Assessing levels of reliability for design criteria for hurricane and storm damage risk reduction structures

Christopher Leslie Dunn
Louisiana State University and Agricultural and Mechanical College, christopher.dunn.pe@gmail.com

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A Dissertation

Submitted to the Graduate Faculty of the
Louisiana State University and
Agricultural and Mechanical College
in partial fulfillment of the
requirements for the degree of
Doctor of Philosophy

in

The Department of Civil and Environmental Engineering

by

Christopher L. Dunn
B.S., Louisiana State University, 1998
M.S., University of New Orleans, 2001
May 2013
To my loving, supportive, and tenacious wife Kelly

&

To my beautiful daughters, Maggie, Abby, and Molly

&

To my parents

&

To my dogs Sissy, Millie, and Doug
ACKNOWLEDGEMENTS

I would like to express my sincere gratitude to my co-advisor and friend Dr. Marc Levitan for his loyalty to me through my extended graduate study at Louisiana State University. Dr. Levitan had many opportunities to give up on me due to my lack of early progress, but he persevered with me to the end. For that I am eternally grateful.

I would like to thank Dr. Carol Friedland, without whom I would not have made it this far. Dr. Friedland has been a cheerleader when I needed motivation, a counselor when I was overwhelmed to the point of not being able to move forward, a constructive critic when I needed advice on manuscripts, and a friend at all times. She is one of the driving forces that has helped me see the end and move toward completion. I can never fully repay her efforts and this dissertation is a testament to her professionalism and dedication to research and higher education.

I am grateful to Dr. Ayman Okeil who graciously agreed to serve as a co-advisor and has provided invaluable guidance in reliability analysis. I am also grateful to Dr. Peter Cali, who guided my efforts in the geotechnical arena and helped me continue to refine the scope of my study. I also owe Dr. Clinton Willson a debt of gratitude for not only his assistance as a committee member, but also as a liaison for me with the Graduate School.

Special thanks to my father-in-law, Dr. John Grieshaber, who served as the inspiration for this endeavor and provided advice and guidance throughout the pursuit of this degree as well as my entire career. Thanks are due to Dr. Bob Ebeling, Dr. Rich Varuso, and Dr. Therese Koutnik, who provided valuable input and assistance when I was dealing with difficult topics beyond my usual sphere of knowledge. I would especially like to thank Jehu Johnson, without whose assistance with SLOPE/W, this dissertation would not have been possible, and Nancy Powell who helped pose the question to prompt this research.
I would like to express my gratitude to the U.S. Army Corps of Engineers for the opportunity to fulfill my full-time residency requirement. Not only did I have the opportunity for full-time study for one year, but thanks to Walter Baumy for supporting the continuation of my year of study in spite of the immediate need for designers in the wake of Hurricane Katrina. This opportunity is one of many he has provided me throughout my career and for that, I thank him. I would like to thank Mark Gonski, who has also played a pivotal role in my career and supported this effort.

I thank my mother, who taught me to value education, and my father who taught me that tenacity was a virtue and quitting is unacceptable. I thank my children, who have known me as a graduate student their entire lives. I thank my dogs, both living (Sissy and Doug) and deceased (Millie) for giving me unspoken support at my feet in my home office. Lastly I thank my wife. She has persevered when it seemed I would never graduate and suffered through the difficulties of my never-ending commitments both to this dissertation and the U.S. Army Corps of Engineers. She has remained patient and understanding, and she has been my rock through this entire journey. To her I say: I love you and we finally reached the end of this incredibly long tunnel! This proves the power of prayer, perseverance and hard work.
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ABSTRACT

In the wake of Hurricane Katrina, the U.S. Army Corps of Engineers (USACE) updated design methodologies and required factors of safety for hurricane and storm damage risk reduction system (HSDRRS) structures to incorporate lessons-learned from the system performance during Katrina and results of state-of-the-art research in storm surge modeling and foundation behavior. However, the criteria (USACE 2008) were not calibrated to a target reliability, which creates the need to understand the reliability provided by designs using those criteria, especially for pile-founded structures subject to global instability. This dissertation presents a methodology for quantifying the reliability of pile-founded structures that can be applied to hurricane risk reduction structures or more broadly to other types of pile-founded structures. The emphasis of this study is on a representative hurricane risk reduction structure designed using the new USACE criteria, for which the reliability is quantified for comparison to industry target reliabilities.

A designer-friendly methodology for quantifying the reliability of hurricane risk reduction structures is presented, along with recommendations developed from a state-of-the-art review of geotechnical, hydraulic, and structural uncertainty data. This methodology utilizes commercial software and routine design methods for the development of inputs into an overarching framework that includes point estimate simulation models and event tree methods to quantify the structure’s system reliability. The methodology is used to illustrate differences in analysis results with and without accounting for variance reductions due to spatial correlation are also presented through stability and flowthrough limit states. Element reliabilities and overarching “system” reliabilities for a representative structure are quantified for hydrostatic hurricane storm surge loadings, soil loading, and dead loads. Wave loadings and impact loadings are not considered.
The use of variance reductions on undrained shear strengths for point estimate simulations produced higher system reliability indices than the simulations not considering variance reductions for the stability and flowthrough limit states. Using the reduced variances, computed element and system reliabilities were above the industry target reliability indices presented in the literature.
CHAPTER 1: INTRODUCTION

In the wake of Hurricane Katrina and the performance of the Greater New Orleans Hurricane and Storm Damage Risk Reduction System (HSDRRS) when tested by Katrina’s storm surge, the U.S. Army Corps of Engineers (USACE) revisited design methodologies and criteria used in the development of the pre-2005 system. USACE changed the way design elevations were determined by developing detailed storm surge models to determine design elevations and by implementing more stringent design criteria including higher confidence levels for determination of design water surface elevations and wave parameters. USACE also increased deterministic geotechnical allowable factors of safety and introduced tighter restrictions on the types of materials that could be used for the construction of levees and floodwalls. The expected result of these changes is a stronger system that will significantly reduce the risk of flooding for the 1% exceedance surge elevation for all areas protected by the Federal system. USACE compiled all these changes in the HSDRRS Design Guidelines (USACE 2008) specifically for use in the Greater New Orleans HSDRRS, with broader applicability to other HSDRRS projects throughout southeast Louisiana.

Given that the criteria developed for inclusion in the HSDRRS Design Guidelines (USACE 2008) were largely developed by separate discipline-specific teams of experts in the fields of hydrology and hydraulics engineering, geotechnical engineering, and structural engineering, without calibration of the design requirements to a target reliability, there is a need to understand exactly what level of reliability is provided by the criteria presented in the Design Guidelines. Not knowing the level of reliability is an important concern. For example, it is possible that a higher reliability than intended exists, which introduces increased robustness into the finished HSDRRS structure. This in turn could result in dramatic increases in cost, which could threaten the viability of not-yet-funded projects.
This dissertation will use a combination of literature review and probabilistic modeling of a representative structure to develop a simplified multi-disciplinary methodology for computing the level of reliability provided by structures designed using the HSDRRS Design Guidelines (USACE 2008). Since probabilistic methods have been applied to levees and I-walls (IPET 2009), this study will fill the void in the literature for probabilistically representing a pile-founded hurricane risk reduction structure. This methodology will incorporate the USACE model for pile-founded structure resistance to global instability and can be used by others as a first step toward quantifying the reliability and risk associated with a variety of pile-founded structures, with an emphasis on HSDRRS structures. Once reliability of pile-founded hurricane risk reduction structures is better understood, the HSDRRS Design Guidelines can be calibrated to provide the intended performance level. The pile-founded hurricane risk reduction T-wall, or T-wall, has been selected for this study. The T-wall foundation is representative of all pile-founded hurricane risk reduction structures, but possesses a relatively simple superstructure.

As part of this work, statistical parameters required to conduct the reliability analysis are investigated and available probabilistic models for pile-founded structures are explored. From these investigations, the methodology is developed and a test model is constructed. The culmination of all efforts is the development of a methodology by which the level of reliability provided by the test-case T-wall can be determined.

1.1 Problem Statement

The development of the USACE 2008 edition of the Hurricane and Storm Damage Risk Reduction Design Guidelines (USACE 2008) after Hurricane Katrina required integration of hydrology and hydraulics engineering, geotechnical engineering, and structural engineering disciplines. This effort relied upon expert judgment and deterministic metrics (factors of safety, load factors, and resistance factors) that historically produced successful designs. Modern design
guidelines, however, are generally calibrated to result in a specified target reliability index. As a result, the overall reliability of hurricane risk reduction structures designed using the HSDRRS Design Guidelines for specified design storm surge exceedance probabilities has not been quantified. Therefore, there is a need to develop a methodology that can be used to quantify the reliability actually being provided as a first step toward understanding the reliability inherent in designs using the HSDRRS Design Guidelines. This methodology can then be used to quantify the reliability provided for comparison to target reliabilities used by other codes.

1.2 Goals of the Study

The primary goals of this research are to develop a simplified, interdisciplinary designer-friendly methodology for the quantification of the reliability of pile-founded hurricane risk reduction structures designed using the current HSDRRS criteria and to apply that methodology to quantify the reliability of a representative structure in southeast Louisiana (the T-wall) for comparison with target reliabilities used by other codes.

1.3 Objectives

Using a literature search and probabilistic modeling, this study accomplishes the goal of developing a methodology for quantifying the level of reliability provided by designs utilizing the HSDRRS Design Guidelines (USACE 2008) and applies that methodology to a representative southeast Louisiana pile-founded structure, which for this study is the T-wall. The five main objectives (and related sub-objectives) to accomplish the goal of this study are:

1. To perform an examination of the 2008 HSDRRS Design Guidelines. This examination reviews the background of the HSDRRS Design Guidelines, the current design methodologies, and software currently in use. The purpose of this examination is the description of the levee-T-wall structure, identification of the load path, limit states and governing equations for the levee-T-wall structure, and development of preliminary
observations about the specific criteria that may have the potential for contributing to the compounding of factors of safety.

2. To develop the dataset for critical variables from the existing body of literature. The existing body of literature is examined to identify the datasets available to support development of the proposed reliability analysis model, including statistical characterization of critical variables and appropriate parameters for use in the reliability model.

3. To assess the suitability of using a design-level dataset for determining spatial correlation of geotechnical parameters. Field and laboratory test data from a real-world floodwall project is analyzed in an attempt to develop variance reductions for undrained shear strengths. Scales of fluctuation for both vertical and horizontal direction are computed. The results are assessed for potential application in quantifying hurricane risk reduction structure reliability.

4. To develop a simulation modeling methodology for the reliability analysis, to develop a method for setting a threshold for global instability in reliability analysis, and to validate the need to use spatial averaging. The state of the art for risk based analysis methods is examined, with an emphasis on reliability analysis through the use of First-Order Reliability Methods, Point Estimate Methods, and Monte Carlo Simulations. It also includes development of performance functions and identification of simplifying assumptions required for development of the probabilistic model that is employed for the reliability analysis. The model is developed from a project site where topographic and foundation conditions, when exposed to the design hurricane storm surge elevations, are known to produce an unbalanced load (Burns 2009) when conventional deterministic design procedures are followed. Here, the model is developed to address reliability relative to global instability and “flowthrough”. A methodology for setting the threshold for considering a levee-T-wall section globally unstable is also developed. The need for spatial
averaging is assessed by comparing the reliability computed both with and without variance reductions for undrained shear strengths. The initial components of a system model is developed using the event tree method.

5. **To construct the remainder of the complete system simulation model for the test case and to quantify its reliability.** A complete simulation model representing all other elements of the levee-T-wall system is developed. Reliability simulations are performed and both element reliability and complete system reliability are quantified.

1.4 **Scope of the Study**

This study focuses on the southeast Louisiana region, given the variable geologic conditions that can dramatically impact designs, especially from the geotechnical standpoint. This study has further applicability to areas having similar loads and foundation conditions. It focuses on the surge component of the hurricane hazard and the resistance provided by a pile-founded T-wall structure. Barge and vessel impact loads are not addressed quantitatively by this study, nor are wave loads, wind loads, or other typical HSDRRS load conditions.


First-Order Reliability Methods, point estimate methods, and Monte Carlo simulations are researched and models are developed for the quantification of reliability. The reliability
model builds upon methods previously utilized by USACE and others (Rosenbleuth 1975, Wolff 1994, Christian 2004, USACE 2006, IPET 2009). The representative structure selected for this study draws from design parameters and constraints from a real-world structure to provide a representative design exposed to both aggressive loading conditions and challenging site constraints that impact structural resistance. This study uses software commonly utilized for the design of levees and pile-founded floodwalls (ENSOFT 2006) (Geo-Slope 2008) (i.e. no specialty software requiring large computational capacity). The computer software packages used are commercially available programs (or derivations thereof).

1.5 Organization of the Dissertation

This dissertation is organized as a series of manuscripts. Chapter 2 presents an examination of the state-of-the-art for statistical characterization of critical variables most affecting survival or failure of the levee-pile-founded-T-wall structure and the selection of the appropriate characterizations for the reliability analysis that will be performed as part of this study, satisfying Objectives 1 and 2. Chapter 3 presents the methodology for addressing geotechnical uncertainty in this type of reliability analysis in situations where limited site-specific geotechnical data are available, meeting the requirements of Objective 3. Chapter 4 develops the element reliability for flowthrough and demonstrates the importance of accounting for spatial autocorrelation in undrained shear strength of soils, satisfying Objective 4. Chapter 5 ties the different components of this study together by presenting the proposed methodology and the results obtained from application of the methodology, satisfying Objective 5. Chapter 6 describes the conclusions drawn from the research effort, proposes future investigations into refinements of the models utilized for this effort, and recommendations for future study.
CHAPTER 2: STATISTICAL REPRESENTATION OF DESIGN PARAMETERS FOR HURRICANE RISK REDUCTION STRUCTURES

2.1 Introduction

Prior to 2005, the U.S. Army Corps of Engineers (USACE) utilized reliability and risk-based methods to assist in prioritizing repairs on aging, existing inland navigation infrastructure (USACE 1992, 1995). In the wake of Hurricane Katrina, these methods were expanded to include quantification of risk in the context of an entire system of levees, floodwalls, and other surge-resisting structures. Much attention was given to hurricane risk reduction levees and geotechnical aspects of I-walls (IPET 2009). However, additional refinement can be made to address the structural components of I-walls, and on the reliability analysis of complex hurricane risk reduction structures with pile foundations such as T-walls, drainage structures, and navigation structures.

A first step in the risk quantification of complex systems is the characterization of the uncertainty inherent in each of its elements. This is true whether the objective is to validate design criteria or to evaluate the adequacy of an aging hurricane risk reduction system. Given the discipline-specific nature of the literature, a researcher could spend a considerable amount of time reviewing the geotechnical, structural, and hydraulics literature for tabulations of relevant statistical data. Even if a site-specific or a problem-specific data sample is used to characterize the uncertainty of different design parameters, it is useful to have a single source of values documented by other researchers to assure no anomalies may skew reliability analysis results. Equally useful is a single source of recommendations for the treatment of design parameters as variables, constants, or moving constants. This chapter adds to the existing body of knowledge by providing a single source for multi-discipline statistical data required to conduct reliability analysis on hurricane risk reduction structures or, more generally, to other problems involving resistance to lateral loads through soil-structure interaction. Tabulations of purely geotechnical or
purely structural data exist (Ellingwood et al. 1980, USACE 1995, Lacasse and Nadim 1996, Phoon and Kulhawy 1999b, Duncan and Wright 2005, USACE 2006). However, no one source contains the critical variables belonging to all disciplines involved in the reliability analysis of all components of a hurricane risk reduction structures with pile foundations. This chapter attempts to fill that need.

Because the emphasis of this chapter is on hurricane risk reduction structures, the assignment of variables, constants, and moving constants presented here is based on the governing equations that form the basis of current U.S. Army Corps of Engineers (USACE) design methodologies and commercial off-the-shelf software utilized in conjunction with the Hurricane and Storm Damage Risk Reduction System (HSDRRS) Design Guidelines (USACE 2008). This approach to categorizing parameters results in a tabulation of values well suited for a production-oriented approach for multiple structures in larger systems, and can also be adapted for highly specialized finite element reliability analysis of a single element. Since it is one of the most common hurricane risk reduction structures, the pile-founded HSDRRS T-wall (hereafter referred to as T-wall) was used as a representative structure for the purposes of identifying key parameters of load and resistance for critical limit states.

2.1.1 Hurricane Risk Reduction T-walls

The T-wall is similar to a retaining wall and can be reduced to five primary elements: the stem, base slab, foundation piles, cut-off sheet piles, and embankment (levee) (Figure 2.1), and the interfaces/connections between them. Scour protection is a secondary element provided only for those floodwalls that rely on embankment sections to assist in resisting global instability. The four elements of greatest interest are the stem, base slab, foundation piles, and embankment. These four elements form the load path transmitting storm surge and wave loadings to the deeper foundation strata. Provided the pile spacing is sufficiently small to prevent flow-through of the
soil between the piles, the cut-off sheet piling functions as seepage or piping cut-off (providing an underground barrier to storm surge), which is not of interest when considering structural resistance (Figure 2.2). Additionally, as noted by Varuso (2010), the sheet piling has little effect on loads transmitted to the foundation piles. All pile-founded HSDRRS structures contain similar elements and are subjected to similar loads, with some variation in the actual load path.

