	0.39
	EPRI/FHWA Service	22	5.56	3.15	0.06	0.04
	Eurocode Ultimate	22	5.56	3.15	0.06	0.04
	Eurocode Service	21	5.15	2.84	0.15	1.57
	0.05 LEL Mean	21	5.15	2.84	0.15	1.57
	0.1 LEL Beta Soil	23	6.07	3.46	0.03	0.01

*Empirical value derived from observed quantities in the sample population in lieu of values computed from a PDF

8.3.2.1 *FHWA/EPRI*

Although different code documents, from an analytical perspective, the FHWA and EPRI design procedures yield identical foundation dimensions and are therefore are considered in unison. With the exception of Zone 2C, the FHWA and EPRI designs provide a relatively consistent level of reliability with β ranging from 2.5 to 2.65 (excluding Zone 2C) (Fig. 8.9). As a transmission line specific document, EPRI, 2012 is developed on the basis of a target β of 2.3 in recognition of industry practices. By comparison, where reliability based design concepts are utilized by FHWA to compute design factors; a target β of 3.0 is employed. However, as noted, the FHWA lateral capacity factor is the product of engineering judgment and is not reflective of reliability analyses conducted for other design modes considered in the document.

Application of the EPRI and FHWA documents in practice where service limits govern is uncertain (Section 8.2.4.1), thus service limit performance is presented on the basis of two approaches (Fig. 8.10). The results show rotation performance for foundations derived for ultimate capacity is highly variable and the use of mean value strength parameters yields similarly high probabilities of exceeding rotation criteria. Alternatively, the factored service limit approach yields relatively low probabilities of exceeding service limits of < 5%.

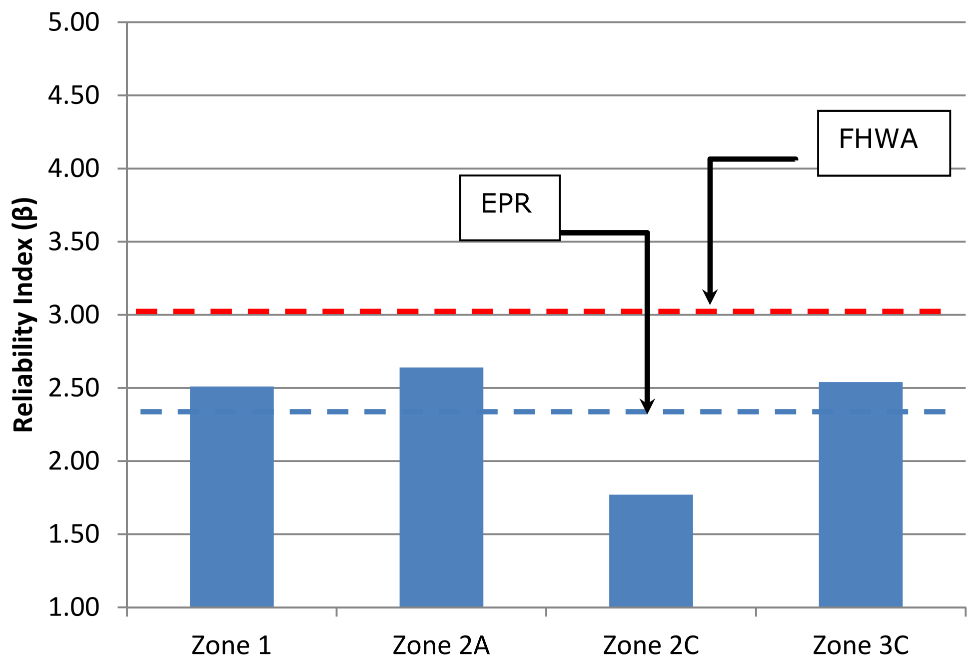


Figure 8.8 – FHWA/EPRI ultimate capacity computed reliability

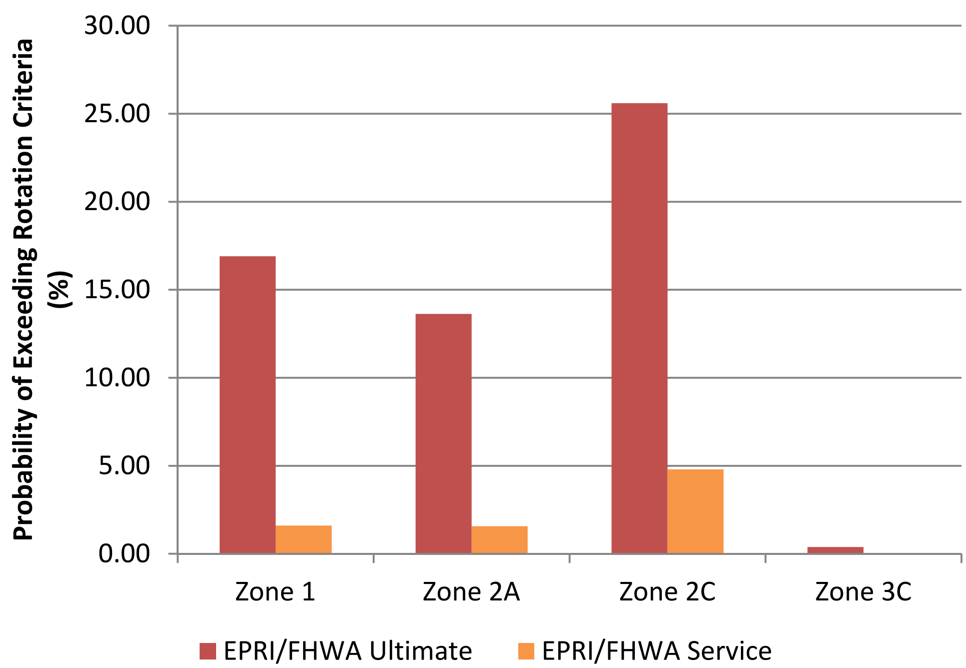


Figure 8.9 – FHWA/EPRI rotation performance

8.3.2.2 *Eurocode*

Generally, the Eurocode design methodology yields more conservative design results than that of EPRI and FHWA. In its application, Eurocode is significantly different than the other code based approaches considered. Of the methodologies considered, Eurocode is only one to employ factored soil parameters in lieu of global resistance factors. It is also the only approach with explicit approaches to both ultimate capacity and service limit design and correspondingly, ultimate capacity design governs for Zones 2A and 3C, whereas service limit design governs for Zones 1 and 2C (Fig. 8.11). The result is fairly consistent levels of reliability across all design zones, including 2C which is problematic for all other design approaches. Reliability indices based on the governing designs varies from 2.8 to 3.3 which is less than the code objective of 3.8, but greater than that required by transmission line industry practice. Some of the disparity in the computed β may be attributable to differences in the load regime about which Eurocode is calibrated in comparison to U.S. codes calibrated on the basis of ASCE 7-05.

In terms of service limit performance, variations in P_e are similarly damped by the exchange of governance amongst ultimate capacity and service limit design considerations. However, P_e is still somewhat variable, ranging from 14% to nearly 0% (Fig. 8.12).

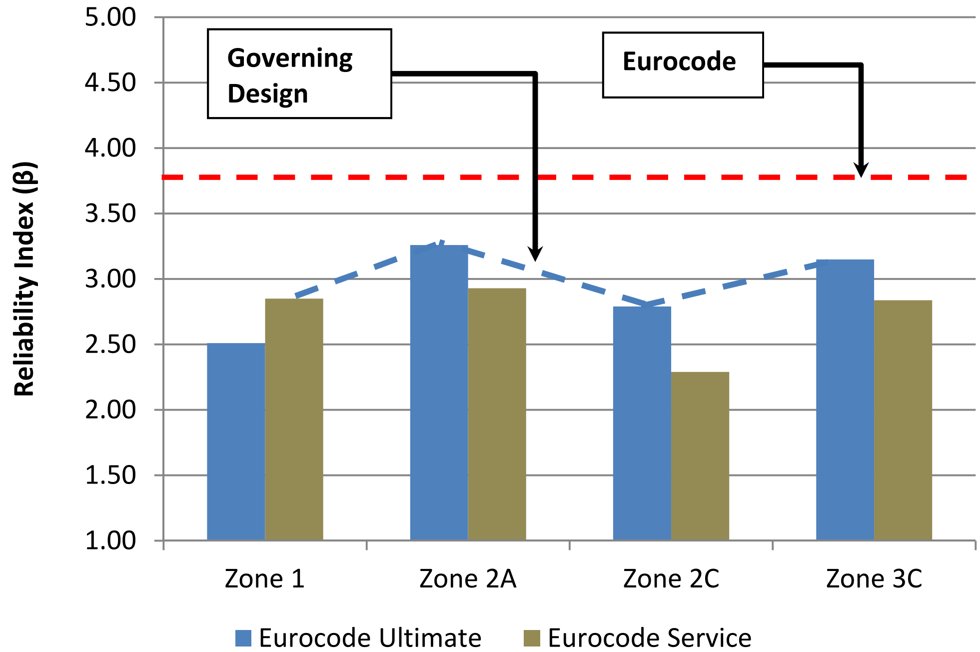


Figure 8.10 – Eurocode computed reliability

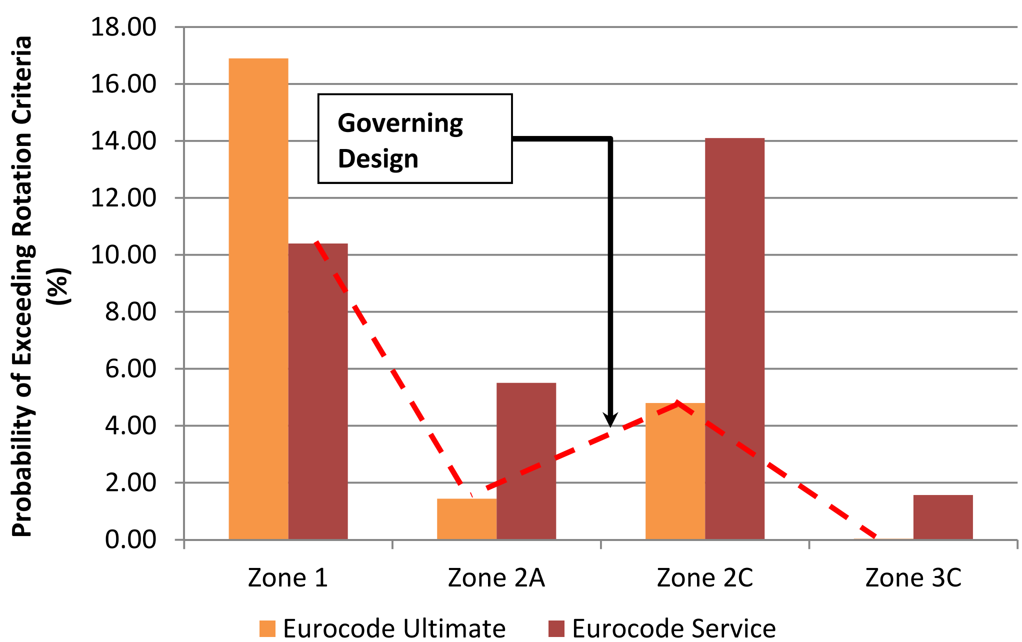


Figure 8.11 – Eurocode rotation performance

8.3.2.3 *5% LEL of Mean Soil Strength*

Of the two site specific soil selection approaches considered, use of the 5% LEL on the mean yields results most comparable to the RBD codes evaluated. Generally, reliability performance yields comparable consistency across the design zones as EPRI and FHWA, although slightly more conservative with β ranging from 2.84 to 2.93, excluding Zone 2C (Fig. 8.13). Similar to other methodologies, Zone 2C yields a reduced β of 2.29. However, this result is compatible with the transmission industry reliability objective of 2.3.

Service limit performance is comparable with the code derived foundation designs with P_e ranging from 1.6% to 14% with Zone 2C as the high deflection value (Fig. 8.14).

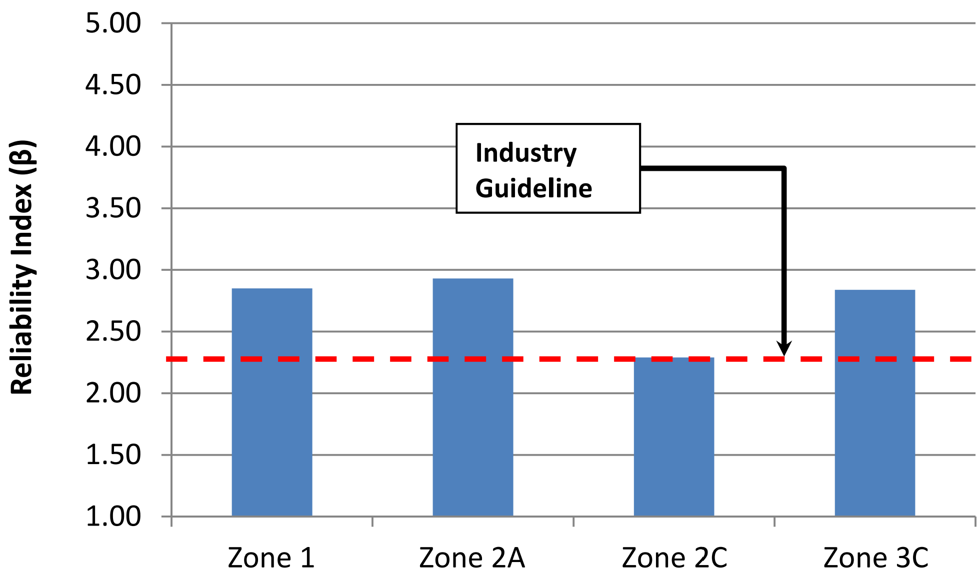


Figure 8.12 – 5% LEL of mean soil strength computed reliability

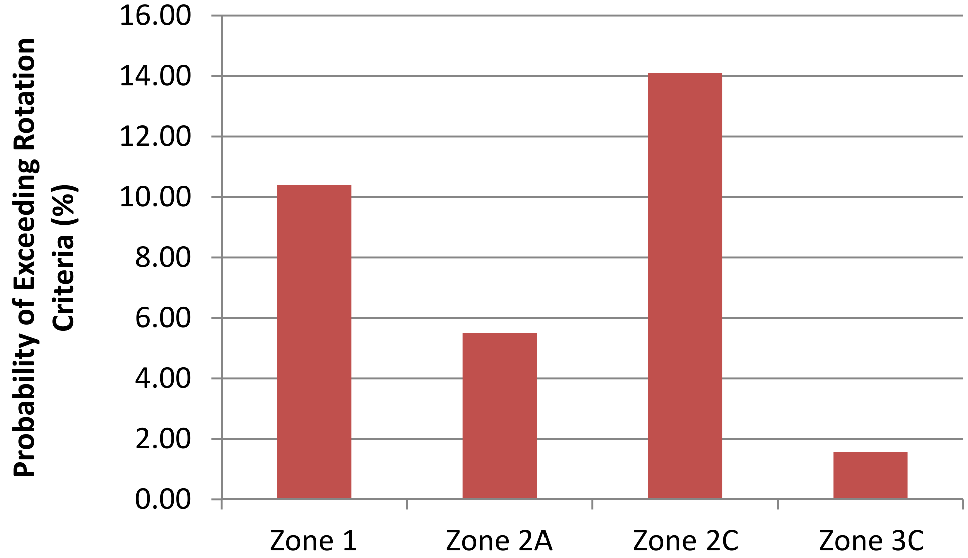


Figure 8.13 – 5% LEL of mean soil strength rotation performance

8.3.2.4 *10% LEL Soil Strength*

Use of lower exclusion limit data predictably yields highly conservative design results, commonly in excess of industry accepted levels of reliability. With highly conservative design also comes greater variability in the computed level of reliability. In practical terms, the highly variable β values are of limited interest. From the perspective of RBD, the values observed are so far in excess of desired practice that reliability computations are themselves less reliability due to the uncertainty associated with computing small probabilities in the presence of highly variable data. Β values computed vary from 3.5 up to 4.95 corresponding to a P_f of 0.0004% (Fig. 8.15).

Service limit computations reveal values for P_e ranging from 5 x 10^{-6}% up to 1.6%, values more commonly associated with ultimate capacity P_f (Fig. 8.16).