![Diagram of typical pile-founded hurricane risk reduction T-wall]

Figure 2.1  Elements of the typical pile-founded hurricane risk reduction T-wall.

During a tropical storm, hurricane, or unusual tidal event, storm surge rises on the flood side of the floodwall, covering the flood side base slab and embankment (Figure 2.2). All gravity loads and uplift due to static head of the storm surge are transmitted directly to the base slab. Lateral forces due to embankment at-rest pressures, storm surge, and waves are transmitted directly to the stem and each face of the base slab. Global instability caused by the storm surge loading and the driving force components of the active zone of the failure surface can also induce loading (unbalanced load) imparted partially to the base slab and partially to the pile-soil mass between the base slab and the failure plane. This global instability, if applicable, is assumed to
be transmitted to the piles in the form of a projected uniform load, termed “unbalanced load” (Figure 2.2).

The loads applied directly to the stem and base slab are transmitted to the pile group in the form of axial forces, shear forces, and moments. The unbalanced load is resisted through the flexural and shear resistance of the piles and the shear resistance of the soil between the piles as demonstrated through both numerical (Geomatrix 2007, Varuso 2010) and physical (centrifuge) modeling (Abdoun and Sasanakul 2007). Ultimately, all loads are transmitted to the deeper foundation strata through lateral soil resistance, skin friction along the pile, and, if applicable, pile end bearing.

![General hurricane loads and dead loads on a typical pile-founded hurricane risk reduction T-wall.](image)

**Figure 2.2** General hurricane loads and dead loads on a typical pile-founded hurricane risk reduction T-wall.

### 2.2 Identification of Design Parameters

For a T-wall, parameters of interest for geotechnical resistance are associated with the method for assessing global stability, the method for computing theoretical axial pile capacity, and the method for addressing a pile group’s ability to resist lateral loads. Parameters of interest
for structural resistance are associated with classical resistance models for steel piles, reinforced concrete superstructure, and the pile-superstructure connection. Loads of primary importance in T-wall designs are dead loads, hurricane storm surge loads, and hurricane induced wave loads.

2.2.1 Identification of Geotechnical Resistance Parameters

2.2.1.1 Global (Slope) Stability

Currently, global stability computations for HSDRRS structures, are typically performed using the Spencer (1967) method. Spencer’s method is a Limit Equilibrium method that assumes a circular slip surface and satisfies both force and moment equilibrium. The governing equations for the Spencer’s Method, as illustrated by Geo-Slope (2008), show that embankment resistance is a function of undrained shear strength, $s_u$, unit weight of soil, $\gamma_{soil}$, unit weight of groundwater, $\gamma_{GWT}$, angle of internal friction of the soil, $\phi'$, and geometric factors such as ground geometry/slope angle(s), depth to groundwater table, and depth of soil strata.

2.2.1.2 Lateral Soil Resistance

Lateral soil resistance can be modeled using a subgrade modulus approach (Terzaghi 1955, Broms 1964) or the p-y approach (Matlock 1970, Reese et al. 1974, 1975, Reese and Welch 1975). Because the p-y approach has shown good correlation to field testing, it more accurately represents actual ground response, thus eliminating some of the uncertainty introduced by using approximations associated with the subgrade modulus approach (Rachel 2003). This chapter centers around the p-y approach using the methods developed by Matlock (1970), Reese et al. (1974, 1975), and Reese and Welch (1975). The specific methodology used for a given soil type subjected to the presence or absence of freewater represents an empirical relationship derived from classical methods and experimental research (Matlock 1970, Reese et al. 1974, 1975, Reese and Welch 1975). Due to their predominant presence in Southeast Louisiana, and other areas with similar foundation conditions, p-y curves constructed for soft
soils in the presence of freewater, stiff clays both in the presence and absence of freewater, and sands both in the presence and absence of freewater are emphasized in this discussion. Silts ($c$-$\phi'$ soils) are also of interest, but in view of the lack of experimental calibration it is assumed they can be modeled either as clay or sand for lateral resistance computations, depending upon expected behavior. This study utilizes static loadings, as recommended by USACE for pile group analysis of hurricane loadings (USACE 2008).

Regardless of soil type, construction of p-y curves (Figure 2.3) is based on an empirical fit corresponding to observed behavior during field tests (Matlock 1970, Reese et al. 1974, 1975, Reese and Welch 1975) and is dependent upon the ultimate lateral soil resistance. Ultimate lateral soil resistance is dependent on depth due to the two observed failure mechanisms of the soil mass: (1) a multiplane wedge type failure in which the soil shears forward and upward, occurring at or near the ground surface or (2) a plastic flow around the pile along only horizontal planes, that occurs at depths at which the soil mass is constrained and unable to fail upward.

For clays, the ultimate lateral resistance is a function of the undrained shear strength, $s_u$, the pile diameter (or width), $B$, and a non-dimensional ultimate resistance coefficient, $N_p$. The
parameter $N_p$ is dependent on (1) the relative stiffness of the clay (i.e., on whether the clay is considered “soft” or “stiff”), (2) the depth at which the lateral resistance is being computed, and (3) the presence or absence of freewater. Additional parameters used to build the p-y curve include a constant $J$, which is dependent upon stiffness of clay, the strain corresponding to one-half the maximum principal stress difference, $\varepsilon_{50}$, the initial spring constant $k_s_{clay}$ (stiff clays), and the empirical adjustment factor $A_{clay}$ (stiff clays).

For sands, the ultimate lateral resistance is a function of $\gamma_{soil}$, $\gamma_{GWT}$ (if submerged), $B$, and $z_i$, as modified by adjustment factors derived from the geometry of the failure surface. Both adjustment factors are tied to $\phi'$ and the at-rest pressure coefficient, $K_o$. Spring constant $k_s_{sand}$, a function of relative density, and the empirical adjustment factors $A_{sand}$ and $B_{sand}$, both functions of $B$ and $z_i$, are used to build the p-y curve.

2.2.1.3 Axial Pile Capacity

Steel H-piles are common in HSDRRS structures and are therefore emphasized in this discussion. Axial pile capacity is a function of the frictional resistance afforded by soil-pile interaction along the perimeter area of the pile shaft, and of the end bearing of the pile in the stratum at the pile tip. Axial pile capacity can be computed as (USACE 1991, Cherubini and Vessia 2005, 2007):

$$Q_u = \sum_{i=1}^{N} [K_j \sigma' \tan \delta + \alpha_{pile} c_{u,i}] A_{u,i} + qA_{tip}$$

(2.1)

where:

$Q_u =$ ultimate axial geotechnical resistance

$A_{u,i} =$ surface area of the pile in contact with the soil for the $i$th stratum,

$A_{tip} =$ effective (gross) area of the pile tip in contact with the soil at the tip,
\( \alpha_{\text{pile}} = \text{adhesion factor}, \)

\( c_u = \text{cohesion intercept for } c-\phi \text{ soils}; c_u = s_u \text{ for clays in undrained conditions.} \)

\( K_j = \text{lateral earth pressure coefficient for the } i^{th} \text{ stratum (} j \text{ is either “c” for compression piles or “t” for tension piles).} \)

\( \sigma'_v = \text{effective overburden pressure}, \)

\( \delta = \text{angle of friction between the soil and pile, and} \)

\( q = \text{unit tip bearing capacity (} q = N_p c_u \text{ for clays and } q = N_q \sigma'_v \text{ for sands).} \)

For H-piles, the theoretical shaft resistance is computed assuming flange-to-soil contact on the top and bottom flanges and soil-to-soil contact along each side of the pile along the web. In other words, the external “square” perimeter of the pile is commonly used rather than the full surface area of the H-pile, based upon the assumption that a plug of soil will form between the flanges during driving as illustrated by Figure 2.4. The area considered for tip bearing of H-piles, however, is either taken as the area of steel or a percentage of the total “block” area. For HSDRRS structures, USACE (2008) recommends 60% of the block area.

Figure 2.4 H-pile soil plug and critical parameters.
For clays and silts, $\alpha_{pile}$ is determined by USACE (1991) in accordance with the API 16th Edition of RP 2A (API 1986) relationship of adhesion to shear strength. In sands, the shaft resistance is a function of $K_j$, $\sigma'_v$, and $\delta$. The lateral earth pressure coefficient, $K_j$, is a constant dependent upon soil type and on whether the pile is in tension or compression. Note that $\sigma'_v$ is a function of $\gamma_{soil}$, $\gamma_{GWT}$, and depth, $z$, with an upper limit dependent upon either the critical depth, which is a function of relative density (USACE 1991) or a limiting value of 168 kPa (3,500 psf) (USACE 2008). The angle of friction between the soil and pile, $\delta$, is a function of $\phi'$. For the H-pile block perimeter, half the frictional resistance is based upon soil-on-soil contact (meaning $\delta = \phi'$).

The unit tip bearing capacity for clays uses $N_p = 9$. For sands and silts, the bearing capacity factor, $N_q$, is a function of $\phi'$.

### 2.2.2 Identification of Structural Resistance Parameters

The structural contributions to the performance of the T-wall are the structural resistance provided by the steel H-piles, the reinforced concrete base slab, the reinforced concrete stem, and the steel tension connectors linking the steel H-piles to the reinforced concrete base slab. For the reinforced concrete components, flexure and flexural shear are of primary concern, except at the pile-to-base slab connection where punching shear and pullout are considered. The prominent modes of failure for the steel H-piles are axial compression, axial tension, flexure, shear, and combinations thereof, although, typically, axial geotechnical capacity governs the design.

#### 2.2.2.1 Reinforced Concrete Stem and Base Slab

The reinforced concrete elements for floodwalls are primarily designed as flexural members, meaning that shear and flexural resistance are of primary concern. The parameters of interest in accounting for uncertainty in material properties are the compressive strength of concrete, $f'_c$ and the yield strength of the reinforcement, $f_y$. The parameters of interest in
addressing uncertainty in fabrication are the effective depth of reinforcement, \(d\), the area of reinforcement, \(A_s\), and the width of the section, \(b\).

2.2.2.2 Steel H-piles and Tension Connectors

Structural steel elements for floodwalls include the steel H-piling and tension connectors. These elements must resist axial compression, axial tension, flexure, shear, and combined loadings. The parameters of interest in accounting for uncertainty in material properties are the specified minimum tensile \((F_u)\) and yield \((F_y)\) stresses of the type of steel being used, and the Modulus of Elasticity, \(E_{steel}\). Geometrical parameters and ratios of interest for H-piles include the gross section area, \(A\); the overall section depth, \(d\); the flange width, \(B\); the radii of gyration about both axes, \(r_x\) and \(r_y\); the radius of gyration of the compression flange and 1/3 of the compression web area, \(r_T\); the flange thickness, \(t_f\); the web thickness, \(t_w\), the ratio of overall section depth to area of the flange, \(d/A_f\); the moments of inertia about both axes, \(I_x\) and \(I_y\); and the torsional constant, \(J\).

2.2.3 Identification of Load Parameters

Loads of interest for T-walls include the dead load of the reinforced concrete superstructure, the weight of any soil present atop the floodwall base slab (treated separately from the loads considered in the previous global stability discussion), uplift forces due to the static storm surge elevation, the weight of the static storm surge bearing on the reinforced concrete base slab, and lateral loads imparted by soil, storm surge, and waves. Although impact forces and wind also contribute to the design of T-walls, this study emphasizes the hydraulic loadings. Consequently, impact and wind loadings are not discussed.

2.2.3.1 Dead Load

The magnitude of the dead loads experienced by the T-wall are dependent on the unit weight of the materials (concrete and soil), the actual dimensions of the reinforced concrete
sections and, in the case of soil bearing on the base slab, the relative position of either face of the floodwall stem with respect to the edge of the slab. The dimensional component of the reinforced concrete sections has been addressed above in the structural resistance discussion. Given its typically small variability, the relative location of the stem is recommended for treatment as a constant. Of the two remaining parameters, the unit weight of soil has previously been discussed, leaving only the unit weight of reinforced concrete.

2.2.3.2 Hurricane Storm Surge Loads

The three critical applied loadings directly resulting from storm surge are hydrostatic storm surge loads, wave loads, and uplift due to the static head induced by the storm surge.

Storm surge elevations, significant wave heights, and wave periods may be obtained from existing datasets, such as the dataset reported by USACE (2007). The dataset was developed using the Joint Probability Method – Optimum Sampling (JPM-OS) (USACE 2007, IPET 2009, Resio et al. 2009), which built upon the work by Niedoroda et al. (2008). When existing datasets are used, a conditional probability of failure for a given recurrence interval can be computed.

The JPM-OS method, as currently applied, requires the use of multiple numerical models, each with its own unique contribution (WAMBDI-Group 1988, Thompson and Cardone 1996, Smith et al. 2001, Luettich and Westerink 2006, USACE 2007). The method uses historical data to develop statistical characterizations of storm parameters, which are then fit by probability distribution functions (Niedoroda et al. 2008). A set of synthetic storms is then developed using conventional JPM methods and the optimum sampling algorithm is then used to reduce sample size. Parameters from the synthetic storms are then run through the suite of numerical models to develop storm surge elevations and maximum wave characteristics for specific points approximately 183 m (600 ft) in front of the levee or floodwall, which are then used for frequency analysis. From the resulting frequency curves, the best estimates of storm surge
elevation and maximum wave characteristics are selected based upon the required design return period. For the Greater New Orleans HSDRRS, the 1% annual probability of exceedance is used for selection of best estimate of hurricane storm surge elevation. For design, the 90% confidence limit of the 1% best estimate is used. Using the resulting surge elevation, hydrostatic pressures are computed for gravity load, uplift, and lateral pressure on the structure. The wave characteristics in conjunction with the relationships in USACE (2006) are used to develop wave loads. In the Greater New Orleans HSDRRS, expected values of peak period computed were between 2.0 and 9.5 seconds, expected values of breaking wave height were between 0.305 and 2.408 m (1.0 and 7.9 feet), and the resulting expected values of wave force computed by (USACE 2006) were between 0 and 229 kN/m (0 and 15,700 lbs/ft) of structure (USACE 2007).

### 2.3 Independent Variables, Dependent Variables, and Constants

Given the varying degrees of uncertainty with the large number of parameters required to compute the reliability of the T-wall, careful consideration must be given to whether a given parameter should be treated as an independent random variable, a dependent variable whose value varies based on the value of an independent random variable, or a constant. The decision whether to represent a given parameter as a variable or a constant may be based upon its impact on system reliability due to its degree of variability or its role in the model. In instances where there is no statistical data available or in instances where a parameter is a term of lower order than other critical parameters, the decision may be made to treat it as a constant. In other situations, there may be a clear relationship between design parameters by virtue of which one may be treated as a dependent variable whose value is tied to an independent variable; such is the case for $\varepsilon_{50}$ and $c_u$. Care must be taken when making these assignments, to assure that a critical source of uncertainty is not neglected, thus skewing the results of the reliability analysis.
2.3.1 Geotechnical Variables and Constants

For T-walls, USACE methodologies and production software influence the selection of random variables. The use of the methods described in EM 1110-2-2906 (USACE 1991) for axial pile capacity, of SLOPE/W (Geo-Slope 2008) for stability analysis, and of GROUP (ENSOFT 2006) for pile group analysis (lateral pile resistance), results in the selection of a set of parameters as variables that differs from sets selected by other methods and software. Table 2.1 presents all relevant parameters, many of which are not discussed in detail here, but are discussed in the literature. Pile spacing, embankment geometry, and depth, \( z_i \), are treated as constants here. Given its relatively small range of values, \( \gamma_{GWT} \) is also treated as a constant.

Table 2.1 Geotechnical resistance variables and constants/dependent variables.

<table>
<thead>
<tr>
<th>Limit State</th>
<th>Random Variables</th>
<th>Constants/Dependent Variables</th>
</tr>
</thead>
<tbody>
<tr>
<td>Global Stability</td>
<td>( s_w ), ( \phi' ), ( \gamma_{soil} )</td>
<td>( \beta, R, \alpha, x, f, e, d, X_L, X_R, \gamma_{GWT}, z_i )</td>
</tr>
<tr>
<td>Lateral Soil/Pile Resistance</td>
<td>( s_w ), ( \phi' ), ( \gamma_{soil} ), ( B )</td>
<td>( \varepsilon_{50}, A_{clay}, A_{sand}, B_{sand}, J, K_o, k_s_{clay}, k_{s_{sand}}, R_g, s, \gamma_{GWT} )</td>
</tr>
<tr>
<td>Axial Pile Resistance-Skin Friction</td>
<td>( s_w ), ( \phi' ), ( \gamma_{soil} ), ( B, d_{pilie} )</td>
<td>( N_p, K_C, K_T, \delta, \alpha_{pile}, \gamma_{GWT}, z_i )</td>
</tr>
<tr>
<td>Axial Pile Resistance-End Bearing</td>
<td>( s_w ), ( \phi' ), ( \gamma_{soil} ), ( B, d_{pilie} )</td>
<td>( N_p, N_q, \gamma_{GWT}, z_i )</td>
</tr>
</tbody>
</table>

In global stability computations, the analysis is performed iteratively for multiple failure surfaces. The geometrical terms (\( \beta, R, \alpha, x, f, e, d \)) and interslice forces (\( X_L, X_R \)) vary with each failure surface iteration (Geo-Slope 2008) and are referred to as “moving constants”. Pore water pressure and slice weight also factor directly into determining the unbalanced load that must be resisted by the foundation, but are dependent upon material properties \( \gamma_{GWT} \) and \( \gamma_{soil} \), respectively, and on geometrical parameters such as depth of failure plane and thickness of stratum, both represented by variations of \( z_i \).