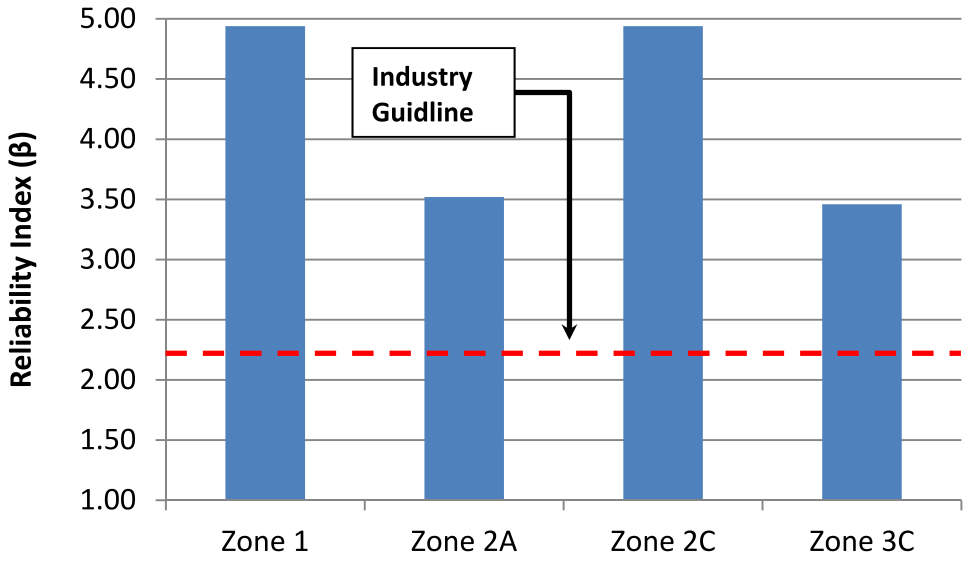


Figure 8.14 – 10% LEL soil strength computed reliability

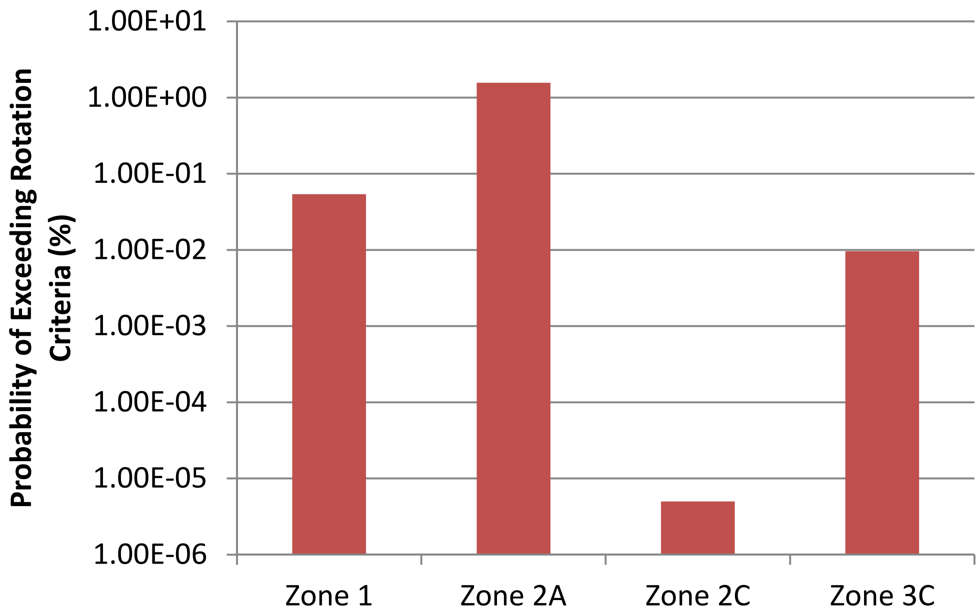


Figure 8.15 – 10% LEL soil strength rotation performance

9 DISCUSSION

Results of the theoretical foundation performance model provide insight about the reliability performance of short, rigid laterally loaded foundations when site-specific strength and spatial variability are considered. Anomalies in reliability levels observed within the dataset make clear the need to consider stratification in the design process to prevent degradation in foundation performance. Similarly, the results highlight the advantages of reliability-based design over deterministic methods, particularly when low bound soil strength parameters are used.

Due to the limited scope of this study, the results are not intended to advocate for one design approach over another. Rather, the objective is to evaluate performance in consideration of design methodologies that differ in their intended application, design equation format and target levels of reliability to understand their implications on reliability when stratification is considered.

9.1 Observed Performance Metrics

9.1.1 *Reliability Performance*

With the exception of the low bound 10% LEL design approach, the methodologies considered in this study generally achieve the objectives of RBD. Across the design scenarios considered, the levels of computed reliability, in consideration of actual site-specific soil variability, are relatively consistent and generally acceptable for transmission line foundations. Also in support of RBD methodologies, the results show design based on lower bound strength parameter yields overly conservative designs that could generate excessive installation costs with unnecessarily high and inconsistent levels of reliability.

Inconsistencies in reliability derived by factor of safety methods are indicated by computation of the mean margin of safety for each theoretical foundation design in comparison to the computed reliability (Fig. 9.1). There is a general trend of increasing reliability with increasing factor of safety. However, across the various design zones, with differing levels of variability in strength parameters, higher factors of safety do not result in higher levels of reliability. This is most notably true in Zone 2C. The mechanisms behind this observation are discussed in Section 9.2 of this document.

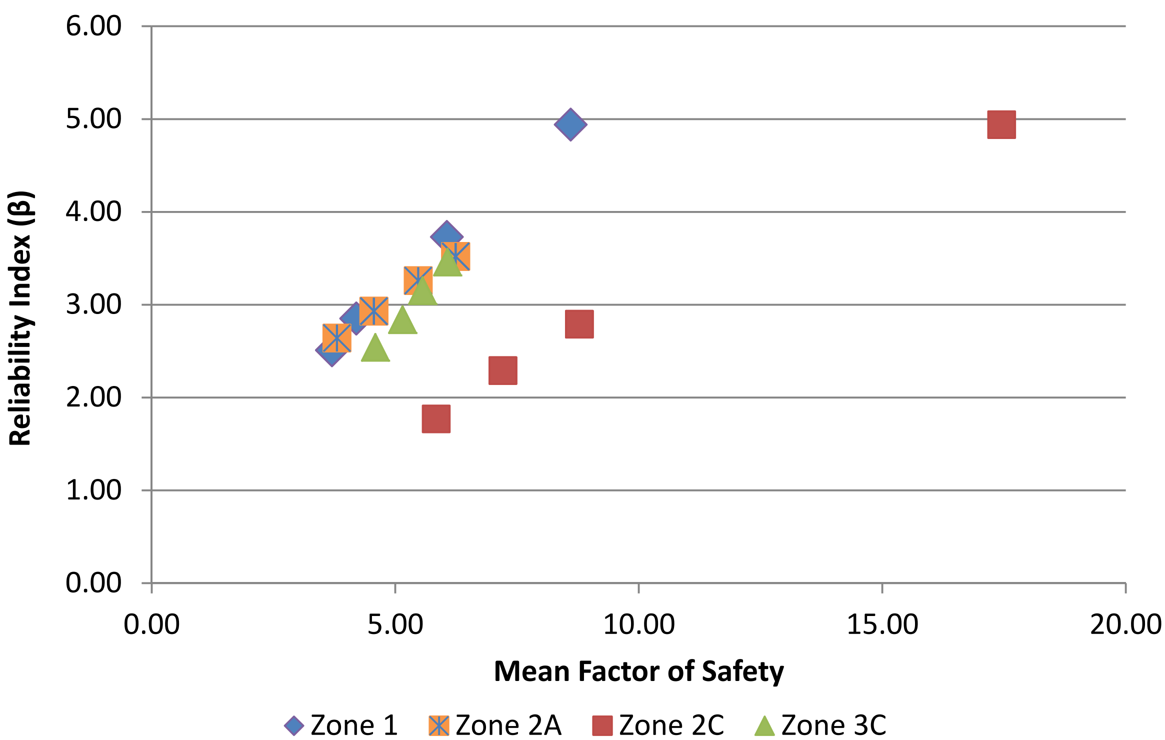


Figure 9.1 - Comparison of computed reliability to mean factor of safety

Although the results indicate the methods examined achieve relatively consistent levels of reliability, the actual reliability thresholds established by each document are largely based on empirically derived objectives. It is important to maintain continuity with existing practice in order to take advantage of the knowledge gained over the history of the geotechnical profession. However, this is not actual reliability based design. As the knowledge base continues to develop within the construct of RBD, further probabilistic assessment of the appropriate level of reliability going forward will be required to achieve the full benefit of reliability methods.

9.1.2 *Service Limit Performance*

The 1 degree rotation service limit threshold is derived from EPRI load test data, which demonstrates rotations on the order of 2 degrees is associated with lateral capacity failure (Kandaris, DiGioia, & Heim, Evaluation of Performance Criteria for Short Laterally Loaded Drilled Shafts, 2012). In the case of short rigid laterally loaded piers, top of pier displacement is inextricably linked to ultimate capacity. Thus, the deflection limit is intended to provide some margin against ultimate failure through derivation of foundation designs that perform in the elastic range of movement (Fig 9.2).

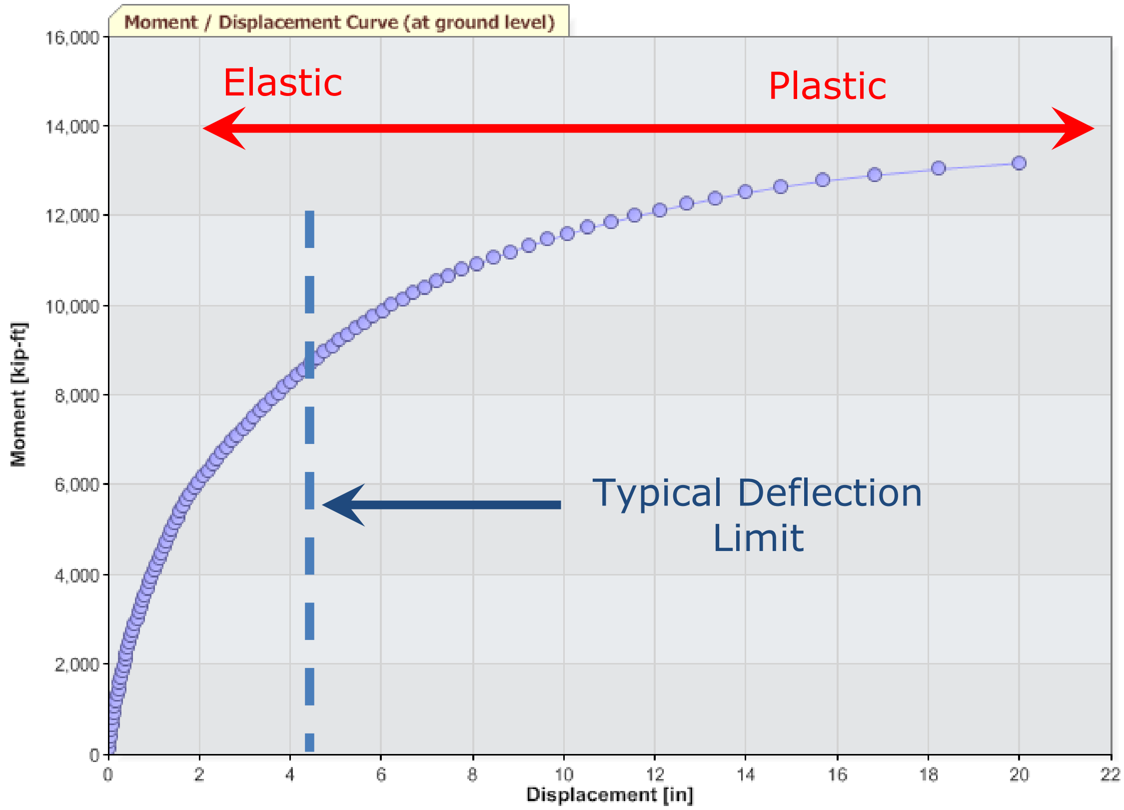


Figure 9.2 - Typical foundation moment displacement curve

To the extent that foundation rotation marginally exceeds 1 degree, the implications are minor. Particularly when only the elastic deformation range is mobilized, higher than anticipated deflections are transient motions that only exist during high wind events. Tubular steel pole structures are generally tolerant of limited foundation movement and provided the p-delta load effect imposed on the supporting structure is not excessive, excess elastic foundation motion is of limited concern.

Given the limited sensitivity of single shaft transmission structures to foundation movement, the probability of exceeding rotation limits should be held to a less stringent P_e compared to P_f for ultimate capacity. Currently, standards for permissible values of P_e due not exist and values compatible with general probabilities of exceedance in civil engineering materials on the order of 5%-10% are noted as plausible values for acceptance. Under this threshold, most theoretical foundation designs perform acceptably, excluding those derived from mean soil strength parameters with no further factor applied for service limit design (FHWA/EPRI Ultimate) and those acting in Zone 2C (Fig. 9.3).

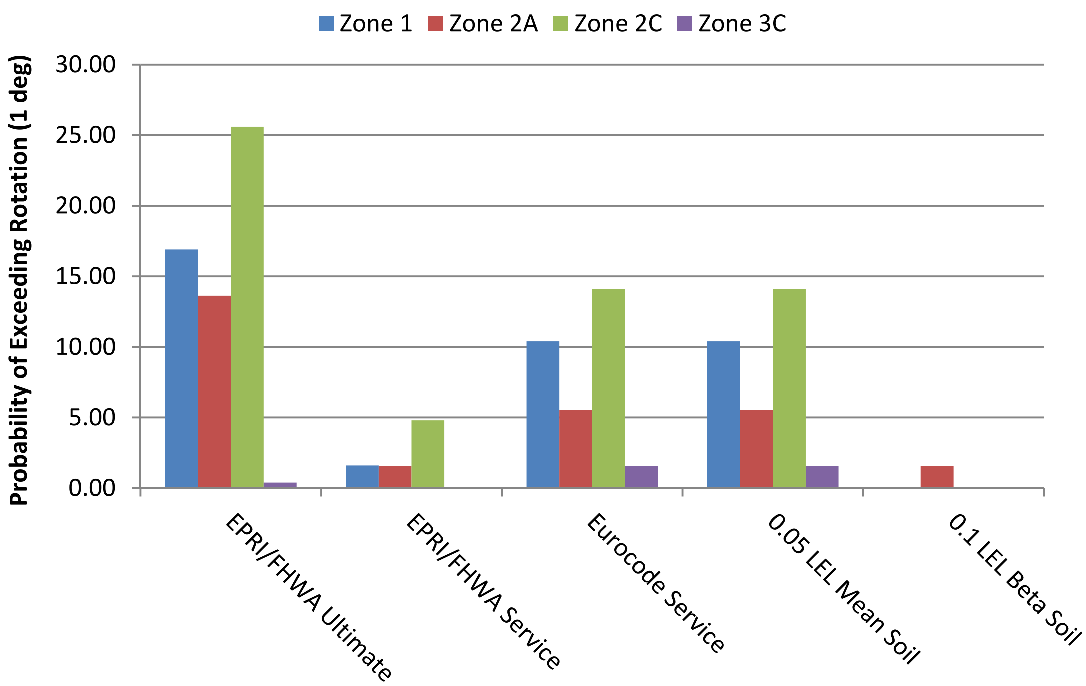


Figure 9.3 - P_e by design method

9.2 Stratification Influences on Reliability

The homogenous analysis case discussed in Section 8.3.1 demonstrates the influence of stratification on foundation reliability from an entirely probabilistic perspective. However, the consistently poorer performance of foundations derived from nominal soil strength properties in Zone 2C suggests a mechanistic influence of much larger magnitude than the phenomenon demonstrated by the homogenous case. The implications of stratification on reliability performance have been noted by others for laterally loaded piles (Fan & Liang, 2013) and axially loaded piers (Cao, Wang, & Wang, 2013).

In situ COV's within each of the design zones vary from 25% to 48% (Table 8.2) with Zone 2A as the least variable and Zone 3C as the most variable. While designs in Zone 2A and Zone 3C achieve nearly identical reliability results, Zone 2C with somewhat typical (for the dataset) COV's of 36%-40% achieves significantly different reliability results. The mechanism that drives this disparity in results is apparent in the soil stratification and relative strength of layers extending over the depth of each foundation (Fig 9.4). Each design presented is based upon nominal 5% LEL mean soil strength parameters to derive foundation depth in each soil zone. Comparison of the stratification along the depth of each foundation reveals, Zone 2C yields a foundation design where a softer upper layer occupies a relatively large portion of the depth (81%) compared to a stiffer underlying layer occupying on 19% of the pier depth. Thus, the foundation as a whole becomes a less reliable system due to the high reliance on the relatively stiffness in layer 2 to derive foundation capacity. As layer 2 exhibits variability in the performance model, the foundation performance is adversely affected.

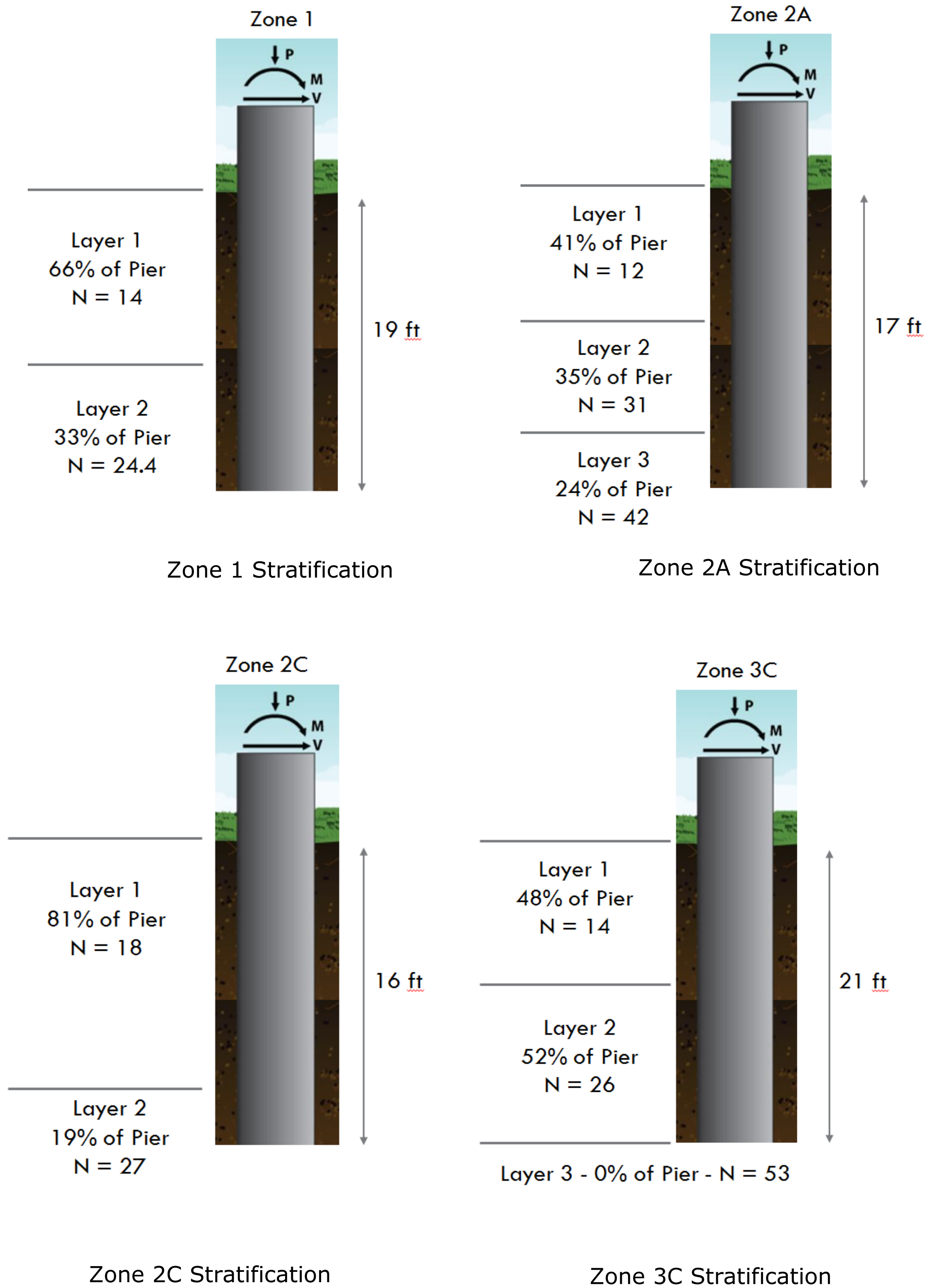
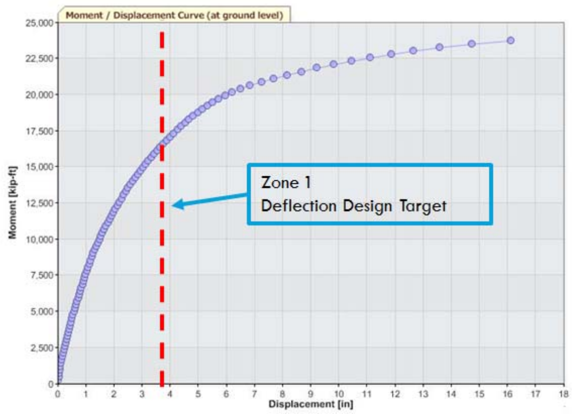


Figure 9.4 - Soil stratification relative to pier depth

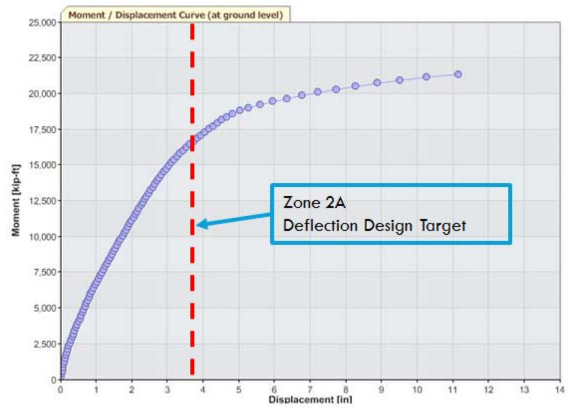
In terms of foundation reliablity performance, Zone 2C theoretical foundations generally satisfy the threshold reliability index of 2.3 with the exception of the 14 ft foundation derived from the EPRI design procedure. For this particular application, a nominal foundation depth of 14 ft is selected, which only extends 1 ft into Layer 2, exacerbating the stratification issues noted for Zone 2C.

From a a practical application perspective, it is infeasible to conduct Monte Carlo simulations during typical design projects to ascertain the reliability of various layering systems. It is therefore useful to make assessments of design viability based upon analysis of the moment-deflection characteristics of a chosen design in comparison to the prescribed delflection limits (Fig. 9.5). Observation of the deflection characteristics for each of the theoretical designs depicted in Fig. 9.5 reveals the design for Zone 2C, although acceptable from an ultimate capacity perspective, is positioned further into plastic portion of the deflection curve.

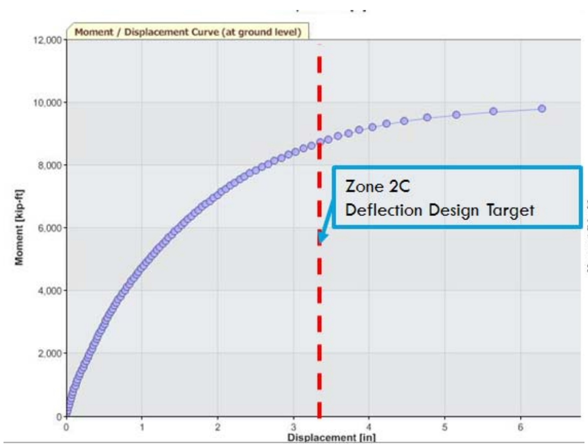
This observation highlights the role that engineering judgement must continue to play as geotechnical practice moves toward RBD methodolgies. From an analytical perspective, the foundations derived for Zone 2C are acceptable. However, engineering judgement in tandem with observation of load-deflection characteristics offers a different conclusion, which ultimately needs to be incorporated in the design process.



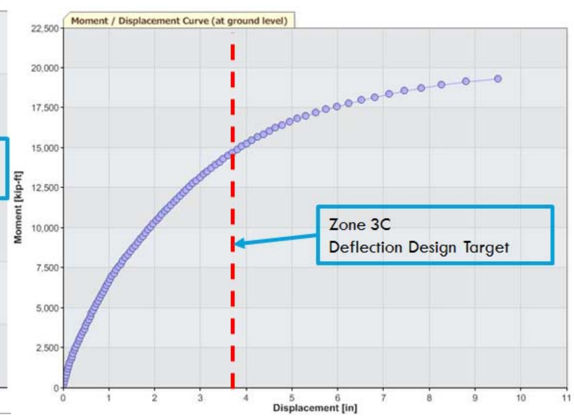
Zone 1 deflection curve



Zone 2A deflection curve



Zone 2C deflection curve



Zone 3C deflection curve

Figure 9.5 - Example load/deflection curves by soil zone

9.3 Analysis Limitations

The analysis presented here is subject to a number of simplifying assumptions, which may affect the accuracy of the reliability results in terms of the absolute values reported, although general trends are probably representative of actual phenomena. The sources of uncertainty in the model may be grouped into three categories: soil database limitations, load analysis compatibility and analysis resolution.

9.3.1 *Assessment of Reliability*

The reliability computed from the foundation performance model is largely derived from correlations to SPT blow count. Therefore the computed values include an assumption the blow count is an unbiased representation of soil strength. It is recognized that the SPT blow count and the associated correlations to strength parameter do in fact contribute bias. However, the intent of this research is to illustrate the influence spatial variability has on reliability in a theoretical environment where such biases are not present. The analysis resulting from the correlations noted is therefore uncertain with regard to its application toward evaluating the reliability achieved on the ABL-PC project in an absolute manner. To the extent theoretical soil profiles can adhere to the behavior represented by the correlation equations, however, the results of this analysis are valid for the purpose of evaluating trends in behavior.

9.3.2 *Soil Database Limitations*

All of the soil variability parameters utilized in the analysis model are derived from the database for the ABL-PC project which is characterized by unsaturated desert southwest soils. As with most regional soil conditions, desert southwest soils exhibit trends and behaviors that are not necessarily compatible with practice in other regions. Correspondingly, the subsurface investigation conducted in support of the ABL-PC project was executed according to a procedure that may not be typical of industry practice. Thus, the quality of data and the variability therein, may be different that that derived from other investigations of the same scope. Utilization of soil strength variability parameters from this investigation therefore inextricably link the results of the analysis to the investigation and strata selection techniques applied. However, the trends and levels of variability observed in the dataset are either comparable to or exceed generally accepted values used in practice.

9.3.3 *Load Analysis Compatibility*

The simulation model incorporates assumptions on both the regional variability of maximum annual wind speeds near the Phoenix, AZ metropolitan area and the mechanism through which wind variability translates to foundation load variability. In particular, the angle structure considered in soil Zones 1, 2A and 3C, an overload factor of 1.65 is used to reduce factored foundation loads to the nominal loading condition. As noted, the factor used reflects the NESC prescribed factor on conductor tensions that generally make the largest contribution to moment loading imposed upon foundations supporting large angle single shaft structure. This is a simplifying assumption however, as wind loads simultaneously contribute to foundation loads and incorporate a load factor of 2.5 in accordance with the NESC. Determination of the actual foundation load factor requires a structural analysis beyond the scope of this study, the results of which would produce a load factor somewhere between 1.65 and 2.5. For this reason, the assumption of 1.65 is conservative and the values for P_f and P_e presented are somewhat higher than would be observed in consideration of the 'mixed' tension/wind load factor.

The soil strength factors applied in accordance with Eurocode 7 are representative of reliability calibrations performed under the requirements of Eurocode 1, Actions on Structures. The load regime applied as part of this study is representative of that required in accordance with the NESC, a derivative of ASCE 7. Thus, some incompatibility exists in the load variability considered in the calibrated resistance factors presented by Eurocode and the load variability employed in this study. This difference may contribute to a global shift in the β values computed from the Eurocode design methodology. Although not reflective of the intent of the Eurocode, the results are still valid in the particular application of Eurocode resistance factors to the design of transmission line foundations in the United States where the NESC load regime, and load factors contained therein, are a legislated requirement. For this reason, the average β of 3.0 computed by this study in comparison to the code objective of 3.8 should not be interpreted as commentary on the code's ability to satisfy its established reliability objectives. Instead, the variability of the β values computed across differing soil profile is of interest.

9.3.4 *Foundation Model Compatibility*

The comparisons to existing design methodologies outside of the transmission line industry (FHWA and Eurocode 7) are performed in accordance with the paradigm of short, rigid laterally loaded drilled piers. For this reason, outright application of the design methods indicated by FHWA and Eurocode 7 are not necessarily indicative of the true performance of each approach. This is particularly true in the case of the FHWA approach which is derived on the basis of laterally loaded drilled piers for bridges. Bridge foundations generally exhibit L/B ratios well in excess of those observed in transmission line foundations. Correspondingly, these foundations do not behave as rigid bodies and are inherently incompatible with the MFAD model for this reason.

To the extent inferences on actual reliability can be made from the analysis discussed herein, this is an example of the important role compatibility amongst the RBD calibration and design model plays.

9.3.5 *Analysis Resolution*

The theoretical foundation designs considered in this study are developed in accordance with the industry standard of practice where foundation depth is varied by 1ft increments. Particularly in stiff soil strata, a 1ft increase in foundation depth can contribute significant lateral capacity, yielding inaccuracy in the β values presented. The theoretical foundation designs presented are the minimum foundation size in 1ft increments that satisfy the stated design requirement. Therefore, the computed β values are conservative and the maximum error is that contributed by 0.9ft of additional foundation depth. In consideration of the change in β with depth (Fig. 9.6) the maximum error in the values computed is approximately 0.3. From a practical application perspective, the results presented are representative the variations in reliability which would be desirable on a typical design project.

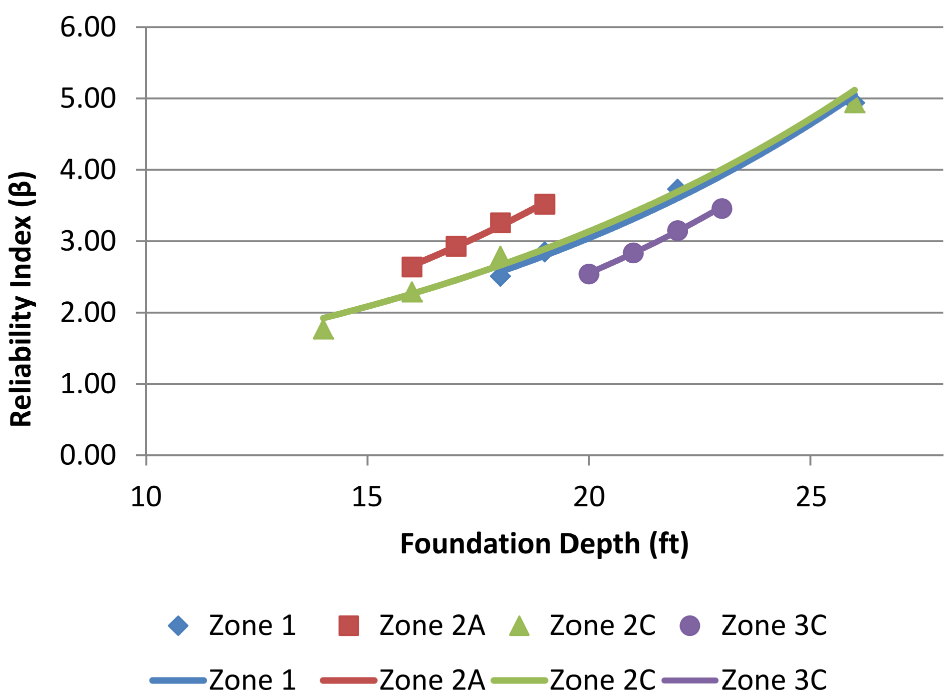


Figure 9.6 - Variation in calculated β with foundation depth

10 CONCLUSIONS

A number of observations are derived from the analysis results presented with regard to the validity of the analysis model in comparison to general reliability theory, areas of interest where current RBD practice does not address factors affecting reliability outcomes and areas requiring further research.

10.1 Model Behavior

The behavior of the analysis model generally agrees with the results of those performing similar analyses (Cao, Wang, & Wang, 2013), (Fan & Liang, 2013). Results derived from the theoretical foundation performance model generally support reasons cited by the geotechnical engineering community to deviate from existing ASD practice in an effort to develop RBD for compatibility with existing practice in the structural engineering community (Section 5). Chief among these reasons are the tendency toward excess conservatism in existing ASD methods and the inability to achieve consistent levels of reliability amongst foundations and the structures they support across variable geotechnical strata and limit states.

As noted by others, in the absence of clarity about the reliability achieved by a particular design method, the standard of practice generally migrates toward a conservative assessment of subsurface conditions (Allen, 2005), (DiGioia Gray and Associates, 2009), (Heim, Kandaris, & Houston, 2011). The use of low bound soil strength parameters in the absence of a defined framework to estimate nominal soil properties is an important consideration in this regard. Correspondingly, the theoretical model results show that selection of low bound strength parameters not only yields foundation designs well in excess of established thresholds for reliability, but also provides highly variable reliability results (Section 8.2.3.4). At the high levels of reliability observed in the model, concern over consistency is overridden by that of economy (Phoon, 2004).

Of the code documents examined, the reliability performance differs from that observed with the use of lower bound strength parameters. In general, the results yield fairly consistent levels of computed reliability with the exception of the foundations developed for Zone 2C. However, in all but one case, including Zone 2C, the transmission industry reliability objective of $\beta = 2.3$ is theoretically satisfied. As an industry specific document (DiGioia Gray and Associates, 2012) generally achieves results most in line with the reliability goals of the transmission line industry. Eurocode, generally yields more conservative results and is a reflection of its broader industry application and calibration on a different load regime.

The alternative approach utilizing soil strength parameters derived from site specific variability achieves somewhat more conservative results, but also suffers less degradation in performance within Zone 2C. Analytically, the results support a site-specific design approach to address the variability encountered in practice that factored design methodologies cannot easily accommodate.

10.2 Potential Enhancements to Existing RBD Practice

In practice, the design of high eccentricity short laterally loaded drilled pier foundations commonly used in the transmission industry is generally governed by deflection limit criteria. However, the reliability based code documents presented are calibrated on the basis of ultimate capacity design. Thus, the reliability based component of the design process is performed as a limit state check and design for the controlling limit state is carried out in the absence of a code based reliability assessment. Therefore, an inherent disconnect exists between the available RBD approaches and their practical application in the transmission line industry. To this point, it is desirable to foster a discussion which will identify the desired level of reliability when service limits govern design and from that, a framework which identifies the methods to insure the specified level of reliability is achieved (e.g. definition of nominal soil properties, specification of an appropriate design model and appropriate resistance factors for use in service limit design).

The anomalous performance of Zone 2C foundations across all design approaches highlights the role that stratification and variation therein play in the reliability of laterally loaded drilled shaft foundations. Existing RBD codes employ resistance factors derived from either empirical calibrations on load test data (DiGioia Gray and Associates, 2012), assessments on the inherent variability/uncertainty of individual soil strength parameters (British Standards, 2004), (Phoon, Kulhaway, & Grigoriu, 1995) or factors derived from engineering judgment in the absence of supporting reliability analyses (Brown, Turner, & Castelli, 2010). For those methods employing reliability analyses, the FORM algorithm is applied to derive resistance factors, a method which is incapable of addressing spatial variability.