Parameters specific to lateral pile resistance using p-y methods \( A_{clay}, A_{sand}, B_{sand}, J, K_o, k_s_{clay}, k_{s_{sand}} \), are recommended for treatment as constants or dependent variables, which corresponds well with the use of the software package GROUP (ENSOFT 2006), which
computes these internally. The treatment of these terms as constants is supported by the parametric study performed by Meyer and Reese (1979), wherein values of $k_{s\text{clay}}$ and $k_{s\text{sand}}$ were varied for both soil types and $K_o$ was varied for sand. The results of the study demonstrated that the values of $k_{s\text{clay}}$, $k_{s\text{sand}}$, and $K_o$ had relatively small effects on pile behavior, especially when compared to the $s_u$, $\phi'$, $\varepsilon_{50}$ for clays, and $\gamma_{\text{soil}}$. It is proposed that $\varepsilon_{50}$ be treated as a dependent variable using correlations with $s_u$ (which equals the cohesion intercept, $c$) presented by Meyer and Reese (1979), due to the preponderance of instances wherein $\varepsilon_{50}$ is not reported in lab analyses.

Since $s/B$ ($s =$ pile spacing) must be considered to account for the effects of leading, trailing, and side-by-side piles (as well as “skewed” or diagonal piles) on one another, group reductions to lateral pile resistance, $R_g$, will be dependent upon $B$. For capacity reductions, the relationships utilized by GROUP (Prakash 1962, Schmidt 1981, Cox et al. 1984, Schmidt 1985, Wang 1986, Brown et al. 1987, Lieng 1988) are proposed for use.

Given the dependence of $\alpha_{\text{pile}}$ upon $c_u$, and the dependence of both $\delta$ and $N_q$ upon $\phi'$, it is recommended that all three parameters be treated as dependent variables. The USACE relationships for determination of $\alpha_{\text{pile}}$ (USACE 1991) and $\delta$ (USACE 2008) are recommended. The relationship presented by Terzaghi et al. (1996) for determining $N_q$ is recommended for use, even though it represents a lower bound for values, as shown in Coduto (1994) and Prakash and Sharma (1990).

For geotechnical resistance, the parameters recommended for treatment as random variables are $s_u$ (or $c_u$), $\phi'$, $\gamma_{\text{soil}}$, $d_{\text{pile}}$ and $B$. The three geotechnical parameters: $s_u$ (or $c_u$), $\phi'$, and $\gamma_{\text{soil}}$, play prominent roles in all geotechnical demand and resistance computations. Global stability, lateral pile capacity, and axial pile capacity are all directly related to these terms.
Furthermore, the two structural parameters factor into both geotechnical resistance and structural resistance.

2.3.2 Structural Variables and Constants

Given the availability of statistics on material properties for concrete and steel (structural steel shapes and concrete reinforcement), $f_y$, $f'c$, $F_y$, $E_{steel}$, $A_s$, and structural steel dimensions can be treated as variables or constants. Concrete dimensions are highly subject to individual workmanship and as such are difficult to statistically quantify. However, some statistical values have been presented in the literature, which are captured in Table 2.3. Typically, plane strain models are used to depict structures such as T-walls and levees. Unless specifically addressing uncertainty due to out-of-tolerance installation of piles in the out-of-plane direction, the unit width can be treated as a constant.

2.3.3 Load Variables and Constants

The unit weight of soils and water were addressed previously. Concrete dead load is dependent upon unit weight and member dimensions. As such, it is recommended that unit weight be treated as a variable.

Given the probabilistic methodology utilized to determine the critical hydraulic parameters, where datasets that provide parameters for a given exceedance event (such as the 2%, 1%, 0.5%, or 0.2% storm surge exceedance event) are readily available, it is recommended to make use of that data for production-oriented models. Nominal values provided from available datasets can be utilized directly. If mean and standard deviations for the best estimate values are available, uncertainty associated with modeling error can be quantified. For more detailed reliability models, either additional surge and wave modeling or additional data reduction can be performed to develop the required statistics.
One such dataset exists for the Greater New Orleans HSDRRS and can be found in USACE (2007). It contains computed means and standard deviations for storm surge elevation best estimates and means for wave parameters. Standard deviations based upon expert judgment for significant wave height and peak period, respectively, are given as 10% of the significant wave height and 20% of the peak period values. Per USACE (2007), the error in all estimates is assumed to be normally distributed. For the USACE dataset, however, it should be noted that: (1) the correlation between the water elevation and wave characteristics is not taken into account (USACE 2007); (2) the maximum storm surge and wave is assumed to occur simultaneously for all locations; and (3) a breaker parameter is used to account for the inevitable reduction in height of waves at the toe of levees and floodwalls compared to the heights provided for points 183 m (600 ft) away from the shore as output by STWAVE (USACE 2007). Typical values for this parameter and guidance for use of these values is described by IPET (2009) and USACE USACE (2006).

2.4 Uncertainty

2.4.1 Uncertainty in Geotechnical Resistance and Load Parameters

Phoon and Kulhawy (1999a) describe three main sources of uncertainty in the establishment of geotechnical parameters for design: (1) the inherent variability of the soil, (2) measurement error, and (3) transformation uncertainty. The soil profile at a given site is the end result of natural geologic processes that produce the local stratigraphy and ultimately cause the evolution of the in situ soil mass. Given the non-uniformity of these processes, there is an inherent variability in the properties of a given strata even across a single project location. Uncertainties due to measurement error are introduced as a limited number of samples are obtained, transported, and laboratory-tested to derive the raw data provided to designers. Equipment, operators, testing procedures, and random testing effects are causes of measurement
error. In situ testing also has its own contributions to measurement error, but this chapter will focus on laboratory testing. The third source of uncertainty results from the translation of the raw data into a usable form by individual designers.

2.4.2 Uncertainty in Structural Resistance and Load Parameters

Similar to the geotechnical parameters, the main sources of uncertainty in establishing the structural resistance of the system are tied to material properties, the structural analysis models used to compute resistance, and the variability of dimensions (Ellingwood et al. 1980). In performing structural reliability analyses, the common method for modeling resistance and taking these uncertainties into account is through the relationship:

\[ R = R_n M F P \]  

(2.2)

where:

\( R \) = resistance,

\( R_n \) = nominal resistance computed using code equations,

\( M \) = factor accounting for uncertainties in materials

\( F \) = factor accounting for uncertainties in fabrication, and

\( P \) = factor accounting for analysis methods/professional judgment.

Of particular interest in structural reliability is the bias factor, \( \lambda \), or the ratio of mean-to-nominal values for design parameters. The bias factor shows the relationship of observed values to theoretical values and is utilized in the literature as a convenient way to make the statistics applicable to a range of situations (Ellingwood et al. 1980).

2.4.3 Geotechnical Parameters

As evidenced by the literature, when treating the stability of slopes (VanMarcke 1980, Griffiths and Fenton 2004, IPET 2009) or axial resistance of piles (Lacasse and Goulois 1989,
Lacasse and Nadim 1996, Cherubini and Vessia 2005, 2007) probabilistically, the variability of shear strength plays a prominent role in determining reliability.

Given the short term nature of a hurricane loading, for slope stability analysis the HSDRRS Design Guidelines require the use of undrained shear strengths as total stresses for slow draining soils and drained strengths as effective stresses for free-draining materials (depending upon interconnectivity with the hurricane storm surge). For silts Duncan and Wright (2005) recommend consideration of both drained and undrained conditions, given the wide range of behavior. In the case of silts, this could require consideration of the correlation between effective cohesion, \( c' \), and angle of internal friction of the soil, \( \phi' \). As cited by Cherubini (2000), this is typically a negative correlation. Cherubini (2000) showed that inclusion of the negative correlation resulted in higher values of reliability index, so not considering this correlation could result in conservative values of reliability index and probability of failure for each foundation element.

Use of the combination of undrained and drained strengths is also used for establishing soil resistance to lateral loadings during pile group analysis. Pile axial capacity computations are required to consider both undrained (Q-Case) and drained (S-Case) conditions (USACE 2008).

Table 2.2 presents values of point COVs (which is a measure of the variability at any given point in space) used by various researchers (COV values presented have not been adjusted for measurement error). As seen from the table, the COV values vary from 5% to 62% for \( s_u \) of clays and from 7% to 20% for \( \phi' \). Phoon and Kulhawy (1999b) reported COV of inherent variability for clays to range between 10% and 30% for laboratory unconsolidated-undrained (UU) triaxial compression tests with a COV of measurement error on the order of 5% to 15%.

The same relationships used for the \( s_u \) of clays can be used for the calculation of COV for \( \phi' \) by substituting the appropriate \( \phi' \) values in lieu of the \( s_u \) values. Phoon and Kulhawy (1999b)
reported point COVs of inherent variability and measurement error for $\phi'$ were both within the range of 5% and 15%.

Table 2.2  Geotechnical coefficients of variation.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>Testing Method</th>
<th>Parameter of Interest</th>
<th>Point Coefficient of Variation, COV</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay</td>
<td>UC(^a) (lab)</td>
<td>$s_u$</td>
<td>0.20 – 0.55</td>
<td>(Phoon and Kulhawy 1996, 1999a)</td>
</tr>
<tr>
<td>Clay</td>
<td>UU(^b) (lab)</td>
<td>$s_u$</td>
<td>0.10 – 0.30</td>
<td>(Phoon and Kulhawy 1999a)</td>
</tr>
<tr>
<td>Clay</td>
<td>q(_u) (CPT(^c))</td>
<td>$s_u$</td>
<td>0.20 – 0.40</td>
<td>(Phoon and Kulhawy 1999a)</td>
</tr>
<tr>
<td>Sand, Clay</td>
<td>Direct (lab)</td>
<td>$\phi'$</td>
<td>0.05 – 0.15</td>
<td>(Phoon and Kulhawy 1999a)</td>
</tr>
<tr>
<td>Sand</td>
<td>$q'_T$ (CPT(^d))</td>
<td>$\phi'$</td>
<td>0.20 – 0.60</td>
<td>(Phoon and Kulhawy 1999a)</td>
</tr>
<tr>
<td>Clay, Silt</td>
<td>Not Given</td>
<td>$\gamma_{soil}$</td>
<td>0.00 – 0.10</td>
<td>(Lacasse and Nadim 1996, Phoon and Kulhawy 1999a, Nadim 2007)</td>
</tr>
<tr>
<td>Clay</td>
<td>UU(^b) (lab)</td>
<td>$s_u$</td>
<td>0.10 – 0.35</td>
<td>(Phoon and Kulhawy 1996)</td>
</tr>
<tr>
<td>Clay</td>
<td>$q'_T$ (CPT(^d))</td>
<td>$s_u$</td>
<td>0.30 – 0.40(^e)</td>
<td>(Phoon and Kulhawy 1996)</td>
</tr>
<tr>
<td>Sand, Clay</td>
<td>Direct (lab)</td>
<td>$\phi'$</td>
<td>0.07 – 0.20,</td>
<td>(Phoon and Kulhawy 1996)</td>
</tr>
<tr>
<td>Sand</td>
<td>$q'_T$ (CPT(^d))</td>
<td>$\phi'$</td>
<td>0.10 – 0.15(^e)</td>
<td>(Phoon and Kulhawy 1996)</td>
</tr>
<tr>
<td>Clay</td>
<td>TC(^d)</td>
<td>$s_u$</td>
<td>0.05 – 0.20</td>
<td>(Lacasse and Nadim 1996, Nadim 2007)</td>
</tr>
<tr>
<td>Clayey Silt</td>
<td>Not Given</td>
<td>$s_u$</td>
<td>0.10 – 0.30</td>
<td>(Lacasse and Nadim 1996, Nadim 2007)</td>
</tr>
<tr>
<td>Sand</td>
<td>Not Given</td>
<td>$\phi'$</td>
<td>0.02 – 0.05</td>
<td>(Lacasse and Nadim 1996, Nadim 2007)</td>
</tr>
<tr>
<td>Sands</td>
<td>Not Given</td>
<td>$\gamma_{soil}$</td>
<td>0.00 – 0.10</td>
<td>(Lacasse and Nadim 1996, Nadim 2007)</td>
</tr>
<tr>
<td>Monta Blue Clay</td>
<td>Not Given</td>
<td>$s_u$</td>
<td>0.33</td>
<td>(Cherubini and Vessia 2005)</td>
</tr>
<tr>
<td>Monta Blue Clay</td>
<td>Not Given</td>
<td>$\gamma_{soil}$</td>
<td>0.05</td>
<td>(Cherubini and Vessia 2005)</td>
</tr>
<tr>
<td>Clay</td>
<td>Not Given</td>
<td>$s_u$</td>
<td>0.13 – 0.40</td>
<td>(Duncan 2000)</td>
</tr>
<tr>
<td>Lacustrine Clay</td>
<td>Field Vane Tests</td>
<td>$s_u$</td>
<td>0.27</td>
<td>(Christian et al. 1994)</td>
</tr>
<tr>
<td>Marine Clay</td>
<td>Field Vane Tests</td>
<td>$s_u$</td>
<td>0.18</td>
<td>(Christian et al. 1994)</td>
</tr>
<tr>
<td>Distributary Clay, Clay</td>
<td>UU(^b)</td>
<td>$s_u$</td>
<td>0.41 – 0.62</td>
<td>(IPET 2009)</td>
</tr>
<tr>
<td>Sands</td>
<td>Not Given</td>
<td>$\gamma_{soil}$</td>
<td>0.05 – 0.20</td>
<td>(Chalermyanont and Benson 2005)</td>
</tr>
<tr>
<td>Sands</td>
<td>Not Given</td>
<td>$\phi'$</td>
<td>0.05 – 0.20</td>
<td>(Chalermyanont and Benson 2005)</td>
</tr>
<tr>
<td>Clay, Silt, Sands</td>
<td>Not Given</td>
<td>$\gamma_{soil}$</td>
<td>0.03 – 0.07</td>
<td>(Duncan 2000)</td>
</tr>
<tr>
<td>Sands</td>
<td>Not Given</td>
<td>$\phi'$</td>
<td>0.02 – 0.13</td>
<td>(Duncan 2000)</td>
</tr>
</tbody>
</table>

\(^a\) unconfined compression test  
\(^b\) unconsolidated-undrained triaxial compression test  
\(^c\) cone penetrometer test  
\(^d\) triaxial compression test  
\(^e\) function of mean

and Roh 2008). Distributions for $\phi'$ and $f_{\text{soil}}$ are commonly taken to be normal (Lacasse and Nadim 1996, Chalermyanont and Benson 2005, Nadim 2007).

2.4.4 Structural Parameters

Values for COV and $\lambda$ used for material properties and dimensional properties used in other studies are presented in Table 2.3. The values for material strength are based upon the assumption that the concrete used for most structures not specifically categorized as mass-concrete structures falls into the ready-mix category.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$\lambda$</th>
<th>COV</th>
<th>Distribution</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>$f'_{c}$ (27.6 MPa, 4,000 psi)</td>
<td>1.0</td>
<td>0.18</td>
<td>Normal</td>
<td>(Ellingwood et al. 1980, Stewart and Rosowsky 1998)</td>
</tr>
<tr>
<td></td>
<td>1.235</td>
<td>0.145</td>
<td>Normal</td>
<td>(Nowak and Szersze 2003)</td>
</tr>
<tr>
<td>$f_{y}$ (413.7 MPa, 60,000 psi)</td>
<td>1.125</td>
<td>0.098</td>
<td>Beta</td>
<td>(Ellingwood et al. 1980)</td>
</tr>
<tr>
<td></td>
<td>1.145</td>
<td>0.05</td>
<td>Normal</td>
<td>(Nowak and Szersze 2003)</td>
</tr>
<tr>
<td></td>
<td>1.09</td>
<td>0.05</td>
<td>Lognormal</td>
<td>(Seo et al. 2010)</td>
</tr>
<tr>
<td>$A_{s}$</td>
<td>1.0</td>
<td>0.015</td>
<td>Not Specified</td>
<td>(Nowak and Szersze 2003)</td>
</tr>
<tr>
<td></td>
<td>1.01</td>
<td>0.04</td>
<td>Modified Lognormal</td>
<td>(Mirza and MacGregor 1982)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>(Adjustment Factor = 0.91)</td>
<td></td>
</tr>
<tr>
<td>$d$</td>
<td>0.99</td>
<td>0.04</td>
<td>Not Specified</td>
<td>(Nowak and Szersze 2003)</td>
</tr>
</tbody>
</table>

Dimensional variations of concrete have been shown to be dependent upon the overall dimensions of the specific member being investigated, which means that the COV varies with increasing member size (Ellingwood et al. 1980). In Ellingwood et al. (1980), values of construction error in slabs were based on slabs of thicknesses 102 to 254 mm (4 to 10 in). Table 2.4 presents uncertainty data for the variability in dimensions for reinforced concrete sections, focusing primarily on clear cover to top and bottom reinforcing steel and overall section thickness.