In general, the results show the resistance factors derived from FORM calibration can yield acceptable reliability performance. The performance of foundations in Zone 2C and similar observations by others is, however, representative of special cases where the reliability achieved by RBD methods is subject to unacceptable performance (Cao, Wang, & Wang, 2013) (Fan & Liang, 2013). In the special case of Zone 2C, observation of the load-deflection characteristics of the theoretical foundation designs shows the potential for suboptimal performance. In the absence of theoretical performance model results, standard analyses performed during design are therefore subject to anomalous performance under the conditions represented by Zone 2C. For this reason, the findings of this analysis indicate, preliminarily, that existing RBD are suitable for commonly encountered soil conditions. However, there are special cases where reliability may be compromised by spatial variability of soil strength. Design practice employing RBD methodologies should include an assessment of the load/deflection performance of perspective foundations to verify performance requirements will be satisfied. Toward this goal, analytical methods to objectively evaluate load/deflection performance are required to insure standard practice in this area is in harmony with the objectives of RBD.

On a conceptual basis, a graphical method for analysis of load-deflection performance is proposed (Fig. 10.1). The proposed assessment of load-deflection performance provides designers with a mechanism to identify and address the anomalous condition encountered in Zone 2C. Further analysis is required to identify the appropriate value for ψ to achieve consistent reliability, however a basic design equation format is proposed for compatibility with existing single factor RBD equations (Eq. 10.2.1).

$$\gamma_D \leq \psi \, \gamma_L \qquad \text{Eq. 10.2.1}$$

Where:

γ_D = Design deflection limit

γ_D = Elastic deflection limit

ψ = Deformation factor – Requires further study

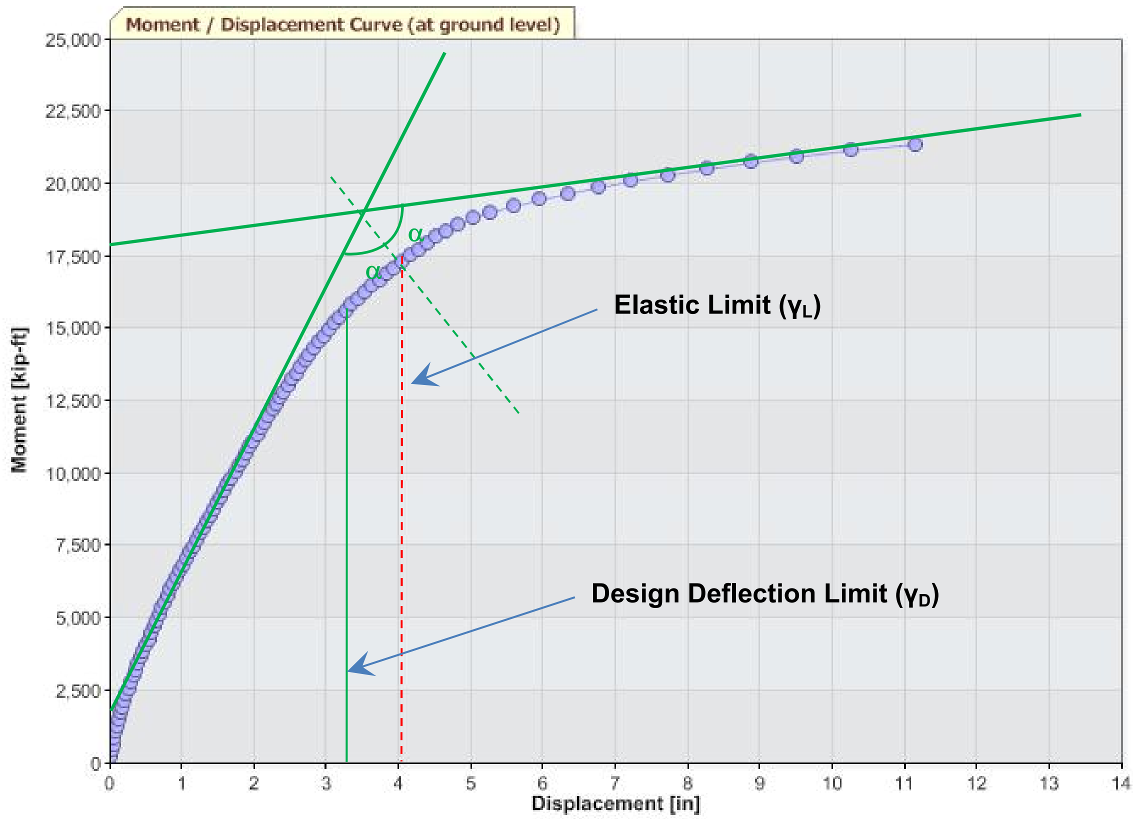


Figure 10.1 - Conceptual Load/Deflection limit design check

10.3 Further Research

The study presented results from a single project database in a specific geotechnical region and is therefore limited in its applicability on a broader scale. Similarly, the analytical methods incorporated in the model are subject to further improvement to provide enhanced understanding on the implications stratification and RBD practice at large has on laterally loaded pier performance. A number of avenues for further research and enhancements to the analytical model include:

Research Enhancements:

- Expand the analysis database to include diversified soil strata beyond that of the ABL-PC project
- Develop a series of prescribed soil profiles with defined disparities in soil strength over pier depth to further evaluate reliability implications illustrated by reliability performance of foundations in Zone 2C.
- Expand analyses to include additional load/foundation configurations.
- Employ results from expanded analyses to derived analytical methods for evaluating foundation load/deflection performance in view of RBD objectives.
- Perform a sensitivity analysis on the relationship between β and foundation depth to establish the level of reliability at which foundations cost begin to increase disproportionality.

Model Enhancements:

- Expand the model capabilities to consider variability in layer depth.
- Consider strength parameter variability protocols that permit currently linked strength parameters to vary in accordance with differing probability density functions.
- Improve computation schema to decrease computing time.

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APPENDIX A

PC-ABL BLOW COUNT DATA

Zone 1 Blow Count Data Analysis

SOIL TYPE: Varies

Dry Unit Weight = 105

(Cohesive = 3, Cohesless = 4) =

(Sample Type: "S" = Spoon, "R" = Ring)

Bor No.	Depth (ft)	Type	Blows	Length (in)	Blows/Ft Mea	SPT-C	SPT-G	Used	Depth	Rel Dens %	Mat Class	Zone No.	ln(used)
4	7.0	R	9	12.0	9	8	5	5	10	43	SP-SM	1	1.60
2	2.0	R	8	12.0	8	7	4	7	12	55	SC-SM	1	1.95
7	7.0	R	8	12.0	8	7	4	7	9	48	SC	1	1.95
4	5.0	R	9	12.0	9	8	5	8	11	54	SM	1	2.08
8	10.0	R	10	12.0	10	8	6	8	10	42	SC-SM	1	2.08
4	2.0	R	10	12.0	10	8	6	8	15	59	CL	1	2.13
8	7.0	R	12	12.0	12	10	7	10	14	56	SC-SM	1	2.30
9	5.0	R	12	12.0	12	10	7	10	15	59	SC-SM	1	2.30
5	10.0	R	12	12.0	12	10	7	10	12	51	CL-CH	1	2.31
8	5.0	R	13	12.0	13	11	7	11	16	62	SM	1	2.40
9	7.0	R	13	12.0	13	11	7	11	15	58	SC-SM	1	2.40
4	10.0	R	16	12.0	16	13	9	13	16	60	SP-SM	1	2.56
3	2.0	R	25	12.0	25	21	14	14	37	77	CL	1	2.64
2	10.0	R	26	12.0	26	22	14	14	27	64	SP-SM	1	2.66
5	5.0	R	17	12.0	17	14	9	14	21	71	CL	1	2.66
7	10.0	R	17	12.0	17	14	9	14	17	64	ML	1	2.66
9	2.0	R	18	12.0	18	15	10	15	27	81	CL	1	2.72
5	2.0	R	19	12.0	19	16	10	16	28	83	SC-CL	1	2.77
1	2.0	R	19	12.0	19	16	10	16	28	83	SC	1	2.77
5	7.0	R	21	12.0	21	18	12	18	24	75	CL	1	2.87
1	5.0	R	21	12.0	21	18	12	18	26	81	SC	1	2.89
6	5.0	R	22	12.0	22	19	12	19	27	83	CL	1	2.92
8	2.0	R	22	12.0	22	19	12	19	33	91	CL	1	2.92
7	2.0	R	24	12.0	24	20	13	20	35	97	CL	1	3.01
7	5.0	R	28	12.0	28	20	15	20	30	88	SC-CL	1	3.02
1	7.0	R	25	12.0	25	21	14	21	28	83	CH	1	3.05
6	2.0	R	25	12.0	25	21	14	21	37	99	CL	1	3.05
6	10.0	R	30	12.0	30	22	17	22	27	80	CL-CH	1	3.09
3	10.0	R	31	12.0	31	23	17	23	28	81	CL	1	3.12
3	7.0	R	23	12.0	23	19	13	23	26	88	CL-ML	1	3.14
1	10.0	R	33	12.0	33	24	18	24	29	83	SC-SM	1	3.18
6	7.0	R	34	12.0	34	25	19	25	33	93	CL-CH	1	3.21
3	5.0	R	39	12.0	39	28	21	28	41	100	CL-ML	1	3.35
9	10.0	R	39	12.0	39	28	21	28	35	90	CL-CH	1	3.35

No. =1

Normal Dist Stats	
Mean	15.9
Std. Dev.	6.4
x	0.4
v	0.0
sample size	34
Variance	40.405
COV	40%

Beta Dist. Stats		Easy Fit	Pham-Gia
Min	4.4	4.4	4.4
Max	35.0	35.0	35.0
Alpha	1.7	1.7	1.7
Beta	2.8	2.8	2.8

Log Dist Stats	
Mean of ln(used)	2.68
Std. Dev. of ln(used)	0.45
Mean	14.5695
Std. Dev.	1.5613

					Mea	SPT-C	SPT-G	Used	Depth	Dens %	Class	No.	ln(used)
1	15.0	R	27	12.0	27	23	15	23	25	79	SC-SM	1	3.14
2	15.0	R	36	12.0	36	26	20	26	28	84	CL-CH	1	3.27
3	15.0	R	35	12.0	35	26	19	26	28	83	CL	1	3.24
4	15.0	R	50	9.0	67	49	37	49	53	100	CL-CH	1	3.88
5	15.0	R	21	12.0	21	18	12	18	19	70	SC-SM	1	2.89
6	15.0	R	42	12.0	42	31	23	31	33	90	CL	1	3.42
7	15.0	R	41	12.0	41	30	23	30	32	89	CL	1	3.40
8	15.0	R	26	12.0	26	22	14	22	24	77	CL-CH	1	3.09
9	15.0	R	50	11.0	54.545	40	30	40	43	99	CL	1	3.68
1	20.0	R	35	12.0	35	26	19	19	25	70	SM	1	2.96
2	20.0	R	26	12.0	26	22	14	22	22	75	CL-CH	1	3.09
3	20.0	R	39	12.0	39	28	21	28	28	84	CL-ML	1	3.35
4	20.0	R	34	12.0	34	25	19	25	24	80	SC	1	3.22
5	20.0	R	29	12.0	29	21	16	21	21	73	SC-SM	1	3.04
6	20.0	R	24	12.0	24	20	13	20	20	72	CL-ML	1	3.01
7	20.0	R	27	12.0	27	23	15	23	22	76	ML	1	3.12
8	20.0	R	17	12.0	17	14	9	14	14	60	CL-ML	1	2.66
9	20.0	R	30	12.0	30	22	17	22	22	75	CL	1	3.09
1	25.0	R	25	12.0	25	21	14	21	19	71	SC	1	3.04
2	25.0	R	36	12.0	36	26	20	20	24	69	SP-SM	1	2.99
3	25.0	R	56	12.0	56	41	31	41	37	94	CL	1	3.71
4	25.0	R	34	12.0	34	25	19	25	23	77	SC-SM	1	3.22
5	25.0	R	42	12.0	42	31	23	23	28	74	SM	1	3.14
6	25.0	R	27	12.0	27	23	15	23	21	74	CL	1	3.12
7	25.0	R	50	12.0	50	36	28	36	33	90	SC	1	3.58
8	25.0	R	48	12.0	48	35	26	35	32	89	CL-CH	1	3.56
9	25.0	R	41	12.0	41	30	23	30	27	83	CL	1	3.40
1	30.0	R	39	12.0	39	28	21	21	24	69	SC	1	3.07
2	30.0	R	23	12.0	23	19	13	19	16	66	CL-ML	1	2.96
3	30.0	R	66	12.0	66	48	36	48	41	97	CL	1	3.87
4	30.0	S	26	12.0	26	26	26	26	22	76	SC-SM	1	3.26
5	30.0	R	45	12.0	45	33	25	33	28	84	CL-CH	1	3.49
6	30.0	R	31	12.0	31	23	17	23	19	71	CL	1	3.12
7	30.0	R	41	12.0	41	30	23	30	25	81	CH	1	3.40
8	30.0	R	36	12.0	36	26	20	26	22	76	CL-CH	1	3.27
9	30.0	R	39	12.0	39	28	21	21	24	69	CL	1	3.04

No. =1

Normal Dist Stats	
Mean	26.6
Std. Dev.	8.0
x	0.5
v	0.0
sample size	36
Variance	64.438
COV	30%

Beta Dist. Stats		Easy Fit	Pham-Gia
Min	2.6	2.6	2.6
Max	50.7	50.7	50.7
Alpha	4.0	3.4	4.0
Beta	4.0	3.2	4.0

Log Dist Stats	
Mean of ln(used)	3.2488
Std. Dev. of ln(used)	0.2809

Zone 2A Blow Count Data Analysis

SOIL TYPE: Varies

Dry Unit Weight = 105

(Cohesive = 3, Cohesless = 4) =

(Sample Type: "S" = Spoon, "R" = Ring)

Legend: consol; new DS; old DS; PI/-#200

No. =2A

Bor No.	Depth (ft)	Type	Blows	Length (in)	Blows/Ft Mea	SPT-C	SPT-G	Used	Depth	Rel Dens %	Mat Class	Zone No.
22	2.0	R	9	12.0	9	8	5	8	13	57	CL	2A
10	2.0	R	10	12.0	10	8	6	8	15	59	CL-CH	2A
23	2.0	R	11	12.0	11	9	6	9	16	62	CL	2A
11	2.0	R	14	12.0	14	12	8	12	21	70	CL	2A
DA15	2.0	R	25	12.0	25	21	14	14	37	76	SC	2A
21	2.0	R	18	12.0	18	15	10	15	27	81	CL	2A
20	2.0	R	20	12.0	20	17	11	17	30	86	CL	2A
24	2.0	R	26	12.0	26	22	14	22	38	100	CL	2A
22	5.0	R	12	12.0	12	10	7	10	15	59	CL	2A
10	5.0	R	17	12.0	17	14	9	14	21	71	CL-CH	2A
23	5.0	R	20	12.0	20	17	11	17	24	78	CL	2A
24	5.0	R	26	12.0	26	22	14	22	32	92	CL	2A
11	5.0	R	32	12.0	32	23	18	23	34	96	CL-CH	2A
								Used	Depth	Dens %	Class	No.
DA15	5.0	R	52	12.0	52	38	29	38	55	100	CL-SC	2A
23	7.0	R	25	12.0	25	21	14	21	28	83	CL	2A
10	7.0	R	39	12.0	39	28	21	28	38	100	CL	2A
22	7.0	R	41	12.0	41	30	23	30	40	100	CL	2A
11	7.0	R	54	12.0	54	39	30	39	53	100	CL-CH	2A
DA15	7.0	R	50	11.0	55	40	30	40	53	100	SC-C	2A
24	7.5	R	38	12.0	38	28	21	28	36	98	CL-CH	2A
23	10.0	R	35	12.0	35	26	19	26	31	86	CL	2A
24	10.0	R	40	12.0	40	29	22	29	35	91	CL-CH	2A
10	10.0	R	55	12.0	55	40	30	40	49	100	CL	2A
22	10.0	R	50	10.5	57	42	31	42	51	100	CL-CH	2A
DA15	10.0	R	50	10.0	60	44	33	44	53	100	SC-C	2A
11	10.0	R	50	8.5	71	51	39	51	63	100	CL-CH	2A
								Used	Depth	Dens %	Class	No.
23	15.0	R	56	12.0	56	41	31	41	44	100	CL	2A
24	15.0	R	53	11.0	58	42	32	42	46	100	CL-CH	2A
22	15.0	R	50	10.0	60	44	33	44	47	100	CL-CH	2A
11	15.0	R	50	10.0	60	44	33	44	47	100	CL-CH	2A
DA15	15.0	R	50	9.5	63	46	35	46	50	100	SC-C	2A
24	20.0	R	50	11.0	55	40	30	40	39	97	CL	2A
DA15	20.0	R	50	10.0	60	44	33	44	43	100	SC-C	2A
23	20.0	R	47	8.0	71	51	39	51	51	100	CL-CH	2A
11	20.0	R	50	8.0	75	55	41	55	54	100	CL-CH	2A
22	20.0	R	50	6.5	92	67	51	67	66	100	CL-CH	2A
10	20.0	R	50	6.0	100	73	55	73	72	100	CL-CH	2A
10	25.0	R	21	12.0	21	18	12	18	16	66	SC	2A
24	25.0	R	42	12.0	42	31	23	31	28	84	CL	2A
11	25.0	R	46	12.0	46	34	25	34	31	88	SC	2A
22	25.0	R	50	12.0	50	36	28	36	33	90	SC	2A
23	25.0	R	54	11.0	59	43	32	43	39	96	CL-CH	2A
DA15	25.0	R	50	8.0	75	55	41	55	50	100	SC-C	2A
23	30.0	R	54	11.0	59	43	32	43	36	93	CL-CH	2A
24	30.0	R	50	9.0	67	49	37	49	41	97	CL-CH	2A
22	30.0	R	50	8.5	71	51	39	51	44	99	SC	2A
DA15	30.0	R	50	7.0	86	62	47	62	53	100	SC-C	2A
11	30.0	R	50	6.0	100	73	55	73	62	100	SC	2A

Normal Dist Stats			
Mean	14.7		
Std. Dev.	5.3		
x	0.4		
v	0.0		
Sample Size	13		
Variance	28.13720974		
Beta Dist. Stats		Easy Fit	Pham-Gia
Min	5.0	5.0	5.0
Max	30.6	30.6	30.6
Alpha	1.7	2.0	1.7
Beta	2.8	3.4	2.8

Normal Dist Stats		
Mean	35.1	
Std. Dev.	8.7	
x	0.5	
v	0.03	
Sample Size	13	
Variance	75.74452387	
Beta Dist. Stats		Easy Fit
Min	8.9	8.9
Max	61.2	61.2
Alpha	4.0	4.5
Beta	4.0	4.5

Normal Dist Stats		
Mean	47.3	
Std. Dev.	13.3	
x	0.5	
v	0.03	
Sample Size	22	
Variance	177.5109018	
Beta Dist. Stats		Easy Fit
Min	7.3	7.3
Max	87.3	87.3
Alpha	4.0	4.1
Beta	4.0	4.0

Zone 2C Blow Count Data Analysis

SOIL TYPE: Varies
Dry Unit Weight = 105
(Cohesive = 3, Cohesless = 4) = 4
(Sample Type: "S" = Spoon, "R" = Ring, "M" = Modified California)

Bor No.	Depth (ft)	Type	Blows	Length (in)	Blows/Ft Mea	SPT-C	SPT-G	Used	Depth	Rel Dens %	Mat Class	Soil No.
20	2.0	R	20	12.0	20	17	11	17	30	86	CL	2C
21	2.0	R	18	12.0	18	15	10	15	27	81	CL	2C
DA12	2.0	R	18	12.0	18	15	10	15	27	80	SC	2C
DA13	2.0	R	40	12.0	40	29	22	29	51	100	CL	2C
21	5.0	R	36	12.0	36	26	20	20	38	86	SP-SC	2C
DA12	5.0	R	11	12.0	11	9	6	9	13	56	SC	2C
DA13	5.0	R	39	12.0	39	28	21	28	41	100	CL-C	2C
20	7.0	R	18	12.0	18	15	10	15	20	69	CL-ML	2C
21	7.0	R	34	12.0	34	25	19	19	33	78	GP-SP	2C
DA12	7.5	R	23	12.0	23	19	12	12	25	60	SM/SP	2C
DA13	7.5	R	38	12.0	38	28	21	28	36	99	SC-C	2C
20	10.0	R	24	12.0	24	20	13	13	25	61	GP-SP	2C
21	10.0	R	47	12.0	47	34	26	26	42	86	SP-SC	2C
DA12	10.0	R	34	12.0	34	25	18	25	30	85	CL-ML	2C
DA13	10.0	R	33	12.0	33	24	18	24	29	83	SC-C	2C
DA11	2.0	R	21	12.0	21	18	11	18	31	90	SC	3A
DA10	5.0	R	33	12.0	33	24	18	24	35	98	SC	3A
DA10	7.5	R	50	12.0	50	36	27	36	48	100	SC	3A
DA11	7.5	R	18	12.0	18	15	10	10	20	55	GP	3A
DA10	10.0	S	33	12.0	33	33	33	33	40	95	SC	3A
DA11	10.0	S	19	12.0	19	19	19	19	23	74	GP	3A
DA11	5.0	R	50	4.5				97		100	SC	3A
DA10	2.0	R	50	8.0				55		100	SC	3A
					Mea	SPT-C	SPT-G	Used	Depth	Dens %	Class	No.
20	15.0	R	42	12.0	42	31	23	23	33	79	GP-SP	2C
21	15.0	R	50	11.5	52	38	29	29	41	88	GP-SP	2C
DA12	15.0	R	50	11.5	52	38	28	38	41	98	SC	2C
DA13	15.0	R	31	12.0	31	23	17	23	24	79	CL/SC-C	2C
20	20.0	R	43	12.0	43	31	24	24	31	78	GP-SP	2C
21	20.0	R	50	10.5	57	42	31	42	41	98	CL-CH	2C
DA13	20.0	S	19	12.0	19	19	19	19	19	70	CL	2C
20	25.0	R	34	12.0	34	25	19	19	23	67	GP-SP	2C
21	25.0	R	50	6.5	92	67	51	67	61	100	CL-CH	2C
DA12	25.0	S	29	12.0	29	29	29	29	26	82	SP/GP	2C
DA13	25.0	R	50	10.0	60	44	33	44	40	97	CL/SC	2C
20	30.0	R	24	12.0	24	20	13	20	17	67	ML	2C
21	30.0	R	50	8.0	75	55	41	55	46	100	CL	2C
DA12	30.0	S	50	12.0	50	50	50	50	42	98	GP	2C
DA13	30.0	R	33	12.0	33	24	18	24	20	73	CL	2C
DA12	20.0	R	50	4.5				97		100	SC	2C
Other categories - 2C & 3A Upper Layer												
Other categories - 2C Lower Layer												

No.
=2C

Normal Dist Stats			
Mean	20.7		
Std. Dev.	7.5		
x	0.4		
v	0.0		
Sample Size	21		
Variance	42.65967989		
Beta Dist. Stats		Easy Fit	Pham-Gia
Min	6.0	6.0	6.0
Max	43.3	43.3	43.3
Alpha	2.7	2.1	2.7
Beta	4.1	3.3	4.1

No.
=2C

Normal Dist Stats			
Mean	33.7		
Std. Dev.	14.9		
x	0.3		
v	0.1		
Variance	221.7254286		
Sample Size	15		
Beta Dist. Stats		Easy Fit	Pham-Gia
Min	13.2	13.2	13.2
Max	78.4	78.4	78.4
Alpha	1.0	1.4	1.0
Beta	2.1	2.9	2.1

Zone 3C Blow Count Data Analysis

SOIL TYPE: Varies
Dry Unit Weight = 105
(Cohesive = 3, Cohesless = 4) = 4
(Sample Type: "S" = Spoon, "R" = Ring, "M" = Modified California)

Bor No.	Depth (ft)	Type	Blows	Length (in)	Blows/Ft Mea	SPT-C	SPT-G	Used	Depth	Rel Dens %	Mat Class	Soil No.
DA3	2.0	R	13	12.0	13	11	7	7	19	55	SC/SP	3C
DA7	2.0	S	10	12.0	10	10	10	10	18	64	SC	3C
DA2	5.0	R	13	12.0	13	11	7	11	16	62	CL	3C
DA3	5.0	R	13	12.0	13	11	7	11	16	62	CL	3C
DA1	5.0	R	18	12.0	18	15	10	15	22	73	SC-C	3C
DA4	2.0	R	20	12.0	20	17	11	17	30	87	SC	3C
DA2	2.0	R	29	12.0	29	21	16	21	37	99	CL	3C
DA6	5.0	S	22	12.0	22	22	22	22	32	92	GC/GP	3C
DA1	2.0	R	31	12.0	31	23	17	23	40	100	SC-C	3C
DA6	2.0	S	30	12.0	30	30	30	30	53	100	SC	3C
DA7	5.0	S	36	12.0	36	36	36	36	52	100	SC	3C

No. =3c

Normal Dist Stats		
Mean	18.5	
Std. Dev.	9.0	
x	0.3	
v	0.1	
Sample Size	11	
Variance	80.87272727	
Beta Dist. Stats		Easy Fit
Min	7.0	7.0
Max	45.4	45.4
Alpha	0.8	1.5
Beta	2.0	3.0

Bor No.	Depth (ft)	Type	Blows	Length (in)	Blows/Ft Mea	SPT-C	SPT-G	Used	Depth	Dens %	Class	No.
DA3	10.0	R	31	12.0	31	23	17	17	28	70	GC/GP	3C
DA2	7.5	R	21	12.0	21	18	11	18	23	74	SC	3C
DA4	7.5	S	29	12.0	29	29	29	29	38	100	SC/SP	3C
DA4	10.0	S	29	12.0	29	29	29	29	35	90	GC/GP	3C
DA2	10.0	R	50	11.0	55	40	30	30	48	92	GC/GP	3C
DA2	15.0	R	50	10.5	57	42	31	31	45	90	GC/GP	3C
DA1	7.5	R	50	9.0	67	49	36	36	64	100	SC/SP	3C
DA3	15.0	S	37	12.0	37	37	37	37	40	97	SC/SP	3C
DA3	7.5	R	50	8.0	75	55	41	55	72	100	SC-C	3C
DA1	10.0	R	50	5.0	120	87	65	65	106	100	SC/SP	3C

No. =3c

Normal Dist Stats		Lognormal
Mean	34.7	3.5
Std. Dev.	15.0	0.4
x	0.3	
v	0.0	
Sample Size	10	
Variance	225.5666667	
Beta Dist. Stats		Easy Fit
Min	11.4	11.4
Max	79.8	79.8
Alpha	1.2	1.6
Beta	2.4	3.1

Bor No.	Depth (ft)	Type	Blows	Length (in)	Blows/Ft Mea	SPT-C	SPT-G	Used	Depth	Dens %	Class	No.
DA2	30.0	S	43	12.0	43	43	43	43	37	93	GC/GP	3C
DA2	20.0	R	50	7.0	86	62	47	47	62	100	GC/GP	3C
DA1	25.0	S	50	12.0	50	50	50	50	46	100	GC/GP	3C
DA2	25.0	S	50	12.0	50	50	50	50	46	100	GC/GP	3C
DA1	20.0	S	71	12.0	71	71	71	71	70	100	SC	3C
DA3	20.0	S	50	5.5	109	109	109	109	108	100	GC/GP	3C
DA4	20.0	S	50	5.5	109	109	109	109	108	100	SC	3C
DA4	25.0	S	50	5.5	109	109	109	109	99	100	GC/GP	3C

No. =3c

Normal Dist Stats		
Mean	73.5	
Std. Dev.	30.5	
x	0.2	
v	0.1	
Sample Size	8	
Variance	932	
Beta Dist. Stats		Easy Fit
Min	43.0	43.0
Max	165.1	165.1
Alpha	0.5	0.9
Beta	1.5	2.3

APPENDIX B

MFAD CALCULATIONS

Zone 1 – 20ft – Homogeneous 5% LEL Mean

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Zone 1 (Homogeneous)**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**2/5/2013 12:00:00 AM**		
Modified Date:	**2/26/2014 12:44:54 AM**		
Comments:	**homogeneous using uper layer of Zone 1**		

STRUCTURE

Structure ID: **Zone 1 homo .5 lel mean**
Description:

CASE-DRILLED SHAFT

Case Name: **zone 1 homo .05 lel mean**
Description: **zone 1 homo .05 lel mean**

Foundation Data (5DCA30-145-2)
Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (Zone 1 Layer 1 Homo .05 lel mean)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	50	110	2.05	30.3	1.03	0

Applied Loads-Top of Shaft (dca30)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	dca30	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **20**
Controlling Applied Load Case Name: **dca30**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	208.2	131.2	1.1
Moment [kip-ft]	12588.8	22646.6	14267.3	1.1

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	2.8
Total Rotation [deg.]	1	1.0
Nonrecoverable Displacement [in]	1.92	0.9
Nonrecoverable Rotation [deg.]	0.5	0.3

	Maximum Value	Depth of Occurance
Shear:	761.9 kips	12.1ft
Moment:	12827.2 kips-ft	0.4 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	2.9	117.9	12588.8	0.0
1	2.7	117.9	12706.7	0.0
Ground Level (0)	2.5	117.9	12824.6	0.0
1	2.2	50.3	12802.1	9.4
2	2.0	-34.1	12681.5	11.5
3	1.8	-135.0	12446.8	13.5
4	1.6	-249.0	12083.5	14.1
5	1.4	-354.9	11604.4	12.6
6	1.2	-449.1	11025.2	11.1
7	1.0	-531.5	10357.7	9.6
8	0.8	-602.1	9613.7	8.2
9	0.6	-660.8	8805.1	6.7
10	0.4	-707.8	7943.7	5.2
11	0.2	-742.9	7041.2	3.7
12	0.0	-761.9	6110.3	0.5
13	-0.2	-745.2	5177.8	-3.6
14	-0.4	-708.5	4273.6	-5.4
15	-0.6	-658.0	3413.1	-7.1
16	-0.8	-593.6	2610.0	-8.8
17	-1.0	-515.3	1878.2	-10.6
18	-1.2	-423.3	1231.5	-12.3
19	-1.4	-317.4	683.9	-14.0
20	-1.6	-197.7	249.0	-15.7
20	-1.6	0.0	0.0	-15.7

Detailed Message:

Zone 1 – 31ft – Homogeneous 10% LEL Beta

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Zone 1 (Homogeneous)**	**Checked By:**	**___________________**
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	**___________________**
Start Date:	**2/5/2013 9:04:17 PM**		
Modified Date:	**2/5/2013 9:22:25 PM**		
Comments:	**homogeneous using uper layer of Zone 1**		

STRUCTURE

Structure ID: **.1 beta**
Description:

CASE-DRILLED SHAFT

Case Name: **.1 beta**
Description: **zone 1 homogeneous**

Foundation Data (5DCA30-145-2)
Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (PC-AB Zone 1 Homogeneous (.1 Beta)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	100	110	0.77	25.8	0.6	0

Applied Loads-Top of Shaft (STR-77 5DCA30-145-2)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5DCA30-145-2	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **31**
Controlling Applied Load Case Name: **5DCA30-145-2**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	296.9	187.1	1.6
Moment [kip-ft]	12588.8	32297.2	20347.2	1.6

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	3.6
Total Rotation [deg.]	1	0.8
Nonrecoverable Displacement [in]	1.92	0.8
Nonrecoverable Rotation [deg.]	0.5	0.2

	Maximum Value	Depth of Occurance
Shear:	586.2 kips	19.5ft
Moment:	12887.1 kips-ft	1.8 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	3.6	117.9	12588.8	0.0
1	3.4	117.9	12706.7	0.0
Ground Level (0)	3.3	117.9	12824.6	0.0
1	3.1	85.0	12875.6	4.6
2	2.9	43.1	12885.6	5.7
3	2.8	-7.6	12849.4	6.8
4	2.6	-64.4	12758.7	7.0
5	2.4	-119.1	12611.9	6.7
6	2.3	-171.0	12411.9	6.3
7	2.1	-220.1	12161.4	6.0
8	1.9	-266.4	11863.2	5.6
9	1.8	-309.9	11520.0	5.3
10	1.6	-350.7	11134.7	4.9
11	1.4	-388.6	10710.1	4.6
12	1.3	-423.7	10249.0	4.2
13	1.1	-456.0	9754.1	3.9
14	0.9	-485.5	9228.3	3.5
15	0.7	-512.3	8674.4	3.2
16	0.6	-536.2	8095.2	2.8
17	0.4	-557.3	7493.5	2.5
18	0.2	-575.4	6872.1	1.9
19	0.1	-585.1	6236.2	0.6
20	-0.1	-584.6	5595.7	-0.7
21	-0.3	-573.7	4960.9	-1.9
22	-0.4	-553.6	4342.0	-2.8
23	-0.6	-529.0	3745.6	-3.3
24	-0.8	-500.5	3175.7	-3.8
25	-0.9	-468.1	2636.3	-4.3
26	-1.1	-431.8	2131.3	-4.8
27	-1.3	-391.6	1664.5	-5.2
28	-1.4	-347.5	1239.8	-5.7
29	-1.6	-299.4	861.3	-6.2
30	-1.8	-247.5	532.8	-6.7
31	-1.9	-191.6	258.1	-7.2
31	-1.9	0.0	0.0	-7.2

Detailed Message:

Zone 1 – 18ft Pier – EPRI, FHWA

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name: Thesis
Responsible Engineer: Not Assigned
Directory: C:\Program Files (x86)\FAD Tools
Start Date: 6/9/2012 12:00:00 AM
Modified Date: 3/2/2013 5:19:09 AM
Comments:

Checked By: ____________________
Date: ____________________

STRUCTURE

Structure ID: STR-77 5DCA
Description:

CASE-DRILLED SHAFT

Case Name: FHWA NORMAL MEAN
Description: MEAN VALUE SOIL PARAMETERS

Foundation Data (5DCA30-145-2)
Diameter of Drilled Shaft: [ft] 8
Stick up above Ground Level: [ft] 2