The professional factor (which addresses the uncertainty in the analysis model, as described previously) that is of most interest for hurricane risk reduction structures concerns
modeling beam shear and flexure. As noted in Nowak and Szersze (2003), values of bias factor and COV presented for reinforced concrete beam shear are 1.075 and 0.10, respectively. Values presented for flexure are 1.02 and 0.06. Alternatively, in the case of bridge T-beams, Nowak (1999) maintained the same values of bias factor for shear, the same values of COV for shear and flexure, but presented a value of 1.00 for bias factor for flexure.

Table 2.4  Variability in dimensions for reinforced concrete sections.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>μ, mm (in)</th>
<th>σ mm (in)</th>
<th>Distribution</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_{Top/PS}$ (Clear Cover to top reinforcement)</td>
<td>+19.8 (+0.78)</td>
<td>19.8 (0.78)</td>
<td>Normal</td>
<td>(Stewart and Rosowsky 1998)$^a$</td>
</tr>
<tr>
<td>$C_{Bot/PS}$ (Clear cover to bottom reinforcement)</td>
<td>+8.64 (+0.34)</td>
<td>10.4 (0.41)</td>
<td>Normal</td>
<td>(Stewart and Rosowsky 1998)$^a$</td>
</tr>
<tr>
<td>$h$ (Total section thickness)</td>
<td>-3.05 (-0.12) (Beam)</td>
<td>6.35 (0.25)</td>
<td>Normal</td>
<td>(Ellingwood et al. 1980, Mirza and MacGregor 1982, Seo et al. 2010)</td>
</tr>
<tr>
<td></td>
<td>+20.6 (+0.81) (Beam)</td>
<td>13.2 (0.55)</td>
<td>Normal</td>
<td>(Ellingwood et al. 1980)</td>
</tr>
<tr>
<td></td>
<td>+0.762 (+0.03) (Slab)</td>
<td>11.9 (0.47)</td>
<td>Normal</td>
<td>(Ellingwood et al. 1980)</td>
</tr>
<tr>
<td></td>
<td>+5.33 (+0.21)</td>
<td>6.60 (0.26)</td>
<td>Normal</td>
<td>(Ellingwood et al. 1980)</td>
</tr>
<tr>
<td></td>
<td>+.762 (+0.03)</td>
<td>11.9 (0.47)</td>
<td>Normal</td>
<td>(Stewart and Rosowsky 1998)$^a$</td>
</tr>
</tbody>
</table>

$^a$ Bridge deck slab.

Values for COV and $\lambda$ used for steel material properties and dimensional properties found in the review are presented in Table 2.5. The values for material strength are based upon the use of ASTM A 992 steel sections (or ASTM A 572 Material) where specifically identified. For the sources that identified ASTM shape groups, data for Group 2 was presented.

For the statistics of fabrication, Ellingwood et al. (1980) and Galambos (2004) observed that the bias factor for hot-rolled sections was equal to 1.0 with a COV of 0.05. This is consistent with the assumptions made by Kala and Kala (2005) where nominal values of section geometry accounting for tolerances and distortions of the steel section were equal to mean values, and 95% of all values measured met the limitations set forth in the governing standard.
Table 2.5  Variability in material properties for steel sections.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$\lambda$</th>
<th>COV</th>
<th>Distribution</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>$F_{y, flg}$</td>
<td>1.05</td>
<td>0.10</td>
<td>Lognormal</td>
<td>(Ellingwood et al. 1980)</td>
</tr>
<tr>
<td></td>
<td>1.06</td>
<td>0.06</td>
<td>Lognormal</td>
<td>(Galambos 2004)</td>
</tr>
<tr>
<td></td>
<td>1.11</td>
<td>0.058</td>
<td>Lognormal</td>
<td>(Bartlett et al. 2003)</td>
</tr>
<tr>
<td>$F_{y, web}$</td>
<td>1.10</td>
<td>0.11</td>
<td>Lognormal</td>
<td>(Ellingwood et al. 1980)</td>
</tr>
<tr>
<td>$F_{y, shear}$</td>
<td>1.11</td>
<td>0.10</td>
<td>Lognormal</td>
<td>(Ellingwood et al. 1980)</td>
</tr>
<tr>
<td>$E_{steel}$</td>
<td>1.00</td>
<td>0.06</td>
<td>Lognormal</td>
<td>(Ellingwood et al. 1980)</td>
</tr>
<tr>
<td></td>
<td>0.993</td>
<td>0.034</td>
<td>Lognormal</td>
<td>(Galambos 2004)</td>
</tr>
<tr>
<td>$F_{u,steel}$</td>
<td>1.10</td>
<td>0.11</td>
<td>Lognormal</td>
<td>(Ellingwood et al. 1980)</td>
</tr>
<tr>
<td></td>
<td>1.101</td>
<td>0.051</td>
<td>Lognormal</td>
<td>(Bartlett et al. 2003)</td>
</tr>
<tr>
<td>$F_{EXX, fillet weld}$</td>
<td>1.05</td>
<td>0.04</td>
<td>Lognormal</td>
<td>(Ellingwood et al. 1980)</td>
</tr>
<tr>
<td>$\tau_{u, fillet weld}$</td>
<td>0.84</td>
<td>0.10</td>
<td>Lognormal</td>
<td>(Ellingwood et al. 1980)</td>
</tr>
<tr>
<td>$P_{cr/FyA}$</td>
<td>0.936 – 0.239</td>
<td>0.02 to 0.08</td>
<td>Lognormal</td>
<td>(Ellingwood et al. 1980, Galambos 2004)</td>
</tr>
</tbody>
</table>

For the statistics of modeling, Ellingwood et al. (1980) and Galambos (2004) presented a bias factor of unity with no variation for the resistance model for tension members and welds. The statistics of other models are presented in Table 2.6.

Table 2.6  Variability in professional factor for steel sections.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>$\lambda$</th>
<th>COV</th>
<th>Distribution</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compact Wide Flange Sections (Flexure)</td>
<td>1.02</td>
<td>0.06</td>
<td>Lognormal</td>
<td>(Ellingwood et al. 1980)</td>
</tr>
<tr>
<td></td>
<td>1.02</td>
<td>0.08</td>
<td>Lognormal</td>
<td>(Galambos 2004)</td>
</tr>
<tr>
<td>Elastic Wide Flange Sections, Lateral Torsional</td>
<td>1.03</td>
<td>0.09</td>
<td>Lognormal</td>
<td>(Ellingwood et al. 1980)</td>
</tr>
<tr>
<td>Buckling</td>
<td>1.14</td>
<td>0.09</td>
<td>Lognormal</td>
<td>(Galambos 2004)</td>
</tr>
<tr>
<td>Beam Columns (Interaction Equations)</td>
<td>1.02</td>
<td>0.10</td>
<td>Lognormal</td>
<td>(Ellingwood et al. 1980)</td>
</tr>
<tr>
<td>Columns</td>
<td>1.03</td>
<td>0.05</td>
<td>Lognormal</td>
<td>(Ellingwood et al. 1980,</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>Galambos 2004)</td>
</tr>
</tbody>
</table>

2.4.5  Load Parameters

Ellingwood et al. (1980) and Nowak (1999) addressed dead loads generically as having a bias factor of 1.05 and a COV of 0.10 with a normal distribution. Patev and Leggett (1995) utilized a mean value of the unit weight of concrete equal to 22.8 kN/m³ (145 pcf) with a standard deviation of 0.785 kN/m³ (5 pcf) with a normal distribution. Duncan (2000) assumed a mean value of 23.6 kN/m³ (150 pcf) with a standard deviation of 0.314 kN/m³ (2 pcf). Thomos
and Trezos (2006) assumed a mean value of 23.9 kN/m³ (152 pcf) with a COV of 0.04 with a lognormal distribution.

2.5 Recommendations for HSDRRS Structure Statistical Parameters

The work of the researchers presented throughout this chapter have been evaluated based upon the current state of practice in both design and construction to develop the recommended values for coefficients of variation and means and standard deviations presented in Table 2.7. The table is limited to nominal values considered industry standard with respect to HSDRRS structures. In some instances, these recommendations utilize specific values provided by an individual researcher and in others a comprehensive range has been provided. These values are intended for use to validate statistical values computed based upon specific data for any given structure of interest, when available. When unavailable, judgment should be applied when selecting an appropriate value from the table. Where observed values are outside the ranges recommended, some adjustment may be required.

Although the literature did not specifically address the professional factor for concrete flexure or beam shear, either the normal or lognormal distributions are recommended for use, depending upon the assumptions used in a given model. Owing to the fact that most critical parameters in the concrete resistance model tend toward normal distributions, it is reasonable to assume that for most cases that the professional factor distribution is normal.

2.6 Conclusions

This chapter assembled and reviewed much of the available data needed for the reliability analysis of hurricane risk reduction structures. It also provided recommendations for representation of statistical parameters used for developing reliability models of structures composed of similar elements and structures similarly loaded.
### Table 2.7  
Recommended statistical representations.

<table>
<thead>
<tr>
<th>Material</th>
<th>Variable</th>
<th>Bias Factor (or Mean)</th>
<th>COV (Std. Deviation)</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay, Silts</td>
<td>$s_u, c_u$ (Lab Tests)</td>
<td>N/A</td>
<td>0.10 – 0.62</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Silts, Sands</td>
<td>$\phi$</td>
<td>N/A</td>
<td>0.02 – 0.20</td>
<td>Normal</td>
</tr>
<tr>
<td>Clay, Silts, Sands</td>
<td>$\gamma_{sild}$</td>
<td>N/A</td>
<td>0.00 – 0.10</td>
<td>Normal</td>
</tr>
<tr>
<td>Reinforced</td>
<td>$f'_c$ (4,000 psi)</td>
<td>1.235</td>
<td>0.145</td>
<td>Normal</td>
</tr>
<tr>
<td>Concrete</td>
<td>$f_c$ (60,000 psi)</td>
<td>1.145</td>
<td>0.05</td>
<td>Normal</td>
</tr>
<tr>
<td></td>
<td>$A_s$</td>
<td>1.00</td>
<td>0.015</td>
<td>Modified Log-Normal (Adjustment Factor = 0.91)</td>
</tr>
<tr>
<td></td>
<td>$d$</td>
<td>0.99</td>
<td>0.04</td>
<td>Normal</td>
</tr>
<tr>
<td></td>
<td>$C_{Top/PS}$</td>
<td>+19.8 mm (+0.78 in)</td>
<td>19.8 mm(0.78 in)</td>
<td>Normal</td>
</tr>
<tr>
<td></td>
<td>$C_{Bot/PS}$</td>
<td>+8.64 mm (+0.34 in)</td>
<td>10.4 mm (0.41 in)</td>
<td>Normal</td>
</tr>
<tr>
<td></td>
<td>$h$</td>
<td>-3.05 mm (-0.12 in)</td>
<td>6.35 mm (0.25 in)</td>
<td>Normal</td>
</tr>
<tr>
<td></td>
<td>$\gamma_{conc}$</td>
<td>23.6 kN/m$^3$ (150 pcf)</td>
<td>0.314 kN/m$^3$ (2 pcf)</td>
<td>Normal</td>
</tr>
<tr>
<td></td>
<td>$P$, Flexure</td>
<td>1.02</td>
<td>0.06</td>
<td>Normal or Lognormal$^a$</td>
</tr>
<tr>
<td></td>
<td>$P$, Beam Shear</td>
<td>1.075</td>
<td>0.10</td>
<td>Normal or Lognormal$^a$</td>
</tr>
<tr>
<td>Steel H-piles and Tension Connectors</td>
<td>$F_y, f_{y, shear}$</td>
<td>1.06</td>
<td>0.06</td>
<td>Lognormal</td>
</tr>
<tr>
<td></td>
<td>$F_y, web$</td>
<td>1.10</td>
<td>0.11</td>
<td>Lognormal</td>
</tr>
<tr>
<td></td>
<td>$E_{Steel}$</td>
<td>0.993</td>
<td>0.034</td>
<td>Lognormal</td>
</tr>
<tr>
<td></td>
<td>$F_{u, steel}$</td>
<td>1.101</td>
<td>0.051</td>
<td>Lognormal</td>
</tr>
<tr>
<td></td>
<td>$F_{EXX, fillet weld}$</td>
<td>1.05</td>
<td>0.04</td>
<td>Lognormal</td>
</tr>
<tr>
<td></td>
<td>$\tau_{wa, fillet weld}$</td>
<td>0.84</td>
<td>0.10</td>
<td>Lognormal</td>
</tr>
<tr>
<td></td>
<td>$P_c/F_{ydA}$</td>
<td>0.936 – 0.239</td>
<td>0.02 to 0.08</td>
<td>Lognormal</td>
</tr>
<tr>
<td></td>
<td>$A, d, B, r_y, r_f, r_t, l_w, d/A, I_x, I_y$, and $J$</td>
<td>1.00</td>
<td>0.05</td>
<td>Normal</td>
</tr>
<tr>
<td></td>
<td>$P$, Flexure Compact Wide Flange Sections</td>
<td>1.02</td>
<td>0.08</td>
<td>Lognormal</td>
</tr>
<tr>
<td></td>
<td>$P$, Flexure Elastic Wide Flange Sections, Lateral Torsional Buckling</td>
<td>1.14</td>
<td>0.09</td>
<td>Lognormal</td>
</tr>
<tr>
<td></td>
<td>$P$, Beam-Columns$^b$</td>
<td>1.02</td>
<td>0.10</td>
<td>Lognormal</td>
</tr>
<tr>
<td></td>
<td>Columns</td>
<td>1.03</td>
<td>0.05</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Storm Surge</td>
<td>$\eta$</td>
<td>In Dataset</td>
<td>In Dataset</td>
<td>Normal</td>
</tr>
<tr>
<td>(Error in Best Estimate)</td>
<td>$H_i$</td>
<td>In Dataset</td>
<td>0.10$H_s$</td>
<td>Normal</td>
</tr>
<tr>
<td></td>
<td>$T_p$</td>
<td>In Dataset</td>
<td>0.2$T_p$</td>
<td>Normal</td>
</tr>
</tbody>
</table>

$^a$These distributions were not provided in the literature, but are consistent with related parameters.

$^b$Interaction equations

From the literature, there is a longer history of employing probabilistic methods in the development of structural design codes, with some of the early work by Ellingwood et al. (1980) continuing to be considered state-of-the-art, as evidenced by the continuing use of the statistical data presented in his work. This chapter reviewed research that supported the updating and
calibration of reinforced concrete, structural steel, and bridge design codes. Values for bias factors and COVs were presented for material strengths, fabricated dimensions, and uncertainty in analysis models.

For production-oriented modeling, this chapter recommended the use of existing datasets (such as the one identified in USACE (2007)) and limiting reliability models to specific mean-recurrence interval storm surge exceedance events. This approach helps reduce the computational burden associated with additional surge and wave modeling or additional data reduction. This chapter does, however, recommend that consideration be given to model uncertainty and it provides recommendations on statistical parameters that might not otherwise be included in existing datasets.

The identification of independent variables and related statistical data presented in this chapter is useful in performing the critical first steps for the reliability analysis for pile-founded structures or any structure involving multiple disciplines in the development of designs. Hurricane risk reduction structures were used as a basis for development of the dataset presented, but the information provided may also be used for the development of discipline-specific or multi-discipline reliability models containing similar components or similar loading conditions, such as more complex HSDRRS structures, retaining walls, pile caps, footings, pile foundations, or reinforced concrete walls. Furthermore, this chapter serves as a useful resource for anyone researching statistical datasets for the design parameters described herein.
CHAPTER 3: CHARACTERIZING GEOTECHNICAL UNCERTAINTY FOR RELIABILITY ANALYSIS OF HURRICANE RISK REDUCTION STRUCTURES USING LIMITED DATASETS

3.1 Introduction

When considering the overarching probability of failure for an entire structure from the top of the superstructure to the deepest component of the foundation, a reasonably accurate probabilistic representation of the foundation is essential to produce meaningful results. Addressing and reducing uncertainty through determination of autocorrelation distances (i.e., scales of fluctuation), where possible, especially with respect to the foundation, is paramount in improving results. Christian and Baecher (2011) noted the difficulty in determining correlation patterns in the field, however, and this is accentuated when inadequate data are available. While aleatory uncertainty may be difficult to reduce in geotechnical design parameters, epistemic uncertainty can be reduced with additional sampling and testing (Christian 2004, Lacasse et al. 2007). In problems involving limited spatial domain (e.g., vertical construction) this may prove to be accomplished without incurring prohibitive costs. However, for horizontal construction over large spatial domains (e.g., hurricane risk reduction levees and floodwalls), the expense associated with additional sampling and testing required to substantially improve the understanding of uncertainty may not be justifiable, leaving the designer to utilize only typically available data.

This chapter provides insight into the suitability of design-quality field investigations to adequately account for uncertainty and spatial averaging effects for application to reliability analysis. This chapter adds to the existing body of knowledge with respect to inherent variability of unit weight, $\gamma_{soil}$, and undrained shear strength, $s_u$. It also adds data on scales of fluctuation through quantification of vertical and horizontal scales of fluctuation for $s_u$ of Holocene and Pleistocene clay deposits found in Southeast Louisiana. This chapter benefits geotechnical
engineers and researchers preparing to conduct reliability analysis on hurricane risk reduction levees and floodwalls designed with design-quality datasets, which are typically limited in comparison to research-quality. The results are more broadly applicable to other horizontal construction projects over large spatial domains (e.g., highways, bridges, and riverine flood protection features) in geological settings with highly stratified soils originating from a combination of fluvial, deltaic, and coastal deposits, similar to Southeast Louisiana (LGS 2008).