Model Options
Side Shear Spring: On
Base Shear Spring: On
Base Moment Spring: On

Geotechnical Parameters (DIN-ABL 1 NORMAL Mean stripped)
Depth to Ground Water: [ft] 100

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	12.5	110	2.32	31.5	1.16	0
2	Soil	40	115	3.68	31.9	1.9	0

Applied Loads-Top of Shaft (STR-77 5DCA30-145-2)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5DCA30-145-2	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **18**
Controlling Applied Load Case Name: **5DCA30-145-2**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	224.2	141.2	1.2
Moment [kip-ft]	12588.8	24387.0	15363.8	1.2

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	2.4
Total Rotation [deg.]	1	0.8
Nonrecoverable Displacement [in]	1.92	0.7
Nonrecoverable Rotation [deg.]	0.5	0.3

	Maximum Value	Depth of Occurance
Shear:	824.1 kips	12.0ft
Moment:	12824.6 kips-ft	0.1 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	2.5	117.9	12588.8	0.0
1	2.3	117.9	12706.7	0.0
Ground Level (0)	2.1	117.9	12824.6	0.0
1	1.9	37.4	12773.1	11.2
2	1.7	-62.8	12604.2	13.6
3	1.6	-182.2	12310.4	15.8
4	1.4	-302.1	11894.4	14.3
5	1.2	-410.2	11364.4	12.8
6	1.0	-506.4	10732.2	11.4
7	0.9	-590.6	10009.8	9.9
8	0.7	-662.9	9209.2	8.4
9	0.5	-723.3	8342.2	6.9
10	0.3	-771.7	7420.8	5.4
11	0.2	-808.1	6457.1	3.9
12	0.0	-823.9	5465.1	-0.2
13	-0.2	-793.1	4428.1	-6.6
14	-0.4	-727.9	3391.5	-9.4
15	-0.5	-640.2	2431.4	-12.2
16	-0.7	-530.1	1570.1	-15.0
17	-0.9	-397.5	830.2	-17.8
18	-1.1	-242.6	234.1	-20.6
18	-1.1	0.0	0.0	-20.6

Detailed Message:

Zone 1 – 22ft EPRI/FHWA Service

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Thesis**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**6/9/2012 12:00:00 AM**		
Modified Date:	**2/24/2014 9:48:31 PM**		
Comments:			

STRUCTURE

Structure ID: **STR-77 5DCA**
Description:

CASE-DRILLED SHAFT

Case Name: **EPRI NORMAL MEAN**
Description: **MEAN VALUE SOIL PARAMETERS**

Foundation Data (5DCA30-145-2)
Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (DIN-ABL 1 NORMAL Mean)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	12.5	110	2.32	31.5	1.16	0
2	Soil	40	115	3.68	31.9	1.9	0

Applied Loads-Top of Shaft (STR-77 5DCA30-145-2)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5DCA30-145-2	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **22**
Controlling Applied Load Case Name: **5DCA30-145-2**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	362.8	228.6	1.9
Moment [kip-ft]	12588.8	39462.1	24861.1	2.0

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	1.5
Total Rotation [deg.]	1	0.4
Nonrecoverable Displacement [in]	1.92	0.3
Nonrecoverable Rotation [deg.]	0.5	0.1

	Maximum Value	Depth of Occurance
Shear:	765.6 kips	15.0ft
Moment:	12830.9 kips-ft	0.5 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	1.5	117.9	12588.8	0.0
1	1.4	117.9	12706.7	0.0
Ground Level (0)	1.3	117.9	12824.6	0.0
1	1.2	37.1	12816.2	11.2
2	1.1	-51.7	12721.0	10.8
3	1.1	-135.8	12539.3	10.2
4	1.0	-214.9	12276.0	9.6
5	0.9	-289.2	11936.0	9.0
6	0.8	-358.6	11524.1	8.4
7	0.7	-423.1	11045.2	7.8
8	0.6	-482.7	10504.4	7.2
9	0.5	-537.3	9906.4	6.5
10	0.4	-587.0	9256.3	5.9
11	0.3	-631.7	8559.0	5.3
12	0.3	-671.4	7819.5	4.7
13	0.2	-715.1	7014.8	6.2
14	0.1	-754.8	6138.9	3.4
15	0.0	-765.3	5237.5	-0.4
16	-0.1	-745.9	4340.6	-4.1
17	-0.2	-697.3	3478.0	-7.3
18	-0.3	-632.9	2673.1	-8.7
19	-0.4	-557.3	1938.2	-10.1
20	-0.5	-470.5	1284.5	-11.5
21	-0.5	-372.4	723.3	-12.9
22	-0.6	-263.0	265.8	-14.3
22	-0.6	0.0	0.0	-14.3

Detailed Message:

Zone 1 – 18ft – Eurocode Ultimate

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Thesis**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**6/9/2012 12:00:00 AM**		
Modified Date:	**2/17/2013 11:34:46 AM**		
Comments:			

STRUCTURE

Structure ID: **STR-77 5DCA**
Description:

CASE-DRILLED SHAFT

Case Name: **EUROCODE ULTIMATE**
Description: **STR-77 5DCA W/ EUROCODE SOIL PROPERTIES**

Foundation Data (5DCA30-145-2)
Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (DIN-ABL 1 EUROCODE ULTIMATE)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	12.5	110	1.76	26.1	0.93	0
2	Soil	40	115	2.63	26.5	1.52	0

Applied Loads-Top of Shaft (STR-77 5DCA30-145-2)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5DCA30-145-2	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **18**
Controlling Applied Load Case Name: **5DCA30-145-2**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	132.1	83.2	0.7
Moment [kip-ft]	12588.8	14366.8	9051.1	0.7

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	6.6
Total Rotation [deg.]	1	2.2
Nonrecoverable Displacement [in]	1.92	4.0
Nonrecoverable Rotation [deg.]	0.5	1.3

	Maximum Value	Depth of Occurance
Shear:	920.0 kips	12.5ft
Moment:	12844.7 kips-ft	0.9 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	6.8	117.9	12588.8	0.0
1	6.3	117.9	12706.7	0.0
Ground Level (0)	5.8	117.9	12824.6	0.0
1	5.4	67.0	12843.2	7.1
2	4.9	3.9	12791.3	8.6
3	4.4	-71.2	12657.3	10.0
4	3.9	-157.8	12429.8	11.5
5	3.5	-255.8	12097.6	12.9
6	3.0	-364.8	11649.8	14.2
7	2.5	-484.8	11075.7	15.6
8	2.1	-614.2	10364.7	16.0
9	1.6	-727.7	9518.4	12.7
10	1.1	-815.4	8560.1	9.5
11	0.7	-877.4	7515.9	6.3
12	0.2	-913.7	6411.5	3.1
13	-0.3	-905.4	5225.9	-4.8
14	-0.7	-844.3	4012.1	-9.9
15	-1.2	-742.4	2865.3	-15.0
16	-1.7	-600.0	1826.3	-20.1
17	-2.1	-417.1	935.8	-25.1
18	-2.6	-193.7	234.4	-30.2
18	-2.6	0.0	0.0	-30.2

Detailed Message:

Applied Shear Load Exceeds Design Capacity
Applied Moment Exceeds Design Capacity

Zone 1 – 19ft – 5% LEL Mean/Eurocode Service

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Thesis**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**6/9/2012 12:00:00 AM**		
Modified Date:	**2/26/2013 11:28:45 PM**		
Comments:			

STRUCTURE

Structure ID: **STR-77 5DCA**
Description:

CASE-DRILLED SHAFT

Case Name: **.05 LEL OF MEAN**
Description: **NORMAL DISTRIBUTION .05 LEL OF THE MEAN**

Foundation Data (5DCA30-145-2)
Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (DIN-ABL 1 MEAN .05 LEL STRIPPED)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	12.5	110	2.05	30.3	1.03	0
2	Soil	40	115	3.35	31	1.74	0

Applied Loads-Top of Shaft (STR-77 5DCA30-145-2)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5DCA30-145-2	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **19**
Controlling Applied Load Case Name: **5DCA30-145-2**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	218.2	137.5	1.2
Moment [kip-ft]	12588.8	23738.9	14955.5	1.2

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	2.6
Total Rotation [deg.]	1	0.9
Nonrecoverable Displacement [in]	1.92	0.8
Nonrecoverable Rotation [deg.]	0.5	0.3

	Maximum Value	Depth of Occurance
Shear:	798.1 kips	12.8ft
Moment:	12827.1 kips-ft	0.4 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	2.7	117.9	12588.8	0.0
1	2.5	117.9	12706.7	0.0
Ground Level (0)	2.3	117.9	12824.6	0.0
1	2.1	50.3	12802.0	9.4
2	1.9	-34.1	12681.4	11.5
3	1.8	-135.1	12447.2	13.5
4	1.6	-247.9	12098.9	13.8
5	1.4	-352.7	11641.0	12.5
6	1.2	-446.9	11083.5	11.2
7	1.0	-530.5	10437.2	9.8
8	0.9	-603.4	9712.7	8.5
9	0.7	-665.7	8920.5	7.2
10	0.5	-717.3	8071.4	5.9
11	0.3	-758.2	7176.0	4.5
12	0.1	-788.6	6245.0	3.2
13	-0.1	-795.6	5243.1	-1.8
14	-0.2	-755.8	4208.4	-6.5
15	-0.4	-692.2	3226.6	-9.1
16	-0.6	-608.2	2318.5	-11.7
17	-0.8	-503.8	1504.7	-14.2
18	-1.0	-378.8	805.5	-16.8
19	-1.1	-233.5	241.5	-19.3
19	-1.1	0.0	0.0	-19.3

Detailed Message:

Zone 1 – 26ft – 10% LEL Beta

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Thesis**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**6/9/2012 12:00:00 AM**		
Modified Date:	**2/16/2013 4:55:42 PM**		
Comments:			

STRUCTURE

Structure ID: **STR-77 5DCA**
Description:

CASE-DRILLED SHAFT

Case Name: **BETA .1 LEL**
Description: **BETA .1 LEL**

Foundation Data (5DCA30-145-2)
Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (DIN-ABL 1 BETA .1 LEL stripped)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	12.5	110	0.76	25.8	0.6	0
2	Soil	40	115	2.07	27.1	1.14	0

Applied Loads-Top of Shaft (STR-77 5DCA30-145-2)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5DCA30-145-2	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **26**
Controlling Applied Load Case Name: **5DCA30-145-2**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	253.0	159.4	1.4
Moment [kip-ft]	12588.8	27518.6	17336.7	1.4

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	3.6
Total Rotation [deg.]	1	0.9
Nonrecoverable Displacement [in]	1.92	1.0
Nonrecoverable Rotation [deg.]	0.5	0.3

	Maximum Value	Depth of Occurance
Shear:	700.7 kips	18.2ft
Moment:	12884.8 kips-ft	1.8 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	3.7	117.9	12588.8	0.0
1	3.5	117.9	12706.7	0.0
Ground Level (0)	3.3	117.9	12824.6	0.0
1	3.1	85.0	12875.6	4.6
2	2.9	43.2	12882.2	5.7
3	2.8	-7.3	12842.4	6.8
4	2.6	-65.5	12747.7	7.4
5	2.4	-122.6	12594.8	6.9
6	2.2	-176.3	12386.6	6.5
7	2.0	-226.8	12126.2	6.1
8	1.8	-273.9	11817.0	5.7
9	1.7	-317.8	11462.3	5.3
10	1.5	-358.4	11065.3	4.9
11	1.3	-395.7	10629.5	4.5
12	1.1	-429.7	10157.9	4.1
13	0.9	-481.2	9598.3	8.7
14	0.8	-545.6	8924.6	7.5
15	0.6	-600.3	8191.2	6.3
16	0.4	-645.4	7408.0	5.1
17	0.2	-680.7	6584.7	3.9
18	0.0	-700.4	5732.2	0.7
19	-0.2	-688.6	4875.5	-3.3
20	-0.3	-653.9	4043.8	-5.0
21	-0.5	-607.9	3252.5	-6.4
22	-0.7	-550.6	2512.8	-7.8
23	-0.9	-482.1	1836.0	-9.2
24	-1.1	-402.3	1233.3	-10.6
25	-1.3	-311.4	716.0	-12.0
26	-1.4	-209.2	295.3	-13.4
26	-1.4	0.0	0.0	-13.4

Detailed Message:

Zone 2A – 16ft – EPRI/FHWA Ultimate

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Thesis**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**6/9/2012 12:00:00 AM**		
Modified Date:	**3/2/2013 5:23:58 AM**		
Comments:			

STRUCTURE

Structure ID: **ZONE 2A**
Description: **ZONE 2A WITH STR77 LOADS**

CASE-DRILLED SHAFT

Case Name: **2A EPRI/FHWA**
Description: **2A EPRI/FHWA**

Foundation Data (5DCA30-145-2)
Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (2A MEAN)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	7	107	1.11	33	0.38	0
2	Soil	13	116	7.08	38.6	1.35	0

3	**Soil**	**40**	**105**	**8.62**	**39.6**	**1.94**	**0**

Applied Loads-Top of Shaft (STR-77 5DCA30-145-2)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5DCA30-145-2	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **16**
Controlling Applied Load Case Name: **5DCA30-145-2**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	219.1	138.0	1.2
Moment [kip-ft]	12588.8	23830.5	15013.2	1.2

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	2.7
Total Rotation [deg.]	1	0.9
Nonrecoverable Displacement [in]	1.92	0.7
Nonrecoverable Rotation [deg.]	0.5	0.3

	Maximum Value	Depth of Occurance
Shear:	939.2 kips	12.1ft
Moment:	12881.4 kips-ft	1.6 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	2.7	117.9	12588.8	0.0
1	2.5	117.9	12706.7	0.0
Ground Level (0)	2.3	117.9	12824.6	0.0
1	2.1	87.8	12874.8	4.4
2	1.9	47.0	12873.9	5.7
3	1.7	-4.7	12811.1	7.1
4	1.5	-66.4	12685.5	7.8
5	1.3	-125.4	12498.6	7.0
6	1.2	-177.9	12255.9	6.2
7	1.0	-223.9	11964.0	5.4
8	0.8	-455.8	11043.3	26.5
9	0.6	-644.0	9912.6	21.1
10	0.4	-788.5	8615.5	15.6
11	0.2	-889.5	7195.7	10.2
12	0.0	-939.2	5697.6	0.4
13	-0.2	-883.7	4202.7	-10.4
14	-0.4	-741.2	2682.5	-21.1
15	-0.6	-539.5	1334.5	-28.6
16	-0.8	-278.4	217.9	-36.0
16	-0.8	0.0	0.0	-36.0

Detailed Message:

Zone 2A – 19ft – EPRI/FHWA Service

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Thesis**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**6/9/2012 12:00:00 AM**		
Modified Date:	**2/18/2013 8:55:11 PM**		
Comments:			

STRUCTURE

Structure ID: **ZONE 2A**
Description: **ZONE 2A WITH STR77 LOADS**

CASE-DRILLED SHAFT

Case Name: **2A EPRI/FHWA**
Description: **2A EPRI/FHWA**

Foundation Data (5DCA30-145-2)
Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (2A MEAN)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	7	107	1.11	33	0.38	0
2	Soil	13	116	7.08	38.6	1.35	0

3	**Soil**	**40**	**105**	**8.62**	**39.6**	**1.94**	**0**

Applied Loads-Top of Shaft (STR-77 5DCA30-145-2)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5DCA30-145-2	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **19**
Controlling Applied Load Case Name: **5DCA30-145-2**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	361.5	227.7	1.9
Moment [kip-ft]	12588.8	39319.2	24771.1	2.0

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	1.6
Total Rotation [deg.]	1	0.5
Nonrecoverable Displacement [in]	1.92	0.4
Nonrecoverable Rotation [deg.]	0.5	0.1

	Maximum Value	Depth of Occurance
Shear:	948.1 kips	14.1ft
Moment:	12899.1 kips-ft	2.1 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	1.7	117.9	12588.8	0.0
1	1.6	117.9	12706.7	0.0
Ground Level (0)	1.5	117.9	12824.6	0.0
1	1.4	87.7	12879.8	4.4
2	1.3	46.8	12899.1	5.7
3	1.1	1.4	12874.1	5.5
4	1.0	-41.1	12805.2	5.2
5	0.9	-80.7	12695.3	4.8
6	0.8	-117.5	12547.1	4.4
7	0.7	-151.4	12363.7	4.1
8	0.6	-328.5	11810.7	21.0
9	0.5	-484.8	11091.1	18.4
10	0.4	-620.4	10225.4	15.8
11	0.3	-735.2	9234.6	13.2
12	0.2	-829.3	8139.4	10.6
13	0.1	-902.7	6960.4	8.0
14	0.0	-948.1	5648.8	0.8
15	-0.1	-906.8	4335.2	-10.1
16	-0.2	-802.0	3099.1	-14.9
17	-0.3	-665.4	1983.8	-18.8
18	-0.4	-497.4	1020.7	-22.8
19	-0.5	-298.0	241.4	-26.7
19	-0.5	0.0	0.0	-26.7

Detailed Message:

Zone 2A – 18ft – Eurocode Ultimate

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Thesis**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**6/9/2012 12:00:00 AM**		
Modified Date:	**2/18/2013 8:51:29 PM**		
Comments:			