In this chapter, the scales of fluctuation using strength data obtained from unconfined compression tests (UC) and unconsolidated-undrained triaxial compression tests (UU) performed on samples obtained from 127 mm (5 in) diameter Shelby tubes as opposed to the more commonly reported values that rely on cone penetrometer tests (Alshibli et al. 2011). The 127 mm (5 in) diameter Shelby tubes are used by the U.S. Army Corps of Engineers as the basis for Hurricane and Storm Damage Risk Reduction System designs in the Greater New Orleans Area (USACE 2008). The computed scales of fluctuation are compared to values found in the literature and recommendations on the suitability of results contained in this chapter can serve as a guide for other researchers preparing to conduct reliability analyses in the face of limited datasets. Finally, the variance reductions described by Vanmarcke (1977), Phoon and Kulhawy (1999a, 1999b), and Jaksa, Kaggwa et al. (2000) are applied to coefficients of variation for $s_u$ and compared to unreduced values.

3.1.1 Uncertainty in Geotechnical Parameters

The main sources of uncertainty in geotechnical parameters are (1) the inherent variability of the soil, (2) measurement error, (3) statistical uncertainty, and (4) transformation model uncertainty (Christian et al. 1994, Phoon and Kulhawy 1999a, IPET 2009). The soil profile at a given site is the end result of natural geologic processes that produce the local stratigraphy and ultimately cause the evolution of the in situ soil mass. Given the non-uniformity
of these processes, there is an inherent variability in the properties of a given stratum even across a single project location. Uncertainties due to measurement error are introduced as a limited number of samples are obtained, transported, and laboratory-tested to derive the raw data provided to designers (De Groot et al. 2005). Equipment, operators, testing procedures, and random testing effects are some causes of measurement error (Lacasse et al. 2007). Statistical error is introduced from the use of small samples and can be reduced by increasing the number of samples. Model uncertainty results from the translation of the raw data into a usable form by individual designers using theoretical models that were derived using limited datasets and assumed approximations.

IPET (2009) and Phoon and Kulhawy (1999a) presented approaches to address geotechnical uncertainty. To properly quantify inherent variability, any deterministic trends in the data should be removed. However, as stated by Phoon and Kulhawy (1999a), if the sampling interval is sufficiently small, the depth variation may not be that great, which means that the coefficient of variation \((COV)\) of the dataset can be used as a rough approximation of the \(COV\) of inherent variability, \(COV_w\). IPET (2009) utilized an indirect method to estimate measurement error based on De Groot and Baecher (1993) and a simplified relationship linking an estimate of statistical error variance to the number of samples. Phoon and Kulhawy (1999a), however, noted that the statistical error component is commonly included in the measurement error term. Transformation uncertainty is difficult to quantify because rigorous statistics are not available (Phoon and Kulhawy 1999b). IPET (2009), however, did partially address transformation uncertainty by assessing the measurement bias associated with legacy test data for undrained shear strengths in conjunction with triaxial compression tests and cone penetrometer test (CPT) data. Because of the emphasis on production-oriented modeling, this chapter utilizes the more generalized approach presented by Phoon and Kulhawy (1999b).
Phoon and Kulhawy (1999b) treated soil profiles as a random field and presented second-moment probabilistic relationships to account for uncertainty. For hurricane risk reduction structures, parameters of most concern are $\gamma_{soil}$, $s_u$, and angle of internal friction, ($\phi'$). Using $s_u$ as an example, the spatial average for the design parameter of consideration can be derived from the relationship:

$$COV_{\gamma_{da}}^2 = \Gamma^2(L)COV_{w}^2 + COV_{e}^2 + COV_{\varepsilon}^2$$

(3.1)

where:

- $COV_{\gamma_{da}} = \text{spatially averaged coefficient of variation for the design parameter}$
- $\Gamma^2(*) = \text{variance reduction function for spatial averaging (if used)}$,
- $L = \text{averaging length}$
- $COV_w = \text{coefficient of variation of inherent soil variability,}$
- $COV_e = \text{coefficient of variation of measurement error, and}$
- $COV_{\varepsilon} = \text{coefficient of variation for transformation model uncertainty.}$

Reported values of $COV_w$ are between 0 and 0.10 for $\gamma_{soil}$ of clays, sands, and silts; between 0.10 and 0.62 for inherent soil variability of $s_u$ of clays and silts; and between 0.02 and 0.2 for $\phi'$ for sands and silts as noted in the literature (Christian et al. 1994, Lacasse and Nadim 1996, Phoon and Kulhawy 1996, 1999a, Duncan 2000, Chalermyanont and Benson 2005, Cherubini and Vessia 2005, Nadim 2007, Alshibili et al. 2009, IPET 2009, Alshibili et al. 2011, Dunn et al. 2012) and described in Chapter 2. Reported values of $COV_e$ are between 0.01 and 0.02 for lab index tests for $\gamma_{soil}$; between 0.08 and 0.38 for triaxial compression tests; between 0.19 and 0.20 for direct shear tests; and between 0.15 and 0.45 for standard penetration tests (SPT) (Phoon and Kulhawy 1999a).
3.1.2 Scales of Fluctuation and Spatial Average Variance Reductions

VanMarcke (1977), and later Phoon and Kulhawy (1999b), described the variability of in-situ soil properties as fluctuating about a smoothly varying trend function. The distance within which a soil property shows strong correlation from fluctuation point to fluctuation point is described as the scale of fluctuation, or autocovariance distance, $\delta$. De Groot and Baecher (1993) built upon this definition by adding that the autocovariance distance is the distance at which the autocovariance function decays to a value of $e^{-1}$.

Methods for computing $\delta$ are presented by Jaksa et al. (2000) and Haldar and Babu (2007). The procedure begins with evaluating the data for stationarity and removing any apparent trend through the use of ordinary least-squares (OLS). Baecher and Christian (2003) observed that using trend surfaces should be kept as simple as possible when performing regression analysis to achieve stationarity of the dataset due to the introduction of additional uncertainty through the estimation of multiple curve-fitting parameters for higher-order trend equations. For this reason, use of a linear trend equation is often employed. The residuals of the detrended data are then examined a second time by visual inspection or Kendall’s T-test to assure trends are adequately removed. Using the detrended data, the sample autocorrelation function (ACF), $r_k$ (Equation 3.2), is plotted versus lag, $k$. Note that lags are sometimes chosen such that one lag is equal to a representative distance.

\[
    r_k = \frac{\sum_{i=1}^{N-k}(X_i - \bar{X})(X_{i+k} - \bar{X})}{\sum_{i=1}^{N}(X_i - \bar{X})^2} \tag{3.2}
\]
where:

\[ X_i = \text{property of value } X \text{ at location } i, \]

\[ \bar{X} = \text{sample mean of the property } X, \]

\[ N = \text{number of data points, and} \]

\[ k = \text{maximum number of lags allowable (typically } k = N/4) \]

The theoretical ACF functions presented by Jaksa et al. (2000), Li and White (1987), and VanMarcke (1977) (Figure 3.1) are fit to the sample ACF by adjusting curve fitting parameters \( a, b, c, d, \) and \( f \) to provide a best-fit to the sample ACF by visual inspection. Scale of fluctuation is determined using the relationships between the curve fitting parameters and \( \delta \) presented in the Figure 3.1. The distance corresponding to the point at which the sample ACF intersects Bartlett’s Limits is also a means to estimate \( \delta \).

\[
\begin{align*}
|r_n| &= \frac{1.96}{\sqrt{N}} \\
\text{Function 1} &\quad \rho_n(\Delta x) = e^{-\left(\frac{|\Delta x|}{a}\right)} \quad \delta_r = 2a \\
\text{Function 2} &\quad \rho_n(\Delta x) = e^{-\left(\frac{|\Delta x|}{b}\right)^2} \quad \delta_r = b\sqrt{\pi} \\
\text{Function 3} &\quad \rho_n(\Delta x) = e^{-\left(\frac{|\Delta x|}{c}\right)^2}\cos\left(\frac{\Delta x}{c}\right) \quad \delta_r = c \\
\text{Function 4} &\quad \rho_n(\Delta x) = e^{-\left(\frac{|\Delta x|}{d}\right)^2}\left[1 + \frac{|\Delta x|}{d}\right] \quad \delta_r = 4d \\
\text{Function 5} &\quad \rho_n(\Delta x) = \begin{cases} 
1 - \frac{|\Delta x|}{f} & \text{for } |\Delta x| \leq f \\
0 & \text{for } |\Delta x| > f 
\end{cases} \quad \delta_r = f
\end{align*}
\]

\( a, b, c, d, f \) = curve-fitting parameters
\( \delta \) = scale of fluctuation
\( \Delta x \) = lag distance
\( \rho_n(\Delta x) \) = Theoretical ACF

Figure 3.1 Theoretical autocorrelation functions.
Values of scale of fluctuation documented in the literature depend upon the material and sampling interval. For $s_u$ of different types of clays, vertical scales of fluctuation ($\delta_v$) have been reported on the order of 3 m (IPET 2009), between 1 and 6 m Phoon and Kulhawy (1999b), and between 2 and 5 m (Hicks and Samy 2002). Horizontal scales of fluctuation ($\delta_h$) for $s_u$ of clays have been reported on the order of 305 m (IPET 2009) and between 40 to 60 m Phoon and Kulhawy (1999b). These values are presented to illustrate order of magnitude of scale of fluctuation. Caution should be exercised when using these values given the differences in geological processes in specific locations and the mineralogy and behavioral differences of different types of clays.

Once $\delta$ has been computed, variance reductions for use in spatial averaging are then computed using Equation 3.3 (Equation 3; VanMarcke 1977). Scale of fluctuation can be computed for vertical, horizontal, and a combination of vertical and horizontal autocorrelation. When computing the latter, the variance reduction is computed for each averaging length and the variance reduction terms are multiplied together to get an overarching reduction (VanMarcke 1977).

\[
\Gamma^2(L) = \begin{cases} 
1 & \text{for } L \leq \delta_v \\
\frac{\delta_v}{L} & \text{for } L > \delta_v
\end{cases}
\]

where:

$L =$ length of the averaging interval

$\Gamma^2(L) =$ variance reduction factor
Smaller values of $\delta_v$ indicate rapid fluctuations about the average and larger values indicate more gradual variations about the average (VanMarcke 1977). Examining the variance reduction function, it is clear that for large averaging distances (greater than $\delta$), fluctuations representing the inherent soil variability tend toward the average and uncertainty is reduced. Conversely, for smaller averaging distances, inherent variations remain exaggerated and uncertainty tends toward point uncertainty.

3.2 Case Study

3.2.1 Subsurface Investigations and Site Geology

The production-level dataset selected is associated with a hurricane risk reduction floodwall near New Orleans, Louisiana, on the west bank of the Mississippi River. This reach of floodwall covers a distance of 1,181 m. The hurricane risk reduction features at this particular site began with an earthen levee, which was replaced by a cantilever sheet pile I-wall shortly after Hurricane Katrina. The permanent structure constructed to defend against hurricane storm surge was a pile-founded T-wall. The original levee crown existing at the time of I-wall construction was at approximately Elevation +2.1 m (7.5 ft) North American Vertical Datum of 1988 (2004.65 Epoch) (NAVD 88 (2004.65)). The natural ground elevation at the protected side toe and flood side toe was between Elevations 0 and – 0.61 m (0 to -2.0 feet). The data used for this analysis draws upon subsurface investigations performed in 2007 and supplemented in 2008 by deeper borings to facilitate development of pile capacities for the T-wall bearing piles. Subsurface investigations were conducted in accordance with USACE (2008) and consisted of undisturbed borings taken with a 127 mm (5 in) Shelby tube along the centerline, flood side toe, and protected side toe of the existing embankment. Undisturbed borings were spaced in accordance with USACE (2008) such that no more than an interval of 304.8 m (1,000 ft) existed between undisturbed borings along the centerline or along both toes. CPTs supplemented the
undisturbed borings to provide a total of three samples per “section” with each “section” spaced approximately 152.4 m (500 ft) on center. Because of the continued emphasis on the use of undisturbed borings for design and the additional data reductions required for the statistical analysis of CPT data, this study focuses on the undisturbed boring data.

Table 3.1 provides the boring locations for the twelve undisturbed borings used in this study relative to the project survey baseline. As shown in the table, most of the original borings ranged in depth from El. -19.3 m (-63 ft) to El. -27.5 m (-90 ft). All but one boring in the second set of borings were washed out to El. -24.4 m (-80 ft) and extended to El. -36.6 m (-120 ft). Boring WWHC-83UPT extending from El. 0.0 m to a depth of El. -45.7 m (-148 ft) (Burns 2009).

Table 3.1  Boring locations.

<table>
<thead>
<tr>
<th>Boring IDa</th>
<th>Horizontal Distance (m)</th>
<th>Offsetb (m)</th>
<th>Vertical Limits</th>
<th>Top (m)</th>
<th>Bottom (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>WWHC-80UCL-07</td>
<td>132</td>
<td>-0.7</td>
<td>2.3</td>
<td>-25.7</td>
<td></td>
</tr>
<tr>
<td>WWHC-80UPT-07</td>
<td>136</td>
<td>-23.0</td>
<td>-0.3</td>
<td>-22.3</td>
<td></td>
</tr>
<tr>
<td>WWHC-81UCL-07</td>
<td>436</td>
<td>-4.4</td>
<td>2.2</td>
<td>-19.7</td>
<td></td>
</tr>
<tr>
<td>WWHC-82UCL-07</td>
<td>748</td>
<td>1.8</td>
<td>2.4</td>
<td>-25.0</td>
<td></td>
</tr>
<tr>
<td>WWHC-82UPT-07</td>
<td>751</td>
<td>-13.7</td>
<td>-0.1</td>
<td>-27.5</td>
<td></td>
</tr>
<tr>
<td>WWHC-83UFT-07</td>
<td>1,046</td>
<td>1.5</td>
<td>0.5</td>
<td>-21.4</td>
<td></td>
</tr>
<tr>
<td>WWHC-83UCL-07</td>
<td>1,046</td>
<td>-0.7</td>
<td>2.6</td>
<td>-19.3</td>
<td></td>
</tr>
<tr>
<td>WWHC-75UPT-08</td>
<td>-</td>
<td>-21.1</td>
<td>-24.4d</td>
<td>-36.6</td>
<td></td>
</tr>
<tr>
<td>WWHC-76UPT-08</td>
<td>277</td>
<td>-12.1</td>
<td>-24.4d</td>
<td>-36.6</td>
<td></td>
</tr>
<tr>
<td>WWHC-77UPT-08</td>
<td>593</td>
<td>-12.8</td>
<td>-24.4d</td>
<td>-36.6</td>
<td></td>
</tr>
<tr>
<td>WWHC-78UPT-08</td>
<td>896</td>
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<td></td>
</tr>
<tr>
<td>WWHC-83UPT-08</td>
<td>1,069</td>
<td>-21.6</td>
<td>0.0</td>
<td>-45.7</td>
<td></td>
</tr>
</tbody>
</table>

07 and 08 denote year boring taken

b Offset (negative values = left; positive = right).


dUpper samples “washed out” to boring top depth

The project area was separated into two soils reaches, with the dividing line drawn at Station 252+67 m (near WWHC-77UPT). Both reaches consist primarily of a combination of fat and lean clay (CH and CL), with pockets of peat and silt (PT and ML). The East Reach contains
a stratum of intradelta sand of variable thickness, which was conservatively neglected in the original design analysis and is neglected herein as well. Strain at failure for both the UU and UC tests were reviewed to assess whether any individual data points warranted adjustment. If the strain measured at failure for data points in two separate adjacent strata were judged to be drastically different, the higher strength value would be adjusted to the common strain value.

Outlying data points were reviewed for possible removal from the dataset. Impacts of individual outliers were reviewed. If no other data points were near the outlying point, the test type was then reviewed. If the outlying data point was obtained from a UC test, it was removed from the dataset. If it was obtained from a UU test, it was left in, given the higher confidence in UU tests more accurately representing in situ conditions. Two points were removed in this manner, both in the West Reach: one at El. -0.5 m (-1.8 ft) where $s_u = 97.0$ kPa (2,025 psf), and one at El. -30.7 m (-100.8) where $s_u = 10.4$ kPa (218 psf).

Since silt and peat were interspersed within strata treated as clay, the different behavior of these materials from the clays warranted a review of how to treat the data points for this analysis. Peat accounted for roughly 5% of the data points for the West Reach (spread over 2 strata) and less than 2% of the data points in the East Reach (all in a single stratum). There were no data points for silt in the West Reach, but silt accounted for roughly 7% of the data points in the East Reach (spread over 4 strata). Sensitivity of the mean and COV for $\gamma_{soil}$ and $s_u$ were reviewed to determine the impact of the silt and peat on the overall sample. In most of the strata reviewed, removal of the samples had little impact on mean or COV for either parameter. Where the peat and silt constituted a large percentage of a stratum’s data points, there were noticeable impacts to COV.

The individual $s_u$ data points for the silt and peat were also compared to the clay data points for consistency. This was done both within the stratum and in comparison to values in
adjacent strata. Values of $s_u$ for the peat and silt were typically within the range of the clay values. Ultimately, data points were left in the dataset if the values of both parameters ($s_u$ and $\gamma_{soil}$) were consistent with values determined for the clay samples. For this dataset, there were a limited number of data points for silt and peat, meaning the strata were predominately clay. For larger concentrations of differing samples or in strata that are predominately silts or peats with a small percentage of clay, inclusion in the dataset may not be appropriate. Careful judgment in conjunction with consultation of an experienced geologist is recommended.

For this study, the centerline and toe boring data were combined for each reach. When dealing with manmade embankments, statistics of $\gamma_{soil}$ and $s_u$ are time dependent, especially if test data are taken prior to placement of substantial quantities of fill. Gains in $s_u$ beneath the embankment centerline will result in different values over time due to the embankment surcharge causing consolidation of the underlying strata. Before making this decision, the data was reviewed for visible differences in centerline and toe strengths. Although the toe strengths were slightly lower than the centerline strengths, there was sufficient overlap in test results that any error introduced is masked by the inherent soil variability and measurement and testing error.

3.2.2 Inherent Soil Variability and Spatial Average of Unit Weight

Samples for lab determination of $\gamma_{soil}$ were taken at approximately 1.2 to 3.0 m intervals per boring. The original design presented separate tabulations of $\gamma_{soil}$ for toe borings and centerline borings for each reach. The sample COV per stratum for each reach, taken as $COV_w$, is presented in Table 3.2. Values of $COV$ for both reaches ranged from 0.04 to 0.15, which shows agreement with the 0 to 0.10 range found in the literature. Given the low COV computed for $\gamma_{soil}$ (as compared to those typically computed for $s_u$), no scales of fluctuation or variance reductions were computed. Using the upper (U) and lower (L) bounds for COV for measurement error (ME), the total COV computed for each soil stratum for East and West reaches are
presented in Table 3.2. As shown in the table, the adjustment for measurement error causes minimal change to $COV_w$.

**Table 3.2  Statistics of $\gamma_{soil}$ for each reach adjusted for measurement error (ME).**

| Stratum | Top Elev. (m) | Mean $\gamma_{soil}$ (kN/m$^3$) | No. Tests | ME, U$^a$ | COV ME, U$^a$ | ME, L$^b$ | COV ME, L$^b$ | Top El. (m) | Mean $\gamma_{soil}$ (kN/m$^3$) | No. Tests | ME, U$^a$ | COV ME, U$^a$ | ME, L$^b$ | COV ME, L$^b$
<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>NG</td>
<td>16.4</td>
<td>9</td>
<td>0.151</td>
<td>0.153</td>
<td>0.152</td>
<td>NG</td>
<td>17.7</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>-1.2</td>
<td>13.3</td>
<td>21</td>
<td>0.142</td>
<td>0.143</td>
<td>0.142</td>
<td>-1.2</td>
<td>13.3</td>
<td>9</td>
<td>0.115</td>
<td>0.116</td>
<td>0.115</td>
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<td></td>
</tr>
<tr>
<td>3</td>
<td>-4.9</td>
<td>17.2</td>
<td>4</td>
<td>0.044</td>
<td>0.048</td>
<td>0.045</td>
<td>-4.9</td>
<td>12.6</td>
<td>2</td>
<td>0.097</td>
<td>0.099</td>
<td>0.097</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>-6.1</td>
<td>16.9</td>
<td>7</td>
<td>0.069</td>
<td>0.072</td>
<td>0.070</td>
<td>-6.1</td>
<td>18.2</td>
<td>3</td>
<td>0.044</td>
<td>0.049</td>
<td>0.046</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>-7.3</td>
<td>17.6</td>
<td>8</td>
<td>0.060</td>
<td>0.063</td>
<td>0.061</td>
<td>-7.3</td>
<td>17.3</td>
<td>7</td>
<td>0.061</td>
<td>0.064</td>
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<tr>
<td>6</td>
<td>-10.7</td>
<td>16.5</td>
<td>34</td>
<td>0.057</td>
<td>0.060</td>
<td>0.058</td>
<td>-10.7</td>
<td>16.6</td>
<td>18</td>
<td>0.038</td>
<td>0.043</td>
<td>0.039</td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>-19.8</td>
<td>17.9</td>
<td>6</td>
<td>0.090</td>
<td>0.093</td>
<td>0.091</td>
<td>-19.8</td>
<td>15.9</td>
<td>4</td>
<td>0.013</td>
<td>0.024</td>
<td>0.016</td>
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<td></td>
</tr>
<tr>
<td>8</td>
<td>-23.8</td>
<td>18.6</td>
<td>33</td>
<td>0.032</td>
<td>0.037</td>
<td>0.033</td>
<td>-22.9</td>
<td>18.5</td>
<td>20</td>
<td>0.039</td>
<td>0.044</td>
<td>0.041</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

$^a$COV adjusted for upper bound of measurement error
$^b$COV adjusted for lower bound of measurement error

Figure 3.2 presents the original design $\gamma_{soil}$ for toe borings and centerline borings along with the computed means for the combined centerline-toe dataset for each soils reach. As shown in the figure, the mean values computed from the combined dataset essentially fell between those used in the original design for toe and centerline. To provide consistency in stratification from $\gamma_{soil}$ to shear strength and from separate toe/centerline data to combined dataset, some soil layer boundaries were adjusted. Except for those instances, there was small change in $\gamma_{soil}$ from that presented in the original design to the sample mean.

### 3.2.3 Inherent Soil Variability of Undrained Shear Strength

Samples for lab determination of $s_u$ were taken at approximately 0.3 to 3.0 m intervals per boring. Undrained shear strengths were determined using laboratory UU and UC tests performed in accordance with ASTM protocols, thus minimizing measurement error. As can be seen from Figure 3.3, the original design strength lines were constructed utilizing linear trend lines. The original design strength lines for centerline and toe strengths were separated by 7.2
kPa to 9.6 kPa (150 to 200 psf), primarily at the shallow depths. As shown in Figure 3.3, the data points between El. -1 and El. -20 are relatively tightly clustered about the trend lines. Scatter in the data is noted in the shallow layers comprising the compacted fill embankment and at deeper layers. The compacted clay embankment is also the reason for the spike in $s_u$ near the ground surface.

Figure 3.2  Unit weight of east reach and west reach.
Separate dataset statistics were computed for each reach using UC data only, UU data only, and a combination of both sets of test data. Figure 3.3 and Table 3.3 show only the combined UU and UC test data. When representing site conditions through construction of a strengthline in conventional design, both UU and UC test results are considered. In the typical design process, designers attempt to test samples at conditions as close as possible to in situ conditions, lending more confidence in UU tests. However, the results of the two test methods...
are often indistinguishable and given the limited number of borings and UU or UC test data, the use of both sets of test results provides a more robust statistical sample. The sample COV per stratum, taken as $COV_w$, is presented in Table 3.3. Although the statistics of shear strength is often treated in terms of $s_u/p$ ratios, where $p$ is the effective overburden pressure (Shansep method), this chapter utilizes a stationary mean for each soil stratum. As expected, values of COV for both reaches ranged from 0.22 to 0.72, which shows pretty good agreement with the 0.10 to 0.62 range found in the literature.

Table 3.3  Undrained shear strength ($s_u$) mean and coefficient of variation for each stratum, East and West reaches.

<table>
<thead>
<tr>
<th>Stratum</th>
<th>East</th>
<th>West</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mean (kPa)</td>
<td>COV</td>
</tr>
<tr>
<td>1</td>
<td>43.1</td>
<td>0.720</td>
</tr>
<tr>
<td>2</td>
<td>12.9</td>
<td>0.559</td>
</tr>
<tr>
<td>3</td>
<td>14.4</td>
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<td>6</td>
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<tr>
<td>7</td>
<td>25.9</td>
<td>0.365</td>
</tr>
<tr>
<td>8</td>
<td>43.1</td>
<td>0.422</td>
</tr>
</tbody>
</table>

3.2.4  Scales of Fluctuation

One of the challenges in computing scale of fluctuation is lack of a sufficient dataset. Typically, borings or CPTs are taken at horizontal intervals of 152 m (500 ft) with sample intervals varying within each boring. For scale of fluctuation computations, it is best to obtain samples at intervals less than the anticipated scale of fluctuation (Jaksa et al. 2000) or for better results at intervals representative of the desired distance between lags. For this study, an attempt was made to quantify vertical and horizontal scales of fluctuation using the available data. Analysis started with the West reach dataset. Separate analyses were performed for the UC data, the UU data, and a combination of UC and UU data. The East reach analysis consisted only of
the combined dataset. Finally, given the similarity in classification and stratification, analyses were performed combining the two reaches for the same 3 cases as the West reach. For the computation of $\delta_h$, assumptions and model simplifications (listed in paragraph 2.4.2) were made to produce results.

3.2.4.1 Vertical Scale of Fluctuation

For each analysis, the methodology described in Section 1.2 was followed. In the initial assessment of the smaller UU and UC datasets separately, it appeared that the data exhibited a polynomial trend, given the spike in values of $s_u$ in the compacted clay levee. With this knowledge and using the observations of Baecher and Christian (2003), however, the data was fit to a linear trend. To determine the impact the bias in the trend line would have on the computed scale of fluctuation, a separate set of No Fill analyses that neglected the data above El. -1.2 were also performed.

Using the OLS best-fit, the trend was subtracted from the original dataset for each case and the residuals were plotted versus depth to confirm stationarity. A distance of 0.3 m (1 ft) between data points was chosen to provide enough data points for fitting the models to the ACF. Before computing the ACF, the detrended data were arranged by elevation in even one-foot increments to assess data gaps. For those elevations where no test data existed, data values were artificially generated using linear interpolation between the detrended residuals. Using an approach similar to Koutnik (2012), where multiple measurements for the same elevation existed, the residuals were averaged.

The ACF was plotted using Equation 3.2. Only 31 lags ($N = 124$ datapoints/4) were plotted. Each of the models described in Figure 3.1 (page 37) were overlain on the ACF plot and the curve-fitting parameters $a$, $b$, $c$, $d$, and $f$ were adjusted to achieve a best fit (Figure 3.4). Scale of fluctuation was then calculated. The West reach ACF was developed first, with separate
cases for UU data only, then UC data only, then Both UU and UC data combined. Then the same three cases (UU, UC, and Both) were used for the All reach (both West and East datasets combined). It became apparent that the volume of data contained in the Both case for West reach and All reach, only the Both case was computed for the East reach. The results for analyses considering all soil strata are presented in Table 3.4. Results for analyses not considering the compacted levee and shallow soils above El. -1.2 are presented in Table 3.5.

Overall, values of $\delta_v$ ranged from a low of 1.07 m (3.5 ft) for the East Reach UU and UC case to a high of 4.88 m (16.0 ft) for the West Reach UC case. The highest values of $\delta_v$ were computed for the West Reach, UC case, with values ranging from 3.81 to 4.88 m t (12.5 to 16.0 ft). The smallest values of $\delta_v$ were computed for the East Reach, Both case, with values ranging from 1.07 to 1.37 m (3.5 to 4.5 ft).

![Figure 3.4](image)

**Figure 3.4** ACF for both soils reaches combined (UU and UC data combined).
Table 3.4  Vertical scale of fluctuation for $s_u$—all data$^{a,b,c}$.

<table>
<thead>
<tr>
<th>Reach</th>
<th>Test</th>
<th>#1$^d$</th>
<th>#2$^d$</th>
<th>#3$^d$</th>
<th>#4$^d$</th>
<th>#5$^d$</th>
<th>BL$^e$</th>
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<tbody>
<tr>
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<td>UU</td>
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<td>1.83</td>
<td>2.32</td>
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<td></td>
<td>UC</td>
<td>4.27</td>
<td>4.86</td>
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<td>4.88</td>
<td>4.57</td>
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<tr>
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<td>Both</td>
<td>2.44</td>
<td>2.43</td>
<td>2.06</td>
<td>2.44</td>
<td>2.44</td>
<td>2.13</td>
</tr>
<tr>
<td>East</td>
<td>Both</td>
<td>1.37</td>
<td>1.34</td>
<td>1.07</td>
<td>1.34</td>
<td>1.37</td>
<td>1.22</td>
</tr>
<tr>
<td>All</td>
<td>UU</td>
<td>1.52</td>
<td>1.62</td>
<td>1.37</td>
<td>1.52</td>
<td>1.52</td>
<td>1.37</td>
</tr>
<tr>
<td></td>
<td>UC</td>
<td>1.77</td>
<td>1.89</td>
<td>1.52</td>
<td>1.83</td>
<td>1.83</td>
<td>1.83</td>
</tr>
<tr>
<td></td>
<td>Both</td>
<td>1.52</td>
<td>1.62</td>
<td>1.52</td>
<td>1.71</td>
<td>1.52</td>
<td>1.52</td>
</tr>
</tbody>
</table>

$^a$All dimensions in m  
$^b$Lag of 0.30 m  
$^c$Bold values indicate closest fit(s) to autocorrelation function  
$^d$Function number corresponding to Figure 3.1  
$^e$BL = Bartlett’s Limits

Table 3.5  Vertical scale of fluctuation for $s_u$—all data below El. -1.2$^{a,b,c}$.

<table>
<thead>
<tr>
<th>Reach</th>
<th>Test</th>
<th>#1$^d$</th>
<th>#2$^d$</th>
<th>#3$^d$</th>
<th>#4$^d$</th>
<th>#5$^d$</th>
<th>BL$^e$</th>
</tr>
</thead>
<tbody>
<tr>
<td>West</td>
<td>UU</td>
<td>1.83</td>
<td>2.04</td>
<td>1.52</td>
<td>1.95</td>
<td>1.98</td>
<td>1.68</td>
</tr>
<tr>
<td></td>
<td>UC</td>
<td>2.44</td>
<td>2.49</td>
<td>1.98</td>
<td>2.44</td>
<td>2.29</td>
<td>1.83</td>
</tr>
<tr>
<td></td>
<td>Both</td>
<td>1.37</td>
<td>1.34</td>
<td>1.04</td>
<td>1.40</td>
<td>1.37</td>
<td>1.22</td>
</tr>
<tr>
<td>East</td>
<td>Both</td>
<td>0.85</td>
<td>0.94</td>
<td>0.70</td>
<td>0.91</td>
<td>0.88</td>
<td>0.98</td>
</tr>
<tr>
<td>All</td>
<td>UU</td>
<td>1.52</td>
<td>1.62</td>
<td>1.31</td>
<td>1.52</td>
<td>1.52</td>
<td>1.37</td>
</tr>
<tr>
<td></td>
<td>UC</td>
<td>0.76</td>
<td>0.81</td>
<td>0.67</td>
<td>0.79</td>
<td>0.76</td>
<td>0.76</td>
</tr>
<tr>
<td></td>
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<td>0.70</td>
<td>0.68</td>
<td>0.53</td>
<td>0.61</td>
<td>0.67</td>
<td>0.61</td>
</tr>
</tbody>
</table>

$^a$All dimensions in m  
$^b$Lag of 0.30 m  
$^c$Bold values indicate closest fit(s) to autocorrelation function  
$^d$Function number corresponding to Figure 3.1  
$^e$BL = Bartlett’s Limits

For the No Fill analysis, values of $\delta_v$ ranged from a low of 0.53 m (1.75 ft) for the combined reaches, UU and UC case, to a high of 2.49 m (8.2 ft) for the West Reach UC case. The highest values of $\delta_v$ were computed for the West Reach, UC case, with values ranging from 1.83 to 2.49 m (6.0 to 8.2 ft). The smallest values of $\delta_v$ were computed for the combined reaches, UU and UC case, with values ranging from 0.53 to 0.70 m (1.75 to 2.3 ft). Comparison of the results in Table 3.4 with those in Table 3.5 show that the removal of the levee fill from consideration results in lower values of $\delta_v$ for all cases. This appears to be due to the improved “goodness-of-fit” of the data below El. -1.2 m to the OLS trend line. The result is that there are
large distances between shifts in values from one side of the trend to the other for the all data case, but the better fit shifts many points to the opposite side of the trend. The former case produces a larger scale of fluctuation (less frequent variations about the trend) and the latter case produces a smaller scale of fluctuation (more frequent variations about the trend). This result is consistent with that observed by Koutnik (2012).

For use in computing variance reductions, the No Fill, the Both case results will be used. Given the skew in the trend line when compacted levee fill is considered in the ACF computations, the No Fill case better represents site conditions statistically. Lastly, there is sufficient data and sufficient variability between the two soil reaches to justify analysis of each soil reach separately. Scale of fluctuation values ($\delta_v$) equal to 0.9 m (3 ft) and 1.4 m (4.5 ft) were selected for the East and West reach, respectively. These values are representative of each reach as presented in Table 3.5 and fall within the range of values presented in the literature.