STRUCTURE

Structure ID: **ZONE 2A**
Description: **ZONE 2A WITH STR77 LOADS**

CASE-DRILLED SHAFT

Case Name: **2A EUROCODE ULTIMATE**
Description: **2A EUROCODE ULTIMATE**

Foundation Data (5DCA30-145-2)
Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (2A EUROCODE ULTIMATE)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	7	107	0.57	25.7	0.21	0
2	Soil	13	116	4.64	30.8	0.92	0

3	**Soil**	**40**	**105**	**5.73**	**31.9**	**1.36**	**0**

Applied Loads-Top of Shaft (STR-77 5DCA30-145-2)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5DCA30-145-2	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **18**
Controlling Applied Load Case Name: **5DCA30-145-2**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	126.2	79.5	0.7
Moment [kip-ft]	12588.8	13725.7	8647.2	0.7

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	6.8
Total Rotation [deg.]	1	2.1
Nonrecoverable Displacement [in]	1.92	3.8
Nonrecoverable Rotation [deg.]	0.5	1.2

	Maximum Value	Depth of Occurance
Shear:	1115.2 kips	13.7ft
Moment:	13052.6 kips-ft	4.0 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	6.9	117.9	12588.8	0.0
1	6.4	117.9	12706.7	0.0
Ground Level (0)	6.0	117.9	12824.6	0.0
1	5.6	105.4	12917.8	1.8
2	5.1	87.8	12990.4	2.5
3	4.7	64.9	13037.1	3.2
4	4.2	36.7	13052.6	3.8
5	3.8	3.1	13031.3	4.5
6	3.4	-36.0	12967.8	5.2
7	2.9	-80.6	12856.5	5.9
8	2.5	-250.2	12447.6	22.0
9	2.0	-434.9	11841.3	24.0
10	1.6	-634.9	11022.6	25.9
11	1.2	-842.1	9977.6	23.7
12	0.7	-996.5	8728.1	15.7
13	0.3	-1086.2	7336.8	7.6
14	-0.2	-1101.4	5697.9	-6.5
15	-0.6	-1003.6	4079.5	-16.9
16	-1.1	-823.0	2574.2	-27.2
17	-1.5	-559.9	1264.9	-37.5
18	-1.9	-214.3	234.1	-47.8
18	-1.9	0.0	0.0	-47.8

Detailed Message:

Applied Shear Load Exceeds Design Capacity
Applied Moment Exceeds Design Capacity

Zone 2A – 17ft – 5% LEL Mean/Eurocode Service

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Thesis**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**6/9/2012 12:00:00 AM**		
Modified Date:	**2/26/2013 11:26:11 PM**		
Comments:			

STRUCTURE

Structure ID: **ZONE 2A**
Description: **ZONE 2A WITH STR77 LOADS**

CASE-DRILLED SHAFT

Case Name: **2A .05 LEL MEAN**
Description: **2A .05 LEL MEAN**

Foundation Data (5DCA30-145-2)
Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (2A 0.05 LEL MEAN)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	7	107	0.79	31	0.26	0
2	Soil	13	116	6.49	36.7	1.15	0

3	**Soil**	**40**	**105**	**8.02**	**37.8**	**1.7**	**0**

Applied Loads-Top of Shaft (STR-77 5DCA30-145-2)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5DCA30-145-2	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **17**
Controlling Applied Load Case Name: **5DCA30-145-2**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	196.4	123.8	1.0
Moment [kip-ft]	12588.8	21367.8	13461.7	1.1

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	2.8
Total Rotation [deg.]	1	0.9
Nonrecoverable Displacement [in]	1.92	0.7
Nonrecoverable Rotation [deg.]	0.5	0.3

	Maximum Value	Depth of Occurance
Shear:	924.4 kips	12.8ft
Moment:	12949.5 kips-ft	2.6 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	2.8	117.9	12588.8	0.0
1	2.6	117.9	12706.7	0.0
Ground Level (0)	2.4	117.9	12824.6	0.0
1	2.2	98.7	12900.9	2.8
2	2.1	71.5	12943.5	3.8
3	1.9	36.3	12944.0	4.9
4	1.7	-7.0	12896.2	5.8
5	1.5	-51.4	12802.6	5.3
6	1.3	-91.3	12666.9	4.7
7	1.1	-126.9	12493.4	4.2
8	0.9	-359.5	11737.9	26.9
9	0.7	-553.0	10752.7	22.0
10	0.5	-707.5	9593.5	17.1
11	0.3	-822.9	8299.4	12.2
12	0.1	-899.2	6909.4	7.3
13	-0.1	-918.8	5466.5	-3.9
14	-0.2	-834.3	3935.6	-13.6
15	-0.4	-696.1	2516.3	-20.3
16	-0.6	-504.9	1261.8	-26.9
17	-0.8	-260.6	225.0	-33.5
17	-0.8	0.0	0.0	-33.5

Detailed Message:

Zone 2A – 19ft – 10% LEL BETA

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Thesis**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**6/9/2012 12:00:00 AM**		
Modified Date:	**2/18/2013 8:47:58 PM**		
Comments:			

STRUCTURE

Structure ID: **ZONE 2A**
Description: **ZONE 2A WITH STR77 LOADS**

CASE-DRILLED SHAFT

Case Name: **2A 0.1 LEL BETA**
Description: **2A 0.1 LEL BETA**

Foundation Data (5DCA30-145-2)
Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (Zone 2A 0.10 LEL BETA)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	7	107	0.41	27.8	0.08	0
2	Soil	13	116	5.55	33.8	0.84	0

3	**Soil**	**40**	**105**	**6.37**	**33.2**	**1.1**	**0**

Applied Loads-Top of Shaft (STR-77 5DCA30-145-2)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5DCA30-145-2	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **19**
Controlling Applied Load Case Name: **5DCA30-145-2**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	149.4	94.1	0.8
Moment [kip-ft]	12588.8	16254.9	10240.6	0.8

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	3.3
Total Rotation [deg.]	1	1.0
Nonrecoverable Displacement [in]	1.92	0.9
Nonrecoverable Rotation [deg.]	0.5	0.3

	Maximum Value	Depth of Occurance
Shear:	912.4 kips	13.9ft
Moment:	13163.9 kips-ft	5.2 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	3.3	117.9	12588.8	0.0
1	3.1	117.9	12706.7	0.0
Ground Level (0)	2.9	117.9	12824.6	0.0
1	2.7	111.6	12930.2	1.0
2	2.5	100.7	13021.9	1.6
3	2.3	85.1	13094.7	2.2
4	2.1	64.6	13143.7	2.8
5	1.9	39.4	13163.7	3.2
6	1.6	14.8	13154.2	2.9
7	1.4	-7.5	13121.3	2.7
8	1.2	-198.0	12716.3	24.8
9	1.0	-400.1	12085.5	24.1
10	0.8	-573.7	11237.3	19.7
11	0.6	-712.0	10206.0	15.3
12	0.4	-814.8	9026.8	10.9
13	0.2	-882.2	7735.0	6.4
14	0.0	-910.3	6309.9	-2.1
15	-0.2	-857.9	4872.0	-9.0
16	-0.5	-761.6	3493.9	-14.5
17	-0.7	-622.0	2233.7	-19.9
18	-0.9	-439.1	1134.7	-25.3
19	-1.1	-212.9	240.4	-30.7
19	-1.1	0.0	0.0	-30.7

Detailed Message:

Applied Shear Load Exceeds Design Capacity
Applied Moment Exceeds Design Capacity

Zone 2C – 14ft – EPRI/FHWA Ultimate

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Zone 2C**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**2/25/2014 1:38:07 PM**		
Modified Date:	**2/25/2014 1:51:58 PM**		
Comments:			

STRUCTURE

Structure ID: **FHWA Mean**
Description:

CASE-DRILLED SHAFT

Case Name: **fhwa mean**
Description: **fhwa mean**

Foundation Data (5DCT)
Diameter of Drilled Shaft: [ft] **7**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (FHWA_EPRI Mean)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	13	107	2.4	34.2	1.08	0
2	Soil	50	107	4.69	34.6	0.5	0

Applied Loads-Top of Shaft (5DCT)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5dct	59	6912	80.4

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **7**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **14**
Controlling Applied Load Case Name: **5dct**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	59.0	99.1	62.4	1.1
Moment [kip-ft]	6912.0	11803.8	7436.4	1.1

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.36	2.0
Total Rotation [deg.]	1	0.9
Nonrecoverable Displacement [in]	1.68	0.6
Nonrecoverable Rotation [deg.]	0.5	0.3

	Maximum Value	Depth of Occurance
Shear:	528.8 kips	8.7ft
Moment:	7030.0 kips-ft	0.1 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	2.1	59.0	6912.0	0.0
1	1.9	59.0	6971.0	0.0
Ground Level (0)	1.7	59.0	7030.0	0.0
1	1.5	-17.1	6936.0	12.2
2	1.3	-114.5	6728.4	15.3
3	1.1	-217.8	6410.9	13.9
4	0.9	-306.9	5997.1	11.8
5	0.7	-381.8	5501.3	9.8
6	0.5	-442.3	4937.8	7.7
7	0.3	-488.6	4320.9	5.7
8	0.1	-520.5	3664.9	3.6
9	-0.1	-524.9	2988.0	-2.4
10	-0.3	-493.4	2327.1	-5.6
11	-0.5	-444.8	1706.4	-8.0
12	-0.7	-379.2	1142.8	-10.5
13	-0.9	-296.7	653.2	-12.9
14	-1.0	-132.0	142.7	-25.5
14	-1.0	0.0	0.0	-25.5

Detailed Message:

Zone 2C – 18ft – EPRI/FHWA Service

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Thesis**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**6/9/2012 12:00:00 AM**		
Modified Date:	**2/24/2013 10:07:43 PM**		
Comments:			

STRUCTURE

Structure ID: **Zone 2C-STR141-5DCT-160**
Description: **Zone 2C-STR141-5DCT-160**

CASE-DRILLED SHAFT

Case Name: **Zone 2C EPRI/FHWA**
Description: **Zone 2C EPRI/FHWA**

Foundation Data (5DCT-160)
Diameter of Drilled Shaft: [ft] **7**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (Zone 2C mean)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	13	107	2.4	34.2	1.08	0
2	Soil	40	107	4.69	34.6	0.5	0

Applied Loads-Top of Shaft (STR-141-5DCT-160)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	1	59	6912	80.4

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **7**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **18**
Controlling Applied Load Case Name: **1**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	59.0	157.9	99.5	1.7
Moment [kip-ft]	6912.0	18816.7	11854.5	1.7

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.36	1.1
Total Rotation [deg.]	1	0.4
Nonrecoverable Displacement [in]	1.68	0.3
Nonrecoverable Rotation [deg.]	0.5	0.1

	Maximum Value	Depth of Occurance
Shear:	476.9 kips	11.7ft
Moment:	7030.0 kips-ft	0.1 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	1.1	59.0	6912.0	0.0
1	1.0	59.0	6971.0	0.0
Ground Level (0)	1.0	59.0	7030.0	0.0
1	0.9	-11.7	6989.9	10.0
2	0.8	-79.3	6880.2	9.4
3	0.7	-142.2	6705.1	8.7
4	0.6	-200.5	6469.5	8.0
5	0.5	-254.2	6177.8	7.4
6	0.5	-303.3	5834.7	6.7
7	0.4	-347.6	5445.0	6.0
8	0.3	-387.3	5013.2	5.4
9	0.2	-422.3	4544.1	4.7
10	0.1	-452.6	4042.3	4.0
11	0.1	-473.3	3513.9	1.8
12	0.0	-475.6	2974.0	-0.9
13	-0.1	-459.4	2441.0	-3.5
14	-0.2	-422.8	1874.0	-6.1
15	-0.3	-373.4	1350.0	-7.9
16	-0.4	-311.2	881.8	-9.7
17	-0.4	-236.3	482.1	-11.5
18	-0.5	-148.6	163.7	-13.3
18	-0.5	0.0	0.0	-13.3

Detailed Message:

Zone 2C – 18ft – Eurocode Ultimate

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Zone 2C**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**2/25/2014 1:38:07 PM**		
Modified Date:	**2/25/2014 1:45:14 PM**		
Comments:			

STRUCTURE

Structure ID: **Eurocode ult**
Description:

CASE-DRILLED SHAFT

Case Name: **Eurocode Ult**
Description: **eurocode ult**

Foundation Data (5DCT)
Diameter of Drilled Shaft: [ft] **7**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (2C Eurocode)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	13	107	1.41	27.1	0.52	0
2	Soil	50	107	2.46	26.6	0.4	0

Applied Loads-Top of Shaft (5DCT)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5dct	59	6912	80.4

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **7**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **18**
Controlling Applied Load Case Name: **5dct**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	59.0	63.9	40.3	0.7
Moment [kip-ft]	6912.0	7614.0	4796.8	0.7

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.36	4.4
Total Rotation [deg.]	1	1.5
Nonrecoverable Displacement [in]	1.68	2.4
Nonrecoverable Rotation [deg.]	0.5	0.8

	Maximum Value	Depth of Occurance
Shear:	522.7 kips	12.4ft
Moment:	7040.1 kips-ft	0.9 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	4.4	59.0	6912.0	0.0
1	4.1	59.0	6971.0	0.0
Ground Level (0)	3.8	59.0	7030.0	0.0
1	3.5	32.1	7039.0	4.4
2	3.2	-3.1	7008.6	5.6
3	2.9	-46.5	6930.7	6.7
4	2.6	-97.8	6797.4	7.9
5	2.3	-157.2	6600.8	9.0
6	2.0	-224.5	6333.0	10.1
7	1.6	-299.4	5986.2	10.9
8	1.3	-369.0	5557.8	9.1
9	1.0	-425.9	5058.4	7.3
10	0.7	-470.1	4500.7	5.5
11	0.4	-501.6	3897.5	3.7
12	0.1	-520.5	3261.4	1.9
13	-0.2	-515.6	2609.0	-2.3
14	-0.5	-481.4	1994.3	-6.4
15	-0.8	-423.7	1418.8	-9.8
16	-1.1	-342.3	906.1	-13.1
17	-1.4	-237.3	479.9	-16.5
18	-1.7	-108.9	163.6	-19.8
18	-1.7	0.0	0.0	-19.8

Detailed Message:

Applied Shear Load Exceeds Design Capacity
Applied Moment Exceeds Design Capacity

Zone 2C – 16ft – 5% LEL Mean/Eurocode Service

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Thesis**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**6/9/2012 12:00:00 AM**		
Modified Date:	**2/26/2013 11:27:37 PM**		
Comments:			

STRUCTURE

Structure ID: **Zone 2C-STR141-5DCT-160**
Description: **Zone 2C-STR141-5DCT-160**

CASE-DRILLED SHAFT

Case Name: **2C .05 lel mean**
Description: **2C .05 lel mean**

Foundation Data (5DCT-160)
Diameter of Drilled Shaft: [ft] **7**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (Zone 2C .05 mean)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	13	107	1.97	32.6	0.65	0
2	Soil	40	107	3.44	32	0.5	0

Applied Loads-Top of Shaft (STR-141-5DCT-160)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	1	59	6912	80.4

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] 7
Stick up above Ground Level: [ft] 2
Depth of Embedment: [ft] 16
Controlling Applied Load Case Name: 1

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	59.0	82.2	51.8	0.9
Moment [kip-ft]	6912.0	9795.2	6171.0	0.9

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.36	2.4
Total Rotation [deg.]	1	0.9
Nonrecoverable Displacement [in]	1.68	0.9
Nonrecoverable Rotation [deg.]	0.5	0.4

	Maximum Value	Depth of Occurance
Shear:	497.0 kips	10.5ft
Moment:	7030.0 kips-ft	0.1 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	2.4	59.0	6912.0	0.0
1	2.2	59.0	6971.0	0.0
Ground Level (0)	2.0	59.0	7030.0	0.0
1	1.8	15.9	7001.7	7.0
2	1.6	-40.5	6906.9	8.9
3	1.4	-110.3	6732.3	10.8
4	1.2	-192.7	6464.9	12.1
5	1.0	-270.9	6109.3	10.5
6	0.9	-338.0	5681.1	8.9
7	0.7	-393.8	5191.6	7.2
8	0.5	-438.3	4651.8	5.6
9	0.3	-471.6	4073.1	4.0
10	0.1	-493.7	3466.7	2.4
11	-0.1	-490.7	2848.7	-2.7
12	-0.3	-465.0	2247.0	-4.5
13	-0.5	-426.1	1677.5	-6.4
14	-0.7	-348.9	1073.8	-12.4
15	-0.9	-249.7	558.2	-15.6
16	-1.1	-128.5	152.9	-18.7
16	-1.1	0.0	0.0	-18.7

Detailed Message:

Applied Shear Load Exceeds Design Capacity
Applied Moment Exceeds Design Capacity

Zone 2C – 26ft – 10% LEL BETA

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Zone 2C**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**2/25/2014 1:38:07 PM**		
Modified Date:	**2/25/2014 1:55:48 PM**		
Comments:			

STRUCTURE

Structure ID: **.1 beta**
Description:

CASE-DRILLED SHAFT

Case Name: **Zone 2c .1 beta**
Description: **Zone 2c .1 beta**

Foundation Data (5DCT)
Diameter of Drilled Shaft: [ft] **7**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (zone 2c .1 beta)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	13	107	1.1	28.5	0	0
2	Soil	50	107	2.01	28.3	0.5	0