3.2.4.2 Horizontal Scale of Fluctuation

Computations for the horizontal scale of fluctuation, $\delta_h$, followed a similar procedure as that used to determine $\delta_v$. The main challenge in determining $\delta_h$ was the large data gaps between data points. Although the lack of sufficient data precludes the ability to compute usable results, simplifications and assumptions were made to permit computation of values of $\delta_h$. The first simplification made was the grouping of soil strata to condense from 8 separate strata to 3, with Stratum A encompassing data from ground surface to El. -6.1, Stratum B encompassing data from El. -6.1 to El. -19.5, and Stratum C encompassing data from El. -19.5 to El. -45.7. The next simplification made was the combination of data from both soil reaches, which is possible only because of the similarity in stratification and properties. Lastly, in an attempt to simulate data points for analysis, two methods were used: (1) linear interpolation between borings (even at close spacing) and (2) averaging values for borings within a close proximity to one another.
An evaluation of $\delta_h$ was performed for the same three test data combinations used for $\delta_v$. For each case, data were plotted and OLS was performed to obtain linear trend lines. Undrained shear strength values for each stratum were obtained by averaging test results over the entire depth range covered by the stratum. These values were used to interpolate intermediate data points to allow for evaluation of 30.5 m (100 ft) lags. The linear trend was then subtracted from the dataset and the ACF was evaluated. Figure 3.5 presents the resulting ACF for the Both case for Stratum B. Table 3.6 presents values of $\delta_h$ obtained by the use of linear interpolation between data points from each boring to populate the remaining data points. Table 3.7 also uses linear interpolation, but instead uses the average values of the residuals for data points from adjacent borings to represent the nearest lag point. The resulting values of $\delta_h$ are presented in Tables 3.6 and 3.7 for the two methods of data point simulation.

![Figure 3.5 ACF for determination of horizontal scale of fluctuation.](image)
Table 3.6  Horizontal scale of fluctuation for $s_u$ – both soils reaches combined (interpolation).

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Test</th>
<th>#1</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
<th>BL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ground</td>
<td>UU</td>
<td>107</td>
<td>135</td>
<td>91</td>
<td>122</td>
<td>107</td>
<td>137</td>
</tr>
<tr>
<td>A</td>
<td>UC</td>
<td>128</td>
<td>122</td>
<td>91</td>
<td>122</td>
<td>122</td>
<td>152</td>
</tr>
<tr>
<td></td>
<td>-6.1</td>
<td>Both</td>
<td>122</td>
<td>135</td>
<td>107</td>
<td>122</td>
<td>130</td>
</tr>
<tr>
<td>B</td>
<td>UC</td>
<td>183</td>
<td>189</td>
<td>152</td>
<td>183</td>
<td>168</td>
<td>168</td>
</tr>
<tr>
<td></td>
<td>-19.5</td>
<td>Both</td>
<td>183</td>
<td>176</td>
<td>152</td>
<td>183</td>
<td>168</td>
</tr>
<tr>
<td>C</td>
<td>UC</td>
<td>76</td>
<td>86</td>
<td>61</td>
<td>91</td>
<td>84</td>
<td>73</td>
</tr>
<tr>
<td></td>
<td>-45.7</td>
<td>Both</td>
<td>137</td>
<td>135</td>
<td>99</td>
<td>122</td>
<td>122</td>
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</table>

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Test</th>
<th>#1</th>
<th>#2</th>
<th>#3</th>
<th>#4</th>
<th>#5</th>
<th>BL</th>
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</thead>
<tbody>
<tr>
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<td>216</td>
<td>183</td>
<td>213</td>
<td>206</td>
<td>175</td>
</tr>
<tr>
<td>A</td>
<td>UC</td>
<td>290</td>
<td>284</td>
<td>229</td>
<td>274</td>
<td>274</td>
<td>274</td>
</tr>
<tr>
<td></td>
<td>-6.1</td>
<td>Both</td>
<td>201</td>
<td>203</td>
<td>168</td>
<td>183</td>
<td>195</td>
</tr>
<tr>
<td>B</td>
<td>UC</td>
<td>192</td>
<td>208</td>
<td>168</td>
<td>189</td>
<td>198</td>
<td>183</td>
</tr>
<tr>
<td></td>
<td>-19.5</td>
<td>Both</td>
<td>229</td>
<td>230</td>
<td>175</td>
<td>213</td>
<td>213</td>
</tr>
<tr>
<td>C</td>
<td>UC</td>
<td>152</td>
<td>135</td>
<td>91</td>
<td>122</td>
<td>122</td>
<td>122</td>
</tr>
<tr>
<td></td>
<td>-45.7</td>
<td>Both</td>
<td>137</td>
<td>135</td>
<td>107</td>
<td>152</td>
<td>130</td>
</tr>
</tbody>
</table>

Using the linear interpolation method (Table 3.6), values of $\delta_h$ for Stratum A ranged from a low of 91 m (300 ft) for the UU case to a high of 168 m (550 ft) for the Both case. The high values of $\delta_h$ for Stratum A were distributed across all three cases with high values of 137, 152, and 168 m (450, 500, and 550 ft) exhibited in the UU, UC, and Both cases, respectively. The lowest values of $\delta_h$ were computed for the UU case, with values ranging from 91 to 137 m (300 to 450 ft).
Stratum B results ranged from a low of 107 m (350 ft) for the UU case to a high of 189 m (620 ft) for the UC case. The highest values of $\delta_h$ for Stratum B were computed for the UC case, with values ranging from 152 m to 189 m (500 to 620 ft). The lowest values of $\delta_h$ were computed for the UU case, with values ranging from 107 to 152 m (350 to 500 ft). The Both case results were close to the UC case, with values ranging from 152 to 183 m (500 to 600 ft).

Stratum C results ranged from a low of 61 m (200 ft) for the UC case to a high of 162 m (532 ft) for the UU case. The highest values of $\delta_h$ for Stratum C were computed for the UU case, with values ranging from 137 m to 183 m (450 to 600 ft). The lowest values of $\delta_h$ were computed for the UC case, with values ranging from 61 to 91 m (200 to 300 ft).

Using the linear interpolation plus averaging method (Table 3.7), values of $\delta_h$ for Stratum A ranged from a low of 168 m (550 ft) for the Both case to a high of 290 m (950 ft) for the UU case. The highest values of $\delta_h$ for Stratum A were computed for the UC case, with values ranging from 229 to 290 m (750 to 950 ft). The lowest values of $\delta_h$ were computed for the Both case, with values ranging from 168 to 203 m (550 to 665 ft). The UU case results fell between the UC and Both cases, with values ranging from 175 to 244 m (575 to 800 ft).

Stratum B results ranged from a low of 168 m (550 ft) for all three cases to a high of 244 m (800 ft) for the UU case. The highest values of $\delta_h$ for Stratum B were computed for the UU case, with values ranging from 168 m to 244 m (550 to 800 ft). The lowest values of $\delta_h$ were computed for the UC case, with values ranging from 168 to 208 m (550 to 682 ft). The Both case results were between the other two cases, with results ranging from 175 to 230 m (575 to 753 ft).

Stratum C results ranged from a low of 91 m (300 ft) for the UC case to a high of 189 m (620 ft) for the UU case. The highest values of $\delta_h$ for Stratum C were computed for the UU case,
with values ranging from 152 m to 189 m (500 to 620 ft). The lowest values of \( \delta_h \) were computed for the UC case, with values ranging from 91 to 152 m (300 to 500 ft). The Both case results were between the other cases, with values ranging from 107 to 152 m (350 to 500 ft).

With the exception of the Both case for Stratum C, the “averaging plus linear interpolation” produced more consistent bands of values across the different combinations of data. For \( \delta_h \), there was no consistent trend from either method that seemed to relate the number of data points to values of \( \delta_h \). As shown in Table 3.7, Stratum A and Stratum C did appear to have reduced values of \( \delta_h \) for the Both dataset.

Using logic similar to that applied for assignment of values for the vertical scale of fluctuation, the Both dataset provides a more complete representation of the project site. Given the spacing of the borings, neither of the methods used to simulate intermediate data points provide a high degree of confidence. Therefore, the mean, median, and mode values for both methods for each grouped stratum were examined. The ultimate values of \( \delta_h \) presented were intentionally selected within or very near the zone of overlapping values. From this dataset, \( \delta_h \) is equal to 122 m (400 ft) for Stratum A, \( \delta_h \) is equal to 183 m (600 ft) for Stratum B, and \( \delta_h \) is equal to 122 m (400 ft) for Stratum C. These values are within the range of values presented in the literature and are all very close to the sampling interval (boring spacing).

It should be noted that both \( \delta_v \) and \( \delta_h \) exhibited sensitivity to removal of the two outlying datapoints. Removal of the datapoints resulted in an increase in \( \delta_v \) of almost 2 m for the West reach UC case for the “all data” analysis and an increase of roughly 0.5 m for the “data below El. -1.2” analysis. Removal of the datapoints also resulted in changes to the UC case for \( \delta_h \) on the order of 120 m for the “(interpolation)” approach and approximately 50 m for the “(averaging and interpolation)” approach.
3.2.5 Spatial Averaging of Undrained Shear Strength

Variance reductions were computed using Equation 3.3 using the scales of fluctuation in the vertical and horizontal directions for each soil reach recommended in the previous section. Averaging lengths coincide with the thickness of each stratum and the horizontal length of each soils reach. Table 3.8 presents the variance reductions for the vertical direction, the horizontal direction, and the combination for each reach. As shown in Table 3.8, the variance reductions can be significant if the averaging length is sufficiently large when coupled with a small scale of fluctuation.

Table 3.8 East and West reach variance reduction factors for $s_u$.

<table>
<thead>
<tr>
<th>Strat.</th>
<th>East $\Gamma^2(L_a)^a$</th>
<th>$\Gamma^2(L_h)^b$</th>
<th>$\Gamma^2(L_v, L_h)^{a,b}$</th>
<th>West $\Gamma^2(L_a)^a$</th>
<th>$\Gamma^2(L_h)^b$</th>
<th>$\Gamma^2(L_v, L_h)^{a,b}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.30</td>
<td>0.21</td>
<td>0.06</td>
<td>0.45</td>
<td>0.20</td>
<td>0.09</td>
</tr>
<tr>
<td>2</td>
<td>0.25</td>
<td>0.21</td>
<td>0.05</td>
<td>0.38</td>
<td>0.20</td>
<td>0.08</td>
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<tr>
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<td>0.20</td>
</tr>
<tr>
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<td>0.24</td>
<td>1.00</td>
<td>0.30</td>
<td>0.30</td>
</tr>
<tr>
<td>5</td>
<td>0.27</td>
<td>0.31</td>
<td>0.09</td>
<td>0.41</td>
<td>0.30</td>
<td>0.12</td>
</tr>
<tr>
<td>6</td>
<td>0.10</td>
<td>0.31</td>
<td>0.03</td>
<td>0.15</td>
<td>0.30</td>
<td>0.05</td>
</tr>
<tr>
<td>7</td>
<td>0.23</td>
<td>0.21</td>
<td>0.05</td>
<td>0.45</td>
<td>0.20</td>
<td>0.09</td>
</tr>
<tr>
<td>8</td>
<td>0.07</td>
<td>0.21</td>
<td>0.01</td>
<td>0.10</td>
<td>0.20</td>
<td>0.02</td>
</tr>
</tbody>
</table>

$^a$Vertical averaging distance = stratum thickness

$^b$Horizontal averaging distance = reach length

Table 3.9 presents the overall spatial average considering the upper and lower bounds for measurement error in addition to the variance reductions. To further illustrate the impact of using variance reductions on reliability analysis, consider the reduction in $COV$ including adjustment for the upper bound of measurement error (ME, U) for Stratum 1 in the East reach. Without variance reduction, $COV_w$ is 0.735. Application of the vertical autocorrelation variance reduction takes this value down to 0.422. The large reduction due to vertical autocorrelation reduces the impact of also accounting for horizontal autocorrelation, as evidenced by the further reduction down to 0.235. As with $\gamma_{soils}$ the impacts of measurement error only increase the total $COV$ by a few percent.
Table 3.9  East and West reach COVs for $s_u$ adjusted for measurement error and spatial averaging.

<table>
<thead>
<tr>
<th>Strat.</th>
<th>ME, U^a</th>
<th>ME, U^b</th>
<th>ME, L^a</th>
<th>ME, L^b</th>
<th>ME, U^c</th>
<th>ME, U^d</th>
<th>ME, L^c</th>
<th>ME, L^d</th>
<th>ME, U^e</th>
<th>ME, U^f</th>
<th>ME, L^e</th>
<th>ME, L^f</th>
</tr>
</thead>
<tbody>
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<td>0.235</td>
<td>0.721</td>
<td>0.397</td>
<td>0.218</td>
<td>0.500</td>
<td>0.353</td>
<td>0.215</td>
<td>0.353</td>
<td>0.187</td>
<td>0.324</td>
</tr>
<tr>
<td>2</td>
<td>0.579</td>
<td>0.317</td>
<td>0.197</td>
<td>0.562</td>
<td>0.284</td>
<td>0.138</td>
<td>0.329</td>
<td>0.234</td>
<td>0.181</td>
<td>0.297</td>
<td>0.186</td>
<td>0.094</td>
</tr>
<tr>
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<td>0.316</td>
<td>0.703</td>
<td>0.610</td>
<td>0.283</td>
<td>0.526</td>
<td>0.526</td>
<td>0.324</td>
<td>0.506</td>
<td>0.506</td>
<td>0.231</td>
</tr>
<tr>
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<td>0.286</td>
<td>0.203</td>
<td>0.286</td>
<td>0.249</td>
<td>0.146</td>
<td>0.388</td>
<td>0.388</td>
<td>0.247</td>
<td>0.362</td>
<td>0.362</td>
<td>0.203</td>
</tr>
<tr>
<td>5</td>
<td>0.501</td>
<td>0.291</td>
<td>0.205</td>
<td>0.480</td>
<td>0.254</td>
<td>0.149</td>
<td>0.407</td>
<td>0.285</td>
<td>0.200</td>
<td>0.382</td>
<td>0.247</td>
<td>0.142</td>
</tr>
<tr>
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<td>0.161</td>
<td>0.327</td>
<td>0.114</td>
<td>0.076</td>
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<td>0.172</td>
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<td>0.068</td>
</tr>
<tr>
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<td>0.242</td>
<td>0.568</td>
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<tr>
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<td>0.159</td>
<td>0.425</td>
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<td>0.072</td>
<td>0.348</td>
<td>0.180</td>
<td>0.158</td>
<td>0.318</td>
<td>0.111</td>
<td>0.067</td>
</tr>
</tbody>
</table>

^aCOV adjusted for upper bound of measurement error
^bCOV adjusted for lower bound of measurement error
^cV = adjusted for vertical autocorrelation; V+H = adjusted for vertical and horizontal autocorrelation

3.3  Summary and Conclusions

This chapter utilized a production-level dataset associated with a hurricane risk reduction floodwall near New Orleans, Louisiana, to provide insight into the suitability of this type of dataset for use in characterizing uncertainty to a level appropriate for reliability analysis. It also provided data on scales of fluctuation for Holocene and Pleistocene clays in Southeast Louisiana. The findings of this study are:

1. The inherent variation in $\gamma_{soil}$, even in the highly-stratified soils of Southeast Louisiana is low relative to $s_u$, which is consistent with the findings in the literature. Even with limited datasets, the sample COV will likely be close to the $COV_w = 10\%$ found in the literature.

2. The inherent variation in $s_u$ can result in values of $COV_w$ approaching the upper value of 62% found in the literature.

3. Before computing scale of fluctuation, it is important to review the data carefully and use caution when removing data outliers. Depending upon the size of the dataset, the computed value can be sensitive, especially if the data point is near the upper or lower spatial bounds of the dataset where data trends can be more drastically impacted.
4. Even with the limited dataset and assumptions required to fill data gaps, values computed for vertical scale of fluctuation computed for individual soils reaches were between 0.85 m and 4.88 m, which is consistent with the values in the literature between 1 and 6 m. This suggests that limited datasets can provide reasonable estimates of \( v \) for use in computing variance reductions. However, the fact that the lower bound value occurs in a soil reach containing peat, silt, and intradelta sands interspersed in clay layers suggests that soil stratification should be reviewed thoroughly before proceeding with analysis.

5. The limited dataset produced horizontal scales of fluctuation between 91 and 290 m, which were within the range of 40 to 300 m presented in the literature. However, given the large number of assumptions required to populate the data gaps, the results may contain unacceptable bias. The level of subsurface definition required and the objective for using the scale of fluctuation should guide the level of accuracy required when planning field investigations. As shown by the drastic reductions in variance that can be computed when scale of fluctuation is low and averaging distances are high, the impact on reliability analysis results could be significant if inappropriate results are applied.

6. The presence of an existing manmade embankment can potentially skew results, as shown by the higher values of vertical scale of fluctuation compared to values when not considering all data above El. -1.2, which is consistent with the findings of Koutnik (2012).

7. If the soil stratification is such that stratum thicknesses are near or less than the lower bound values of \( v \) found in the literature, computation of \( v \) is not recommended as the averaging distance would result in no variance reduction.