Applied Loads-Top of Shaft (5DCT)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5dct	59	6912	80.4

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **7**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **26**
Controlling Applied Load Case Name: **5dct**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	59.0	110.6	69.7	1.2
Moment [kip-ft]	6912.0	13180.9	8303.9	1.2

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.36	3.1
Total Rotation [deg.]	1	0.7
Nonrecoverable Displacement [in]	1.68	1.3
Nonrecoverable Rotation [deg.]	0.5	0.3

	Maximum Value	Depth of Occurance
Shear:	443.6 kips	19.0ft
Moment:	7203.0 kips-ft	4.8 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	3.1	59.0	6912.0	0.0
1	3.0	59.0	6971.0	0.0
Ground Level (0)	2.8	59.0	7030.0	0.0
1	2.7	57.3	7086.7	0.5
2	2.5	52.2	7136.2	1.0
3	2.4	43.2	7174.7	1.5
4	2.2	30.3	7198.1	2.1
5	2.1	13.1	7201.9	2.7
6	1.9	-8.5	7181.6	3.4
7	1.8	-34.8	7132.4	4.1
8	1.6	-65.9	7049.4	4.8
9	1.5	-102.1	6927.4	5.5
10	1.3	-143.1	6761.2	5.9
11	1.2	-182.4	6548.5	5.4
12	1.0	-217.7	6295.0	4.8
13	0.9	-249.2	6008.1	4.2
14	0.7	-300.8	5635.4	6.9
15	0.6	-345.2	5214.7	5.9
16	0.4	-382.3	4753.3	4.8
17	0.3	-412.0	4258.5	3.8
18	0.1	-434.6	3737.5	2.8
19	0.0	-443.4	3199.5	-0.3
20	-0.2	-429.1	2664.7	-3.0
21	-0.3	-403.1	2150.8	-4.3
22	-0.5	-368.6	1667.2	-5.5
23	-0.6	-325.6	1222.4	-6.7
24	-0.8	-274.0	824.8	-7.9
25	-0.9	-214.0	483.0	-9.1
26	-1.1	-145.5	205.5	-10.3
26	-1.1	0.0	0.0	-10.3

Detailed Message:

Zone 3A – 20ft EPRI/FHWA Ultimate

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Thesis**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**6/9/2012 12:00:00 AM**		
Modified Date:	**3/2/2013 5:26:36 AM**		
Comments:			

STRUCTURE

Structure ID: **ZONE 3C**
Description: **ZONE 3C**

CASE-DRILLED SHAFT

Case Name: **3C FHWA/EPRI**
Description: **3C FHWA/EPRI**

Foundation Data (5DCA30-145-2)
Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (3C MEAN)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	10	104	2.06	35.7	0.5	0
2	Soil	21	120	4.89	37.3	0.5	0

3	**Soil**	**40**	**127**	**11.04**	**46.4**	**0**	**0**

Applied Loads-Top of Shaft (STR-77 5DCA30-145-2)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5DCA30-145-2	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft]	**8**
Stick up above Ground Level: [ft]	**2**
Depth of Embedment: [ft]	**20**
Controlling Applied Load Case Name:	**5DCA30-145-2**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	208.4	131.3	1.1
Moment [kip-ft]	12588.8	22663.4	14277.9	1.1

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	2.2
Total Rotation [deg.]	1	0.7
Nonrecoverable Displacement [in]	1.92	0.7
Nonrecoverable Rotation [deg.]	0.5	0.2

	Maximum Value	Depth of Occurance
Shear:	778.4 kips	13.8ft
Moment:	12839.9 kips-ft	0.8 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	2.2	117.9	12588.8	0.0
1	2.1	117.9	12706.7	0.0
Ground Level (0)	1.9	117.9	12824.6	0.0
1	1.8	73.0	12836.0	6.4
2	1.7	13.3	12771.4	8.3
3	1.5	-61.4	12626.1	10.2
4	1.4	-150.8	12398.6	11.7
5	1.2	-240.5	12079.9	10.8
6	1.1	-322.2	11675.4	9.8
7	0.9	-396.0	11193.2	8.8
8	0.8	-462.0	10641.1	7.8
9	0.7	-520.1	10026.9	6.8
10	0.5	-570.4	9358.6	5.8
11	0.4	-654.3	8453.9	9.4
12	0.2	-718.4	7475.1	6.9
13	0.1	-762.8	6442.2	4.4
14	0.0	-776.2	5377.4	-1.7
15	-0.2	-740.1	4326.3	-5.9
16	-0.3	-680.5	3323.4	-8.7
17	-0.5	-598.1	2391.5	-11.6
18	-0.6	-493.1	1553.3	-14.4
19	-0.7	-365.3	831.5	-17.2
20	-0.9	-214.8	248.9	-20.1
20	-0.9	0.0	0.0	-20.1

Detailed Message:

Zone 3A – 22ft – EPRI/FHWA Service

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Thesis**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**6/9/2012 12:00:00 AM**		
Modified Date:	**2/19/2013 9:10:03 PM**		
Comments:			

STRUCTURE

Structure ID: **ZONE 3C**
Description: **ZONE 3C**

CASE-DRILLED SHAFT

Case Name: **3C FHWA/EPRI**
Description: **3C FHWA/EPRI**

Foundation Data (5DCA30-145-2)

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**

Model Options

Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (3C MEAN)

Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	10	104	2.06	35.7	0.5	0
2	Soil	21	120	4.89	37.3	0.5	0

3	**Soil**	**40**	**127**	**11.04**	**46.4**	**0**	**0**

Applied Loads-Top of Shaft (STR-77 5DCA30-145-2)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5DCA30-145-2	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **22**
Controlling Applied Load Case Name: **5DCA30-145-2**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	280.9	176.9	1.5
Moment [kip-ft]	12588.8	30551.1	19247.2	1.5

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	1.5
Total Rotation [deg.]	1	0.4
Nonrecoverable Displacement [in]	1.92	0.4
Nonrecoverable Rotation [deg.]	0.5	0.1

	Maximum Value	Depth of Occurance
Shear:	775.1 kips	15.9ft
Moment:	12846.8 kips-ft	1.1 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	1.5	117.9	12588.8	0.0
1	1.4	117.9	12706.7	0.0
Ground Level (0)	1.4	117.9	12824.6	0.0
1	1.3	72.8	12846.8	6.5
2	1.2	12.9	12816.6	8.3
3	1.1	-59.5	12719.3	9.1
4	1.0	-130.0	12549.9	8.6
5	0.9	-196.3	12312.1	8.1
6	0.8	-258.4	12010.1	7.5
7	0.8	-316.3	11648.1	7.0
8	0.7	-370.1	11230.2	6.5
9	0.6	-419.7	10760.6	6.0
10	0.5	-465.2	10243.5	5.5
11	0.4	-546.6	9560.2	9.6
12	0.3	-617.0	8801.0	8.2
13	0.2	-676.3	7977.0	6.8
14	0.2	-724.6	7099.1	5.4
15	0.1	-761.9	6178.5	4.0
16	0.0	-774.6	5230.6	-0.6
17	-0.1	-749.7	4289.1	-4.8
18	-0.2	-703.4	3384.9	-6.6
19	-0.3	-643.3	2533.9	-8.3
20	-0.4	-569.3	1750.0	-10.0
21	-0.4	-481.4	1047.0	-11.8
22	-0.5	-279.5	265.5	-27.0
22	-0.5	0.0	0.0	-27.0

Detailed Message:

Zone 3A – 22ft – Eurocode Ultimate

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Thesis**	**Checked By:**	**____________________**
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	**____________________**
Start Date:	**6/9/2012 12:00:00 AM**		
Modified Date:	**2/19/2013 9:11:22 PM**		
Comments:			

STRUCTURE

Structure ID: ZONE 3C
Description: ZONE 3C

CASE-DRILLED SHAFT

Case Name: 3C EUROCODE ULTIMATE
Description: 3C EUROCODE ULTIMATE

Foundation Data (5DCA30-145-2)
Diameter of Drilled Shaft: [ft] 8
Stick up above Ground Level: [ft] 2

Model Options
Side Shear Spring: On
Base Shear Spring: On
Base Moment Spring: On

Geotechnical Parameters (3C EUROCODE ULTIMATE)
Depth to Ground Water: [ft] 100

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	10	104	0.98	26.8	0.4	0
2	Soil	21	120	2.34	28	0.4	0

3	**Soil**	**40**	**127**	**5.54**	**33.8**	**0**	**0**

Applied Loads-Top of Shaft (STR-77 5DCA30-145-2)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5DCA30-145-2	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **22**
Controlling Applied Load Case Name: **5DCA30-145-2**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	119.5	75.3	0.6
Moment [kip-ft]	12588.8	13001.1	8190.7	0.7

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	8.7
Total Rotation [deg.]	1	2.3
Nonrecoverable Displacement [in]	1.92	5.7
Nonrecoverable Rotation [deg.]	0.5	1.6

	Maximum Value	Depth of Occurance
Shear:	931.3 kips	15.8ft
Moment:	12931.0 kips-ft	2.4 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	8.8	117.9	12588.8	0.0
1	8.3	117.9	12706.7	0.0
Ground Level (0)	7.8	117.9	12824.6	0.0
1	7.3	94.4	12894.4	3.4
2	6.8	63.6	12928.7	4.3
3	6.3	25.4	12920.2	5.2
4	5.8	-20.1	12861.7	6.1
5	5.3	-72.9	12745.8	7.0
6	4.8	-132.9	12565.3	7.9
7	4.3	-200.3	12312.8	8.8
8	3.8	-275.0	11981.0	9.8
9	3.3	-357.1	11562.6	10.7
10	2.8	-446.1	11050.2	11.2
11	2.3	-551.3	10417.1	13.7
12	1.8	-665.4	9663.8	14.8
13	1.3	-780.9	8782.8	12.9
14	0.8	-865.7	7789.7	8.7
15	0.3	-916.5	6717.9	4.5
16	-0.2	-927.4	5602.3	-2.3
17	-0.7	-888.9	4491.6	-6.8
18	-1.2	-814.9	3426.0	-11.2
19	-1.7	-705.6	2441.0	-15.6
20	-2.2	-561.1	1571.6	-20.1
21	-2.7	-381.2	853.1	-24.5
22	-3.1	-177.3	265.8	-26.3
22	-3.1	0.0	0.0	-26.3

Detailed Message:

Applied Shear Load Exceeds Design Capacity
Applied Moment Exceeds Design Capacity

Zone 3A – 21ft – 5% LEL Mean/Eurocode Service

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Thesis**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**6/9/2012 12:00:00 AM**		
Modified Date:	**2/26/2013 11:29:30 PM**		
Comments:			

STRUCTURE

Structure ID: **ZONE 3C**
Description: **ZONE 3C**

CASE-DRILLED SHAFT

Case Name: **3C 0.05 LEL MEAN**
Description: **3C 0.05 LEL MEAN**

Foundation Data (5DCA30-145-2)
Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**

Model Options
Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (3C 0.05 LEL MEAN)
Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	10	104	1.37	32.3	0.5	0
2	Soil	21	120	3.28	33.6	0.5	0

3	**Soil**	**40**	**127**	**7.76**	**39.9**	**0**	**0**

Applied Loads-Top of Shaft (STR-77 5DCA30-145-2)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5DCA30-145-2	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **21**
Controlling Applied Load Case Name: **5DCA30-145-2**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	177.7	112.0	0.9
Moment [kip-ft]	12588.8	19331.0	12178.5	1.0

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	3.3
Total Rotation [deg.]	1	1.0
Nonrecoverable Displacement [in]	1.92	1.1
Nonrecoverable Rotation [deg.]	0.5	0.3

	Maximum Value	Depth of Occurance
Shear:	757.0 kips	14.5ft
Moment:	12859.5 kips-ft	1.2 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	3.3	117.9	12588.8	0.0
1	3.1	117.9	12706.7	0.0
Ground Level (0)	2.9	117.9	12824.6	0.0
1	2.7	80.6	12859.2	5.3
2	2.5	31.7	12834.1	6.8
3	2.3	-29.0	12737.6	8.2
4	2.1	-101.5	12559.5	9.7
5	1.9	-184.8	12300.5	10.6
6	1.7	-265.2	11958.4	9.6
7	1.5	-338.0	11539.6	8.7
8	1.3	-403.3	11051.8	7.7
9	1.1	-461.1	10502.4	6.8
10	0.9	-511.4	9899.0	5.9
11	0.7	-600.2	9073.1	10.0
12	0.5	-670.0	8157.5	7.7
13	0.3	-720.9	7181.5	5.3
14	0.1	-753.0	6164.1	3.0
15	-0.1	-746.0	5131.2	-3.4
16	-0.3	-706.8	4124.1	-6.1
17	-0.5	-646.5	3166.7	-8.7
18	-0.7	-565.2	2280.1	-11.4
19	-0.9	-462.8	1485.4	-14.0
20	-1.1	-339.4	803.6	-16.6
21	-1.3	-194.9	255.8	-19.2
21	-1.3	0.0	0.0	-19.2

Detailed Message:

Applied Shear Load Exceeds Design Capacity
Applied Moment Exceeds Design Capacity

Zone 3A – 23ft – 10% LEL BETA

Foundation Analysis and Design Tools
MFAD Version 5.1.11

Project Name:	**Thesis**	**Checked By:**	____________________
Responsible Engineer:	**Not Assigned**		
Directory:	**C:\Program Files (x86)\FAD Tools**	**Date:**	____________________
Start Date:	**6/9/2012 12:00:00 AM**		
Modified Date:	**2/19/2013 9:06:32 PM**		
Comments:			

STRUCTURE

Structure ID: **ZONE 3C**
Description: **ZONE 3C**

CASE-DRILLED SHAFT

Case Name: **3C 0.1 LEL BETA**
Description: **3C 0.1 LEL BETA**

Foundation Data (5DCA30-145-2)

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**

Model Options

Side Shear Spring: **On**
Base Shear Spring: **On**
Base Moment Spring: **On**

Geotechnical Parameters (3C 0.1 LEL BETA)

Depth to Ground Water: [ft] **100**

Layer No.	Layer Type	Depth to Bottom of Layer [ft]	Total Unit Weight [pcf]	Deformation Modulus [ksi]	Friction Angle [Deg]	Undrained Shear Strength or Rock Cohesion [ksf]	Rock / Concrete Bond Strength [ksf]
1	Soil	10	104	0.96	29.7	0.5	0
2	Soil	21	120	2.03	30	0.5	0

3	**Soil**	**40**	**127**	**6.8**	**37.9**	**0**	**0**

Applied Loads-Top of Shaft (STR-77 5DCA30-145-2)

Load Case No.	Load Case Name	Shear Load [kips]	Moment [kip-ft]	Axial Load [kips]
1	5DCA30-145-2	117.9	12588.8	94.9

DESIGN RESULTS

Diameter of Drilled Shaft: [ft] **8**
Stick up above Ground Level: [ft] **2**
Depth of Embedment: [ft] **23**
Controlling Applied Load Case Name: **5DCA30-145-2**

Capacity Verification

Loading Mode	Applied Load	Nominal Capacity	Design Capacity	Design Capacity / Applied Load
Shear Load [kips]	117.9	182.3	114.8	1.0
Moment [kip-ft]	12588.8	19828.2	12491.7	1.0

Design Capacity is based on a Strength Factor of 0.63

Performance Verification (Top of Shaft)

	Criteria	Actual
Total Displacement [in]	3.84	3.3
Total Rotation [deg.]	1	0.8
Nonrecoverable Displacement [in]	1.92	1.0
Nonrecoverable Rotation [deg.]	0.5	0.3

	Maximum Value	Depth of Occurance
Shear:	735.7 kips	17.1ft
Moment:	12877.7 kips-ft	1.6 ft

Summary of Results For Controlling Applied Load Case

Elevation [ft]	Displacement [in]	Shear Force [kips]	Flexural Moment [kips-ft]	Lateral Pressure [ksf]
Top of Stick (2)	3.3	117.9	12588.8	0.0
1	3.2	117.9	12706.7	0.0
Ground Level (0)	3.0	117.9	12824.6	0.0
1	2.8	85.1	12872.2	4.7
2	2.6	42.5	12869.8	5.9
3	2.5	-10.0	12815.8	7.1
4	2.3	-72.2	12704.5	8.3
5	2.1	-138.4	12527.9	8.0
6	1.9	-200.3	12287.2	7.5
7	1.8	-258.0	11986.7	7.0
8	1.6	-311.5	11630.6	6.4
9	1.4	-360.7	11223.1	5.9
10	1.2	-405.7	10768.6	5.4
11	1.1	-482.2	10173.7	9.0
12	0.9	-549.2	9507.0	7.8
13	0.7	-606.5	8778.2	6.6
14	0.5	-654.2	7996.9	5.4
15	0.4	-692.4	7172.7	4.2
16	0.2	-721.0	6315.1	3.0
17	0.0	-735.7	5434.5	0.3
18	-0.2	-721.3	4553.6	-3.0
19	-0.3	-690.9	3696.4	-4.4
20	-0.5	-649.3	2875.3	-5.8
21	-0.7	-596.5	2101.3	-7.2
22	-0.9	-421.5	1093.0	-23.9
23	-1.0	-210.5	271.0	-28.4
23	-1.0	0.0	0.0	-28.4

Detailed Message:

Applied Shear Load Exceeds Design Capacity
Applied Moment Exceeds Design Capacity