8. For many typical hurricane risk reduction floodwall reaches, the length of floodwall may not approach published values of \( h \), especially the upper bound of 300 m. The use of published values of \( h \) is recommended for use in computing variance reductions unless sufficient
undisturbed borings and CPTs are available to provide a complete dataset at a sampling interval that adequately represents all soil strata and supports computation of scale of fluctuation without need to fill data gaps through interpolation, simulation, or grouping of strata.
CHAPTER 4: GLOBAL INSTABILITY AND SPATIAL CORRELATION IN RELIABILITY ANALYSIS OF PILE-FOUNDED HURRICANE RISK REDUCTION STRUCTURES

4.1 Introduction

Since 2005, the U.S. Army Corps of Engineers (USACE) has built upon its efforts to quantify reliability and risk (USACE 1992, 1995) and communicate risk to the public, especially with respect to hurricane risk reduction structures and systems. While much attention has been focused on hurricane risk reduction levees and geotechnical aspects of I-walls (IPET 2009), pile-founded structures have not received the same level of scrutiny in the context of system analysis given their inherent robustness when compared to levees and I-walls. Additional attention to the reliability of pile-founded structures is warranted to provide more quantitative support for reliability model results. Similar to levees and I-walls, a critical element of determining the reliability of pile-founded structures is the analysis of global foundation stability when subjected to hurricane storm surge loading. A major difference between pile-founded structures and levees and floodwalls, however, is the resistance to global instability by the pile foundation.

The pile foundation resistance to global instability is a result of the transmission of the force imparted by the translation of the external soil mass to the pile-soil mass (pile-soil mass is the shaded area denoted \( A_p \) on Figure 4.1) formed by the structural foundation. The imparted force, referred to as the unbalanced load \( (F_{ub}) \), is the lateral force per unit length of structure (perpendicular to the load direction) required to produce equilibrium in the foundation (or slope as the case may be) relative to a pre-established factor of safety for a specific failure surface. In a plane strain analysis, the \( F_{ub} \) is a concentrated load (per unit width) theoretically located between the critical failure plane (failure surface producing the largest \( F_{ub} \)) and the ground surface at the toe of the structure of interest. The \( F_{ub} \) forms the basis for assessing the foundation resistance to global instability.
This chapter proposes a method for incorporating the unbalanced force caused by global instability and applied directly to the pile foundation into a point-estimate reliability model. The proposed method draws upon current USACE design practices (USACE 2008) in combination with the Taylor Series (Wolff 1994) and Rosenbleuth (1975) point estimate methods to provide a production-oriented technique for applying the unbalanced force within the framework of a system reliability model. The proposed model accounts for the uncertainties in different structural components and geotechnical properties in pile-founded hurricane protection systems. An example drawing upon a dataset presented in Chapter 4 is presented to demonstrate the methodology for incorporating unbalanced load into reliability calculations. The example examines global stability analysis and flowthrough computations utilizing both spatially averaged undrained shear strengths and unadjusted undrained shear strengths to highlight the
importance of accounting for spatial correlation of undrained shear strengths to produce realistic probabilities of unsatisfactory performance.

The developed methodology for incorporating unbalanced forces into reliability analysis using routine design tools bridges a critical gap that permits computation of reliability using methods other than complex finite element models. Results obtained by incorporating and neglecting spatial averaging of undrained shear strengths underscore the importance of incorporating spatially averaged undrained shear strengths to obtain realistic probabilities of failure. When determining the reliability of structures that have consequences of failure with respect to human life and cost of infrastructure, it is imperative that models approximate actual conditions as closely as practicable. This chapter demonstrates that using “conservative” values of undrained shear strength (that ignore spatial averaging effects) can result in a high degree of variability that translates into different probabilities of failure. If taken out of context when communicated to the public, inaccurate “conservative” results could unduly undermine public confidence in structural performance.

4.1.1 Global Instability and Pile-Founded Hurricane Risk Reduction Structures

Current USACE design methodology, which has been validated with detailed numerical modeling and centrifuge tests (Abdoun and Sasanakul 2007, Varuso 2010), assumes global instability is resisted by the hurricane risk reduction floodwall pile foundation and its interaction with the surrounding soils. The first step in the process is the global stability analysis, which is performed utilizing Spencer’s method (Spencer 1967) through commercial software such as SLOPE/W (Geo-Slope 2008). Global stability analysis as performed in conjunction with the USACE design methodology for pile resistance to global instability is subjected to several constraints and underlying assumptions (USACE 2008):
1. Any loads imparted directly on the structure (e.g. water, soil, surcharge, or other live or dead loads) are not included in the analysis as they are transmitted directly to the pile foundation through the superstructure.

2. The neutral block, or horizontal failure surface, length is limited to the greater of 0.7\(H\) or the base width of the structure, where the dimension \(H\) is the vertical distance between the horizontal component of the failure surface and the ground surface.

3. The effects of pile-founded structures to the protected side of the hurricane risk reduction structure of interest (such as existing pump stations adjacent to hurricane risk reduction floodwalls) are ignored.

4. For pile foundations of large structures having many rows of piles, limitations are placed on the number of piles included in resisting \(F_{ub}\).

Upon completion of the global stability analysis, the computed nominal factor of safety \(FS_{scN}\) is compared to the allowable factor of safety \(FS_{all}\) for global stability specified in the HSDRRS Design Guidelines (USACE 2008). If \(FS_{all} \geq FS_{scN}\) the section is considered stable. If \(FS_{all} < FS_{scN}\), the section is considered unstable and an unbalanced force must be calculated.

Simply stated, \(F_{ub}\) is the horizontal load per unit width (see Figure 4.1) that is capable of providing sufficient additional resistance to improve the global stability to an acceptable value (in the global stability analysis the direction is opposite that shown in Figure 4.1, as it is a resisting force; for pile group analysis, \(F_{ub}\) is an applied driving force). In the stability model, this force is located beneath the structure base slab, halfway between the ground surface at the toe of the wall and the failure plane as shown in Figure 4.1. The unbalanced force is determined using trial and error by manually varying the force in the stability analysis until the \(FS_{all}\) for global stability is achieved. It is then converted into an applied load that is resisted by the pile-soil system supporting the structure using the methods described in USACE (2008).
The USACE methodology considers $F_{ub}$ to be transmitted into the pile foundation and transmitted through the piles to the deeper soil strata. To transmit this force, the pile must resist the soil force via passive resistance and the soil mass bounded by the extreme flood side and extreme protected side rows of piles, the bottom of the floodwall base slab and the critical failure plane must remain intact. These two limit states are both considered “flowthrough checks,” with the former mechanism considered Flowthrough Check 1 and the latter considered Flowthrough Check 2. If the flowthrough checks are within the $FS_{all}$ for each check specified in the HSDRRS Design Guidelines, the resistance of the structure to $F_{ub}$ then becomes dependent upon the structural resistance of individual piles (and by extension the overall geotechnical and structural resistance of the pile group) to the component of $F_{ub}$ transmitted directly to the pile. Assuming a pile-founded structure with uniform pile spacing for all rows (no staggered piles), the equations for computing flowthrough resistance for a 2D plane-strain problem of a unit width equal to the transverse pile spacing are provided in Figure 4.2. More details about the procedures can be found elsewhere (USACE 2008).

As shown in Figure 4.2, the initial Flowthrough Check 1 (Figure 4.2, Equation 1a) determines whether the passive resistance provided by the flood side row of piles is sufficient to resist 1/2 the total $F_{ub}$ converted to $F_p$ (computed in accordance with Equation 3 of Figure 4.2) applied across the unit width (taken to be the transverse pile spacing, $s_t$ shown in Figure 4.1, Section A-A). If the flood side piles’ passive resistance is insufficient to resist $F_p/2$, the entire magnitude of $F_p$ is considered to be resisted by the passive resistance of all three pile rows, with group reductions for spacing determined in accordance with Equations 4a and 4b of Figure 4.2. Flowthrough Check 2 is performed using Equation 3 of Figure 4.2.
If both flowthrough checks are satisfactory, the pile group analysis is run with $F_p/2$ applied directly to the pile in GROUP (ENSOFT 2006). Axial pile capacity is neglected above the critical failure surface, and lateral resistance above the critical failure surface is adjusted according to whether $FS_{scN}$ is less than 1.0 or $FS_{scN}$ is between 1.0 and $FS_{all}$. If $FS_{scN}$ is less than 1.0, then the p-y curves (or corresponding GROUP input for internally generated p-y curves) are completely zeroed out to reflect no lateral resistance being provided to the pile by the soil. If
If $FS_{scN}$ is between 1.0 and $FS_{all}$, the p-y resistance is pro-rated based on the value of $FS_{scN}$ relative to $FS_{all}$.

### 4.2 Reliability Analysis

Two key terms in the context of this chapter are reliability index, $\beta$, and probability of unsatisfactory performance, $P_{up}$. The reliability index is a measure of the number of standard deviations the expected value of a performance function is from the limit state. The probability of unsatisfactory performance is the probability that a limit state is exceeded, which is not necessarily the same as catastrophic failure of an element or a system. In the context of this study, $P_{up}$ can mean either catastrophic failure or a serviceability failure wherein an element remains intact, but may be irreparably damaged.

As noted by Christian (2004), methods available for geotechnical and, by extension, soil-structure interaction reliability problems include First Order Second Moment (FOSM), First Order Reliability Methods (FORM), point estimate methods, and Monte Carlo Simulation. For hurricane risk reduction structures (particularly in areas with highly stratified soils originating from a combination of fluvial, deltaic, and coastal deposits, similar to Southeast Louisiana (LGS 2008)), the number of variables to consider for a given limit state such as global stability can be quite large in a given analysis. For example, the case study presented in this chapter contains 8 different clay strata, which translates into 16 variables when undrained shear strength ($s_u$) and unit weight ($\gamma_{soil}$) are both considered. If $c$-$\phi$ soils (such as silts) are present, the number of variables increases further. Assumptions about the probability distributions of some variables are available in the literature such as:


However, information about the probability distributions for other variables may not be readily available (see Chapter 2) for tabulations of available probability distribution data, and may not fall into the normal or lognormal categories. Based on this limitation, FORM and Monte Carlo Simulation, were not selected for this study. Further, the probability of instability of a hurricane risk reduction structure is only one desired aspect of reliability analysis of a floodwall. Providing critical load and resistance parameters for computation of the reliability of other elements requires that the $F_{ub}$ and CFP must be considered for individual simulations. Finally, the goal of this study is to integrate results from routine design tools, methods, and software, rather than complex finite element models that are computationally expensive, which renders them unsuitable for a design environment, especially in the case of highly nonlinear iterative solutions. Application to routine design tools will contribute additional capability to conduct reliability analysis for large-scale systems.

Based on these criteria, the Taylor series (Wolff 1994, Duncan 2000) method and the $2k+1$ (where $k =$ number of variables) point estimate method (Rosenbleuth 1975) were selected for calculation of global instability and flowthrough, respectively, for this study. The Taylor series method was selected for use in determining the probability of instability given its ease of application and applicability to slope stability reliability problems (Duncan 2000, USACE 2006). For flowthrough computations, the $2k+1$ method utilizing the margin of safety ($g(x) = \text{Capacity} - \text{Demand}$) approach was selected. The margin of safety approach utilized in the $2k+1$ analysis
produced values of reliability index and probability of unsatisfactory performance that appeared to be in reasonable agreement with the high margins of safety computed.

These two methods are compatible for use in the same model given the simulation technique used by both methods. For both methods, the expected value of factor of safety (i.e. margin of safety) is determined using expected values of all input variables in the analysis. Both methods also simulate 2k additional points on the response surface by each variable by $+1\sigma$ and $-1\sigma$ (where $\sigma$ is standard deviation) while holding all other variables at their expected values. The differences in the two methods are reflected in the computation of the reliability index, $\beta$, for each method (Figure 4.3 presents the governing equations). For this study, it is assumed that all variables are independent and uncorrelated.

Taylor Series

$$\sigma_{FS} = \sqrt{\sum_{i=1}^{N} \left( \frac{FS^+_i - FS^-_i}{2} \right)^2}$$

$$COV_{FS} = \frac{\sigma_{FS}}{FMLV}$$

$$\ln \left( \frac{FMLV}{\sqrt{1 + COV_{FS}^2}} \right)$$

$$\beta_n = \frac{1}{\ln(1 + COV_{FS}^2)}$$

$FMLV =$ most likely value of factor of safety computed with all variables equal to mean values
$\sigma_{FS} =$ standard deviation of factor of safety
$COV_{FS} =$ coefficient of variation for factor of safety
$FS^+_i, FS^-_i =$ factor of safety computed for variable $i$ at $+1\sigma$ and $-1\sigma$
$\beta_n =$ lognormal reliability index

Rosenbleuth 2k+1

$$COV_{MS} = \frac{1}{\prod_{i=1}^{K} \left( 1 + \left( \frac{ms^+_i - ms^-_i}{ms^+_i - ms^-_i} \right)^2 \right)^2 - 1}$$

$$\beta = \frac{1}{COV_{MS}}$$

$COV_{MS} =$ coefficient of variation for margin of safety
$ms^+_i, ms^-_i =$ margin of safety computed for variable $i$ at $+1\sigma$ and $-1\sigma$
$\beta =$ reliability index

Figure 4.3  Taylor Series (Wolff 1994) and Rosenbleuth (1975) 2k+1 reliability equations.
4.3 Methodology to Determine Unbalanced Force for Reliability Analysis

4.3.1 Case Study Model Setup

The dataset presented in Chapter 3 was used to develop the simplified approach to compute an unbalanced force for reliability analysis. In that analysis, the statistics of test data from undisturbed borings for unit weight and undrained shear strengths were adjusted for the upper bound of measurement error and, in the case of undrained shear strength, variances were computed for two cases, namely adjusting for spatial averaging effects and not considering them. Unit weight, undrained shear strength, and hurricane surge elevation were all modeled as random variables in the stability analysis (see Table 4.1). For the subsequent flowthrough analysis, the pile width \( (B) \) was also treated as a random variable. All other parameters (e.g. unit weight of water (9.897 kN/m\(^3\)) and transverse pile spacing (1.70 m) were treated as constants. Critical

<table>
<thead>
<tr>
<th>Variable(^b)</th>
<th>( \mu )</th>
<th>( \mu + \sigma )</th>
<th>( \mu - \sigma )</th>
<th>( \mu + \sigma )</th>
<th>( \mu - \sigma )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.2%: ( E_{\text{surge}} )</td>
<td>3.78</td>
<td>4.24</td>
<td>3.32</td>
<td>4.24</td>
<td>3.32</td>
</tr>
<tr>
<td>0.5%: ( E_{\text{surge}} )</td>
<td>3.26</td>
<td>3.63</td>
<td>2.89</td>
<td>3.26</td>
<td>2.89</td>
</tr>
<tr>
<td>1%: ( E_{\text{surge}} )</td>
<td>2.83</td>
<td>3.10</td>
<td>2.56</td>
<td>3.10</td>
<td>2.56</td>
</tr>
<tr>
<td>2%: ( E_{\text{surge}} )</td>
<td>2.29</td>
<td>2.47</td>
<td>2.11</td>
<td>2.47</td>
<td>2.11</td>
</tr>
<tr>
<td>( \gamma_{\text{soil},1} )</td>
<td>16.4</td>
<td>18.8</td>
<td>13.8</td>
<td>18.8</td>
<td>13.8</td>
</tr>
<tr>
<td>( \gamma_{\text{soil},2} )</td>
<td>13.3</td>
<td>15.3</td>
<td>11.4</td>
<td>15.3</td>
<td>11.4</td>
</tr>
<tr>
<td>( \gamma_{\text{soil},3} )</td>
<td>17.2</td>
<td>18.1</td>
<td>16.5</td>
<td>18.1</td>
<td>16.5</td>
</tr>
<tr>
<td>( \gamma_{\text{soil},4} )</td>
<td>16.9</td>
<td>18.2</td>
<td>15.7</td>
<td>18.2</td>
<td>15.7</td>
</tr>
<tr>
<td>( \gamma_{\text{soil},5} )</td>
<td>17.6</td>
<td>18.7</td>
<td>16.5</td>
<td>18.7</td>
<td>16.5</td>
</tr>
<tr>
<td>( \gamma_{\text{soil},6} )</td>
<td>16.5</td>
<td>17.5</td>
<td>15.5</td>
<td>17.5</td>
<td>15.5</td>
</tr>
<tr>
<td>( \gamma_{\text{soil},7} )</td>
<td>17.9</td>
<td>19.6</td>
<td>16.2</td>
<td>19.6</td>
<td>16.2</td>
</tr>
<tr>
<td>( \gamma_{\text{soil},8} )</td>
<td>18.6</td>
<td>19.4</td>
<td>18.0</td>
<td>19.4</td>
<td>18.0</td>
</tr>
<tr>
<td>( s_{u,1} )</td>
<td>43.1</td>
<td>74.8</td>
<td>11.4</td>
<td>58.9</td>
<td>27.4</td>
</tr>
<tr>
<td>( s_{u,2} )</td>
<td>12.9</td>
<td>20.4</td>
<td>5.4</td>
<td>17.0</td>
<td>8.8</td>
</tr>
<tr>
<td>( s_{u,3} )</td>
<td>14.4</td>
<td>24.7</td>
<td>4.1</td>
<td>23.4</td>
<td>5.4</td>
</tr>
<tr>
<td>( s_{u,4} )</td>
<td>13.3</td>
<td>17.6</td>
<td>9.1</td>
<td>17.1</td>
<td>9.5</td>
</tr>
<tr>
<td>( s_{u,5} )</td>
<td>21.4</td>
<td>32.1</td>
<td>10.7</td>
<td>27.6</td>
<td>15.2</td>
</tr>
<tr>
<td>( s_{u,6} )</td>
<td>26.9</td>
<td>36.5</td>
<td>17.3</td>
<td>31.8</td>
<td>22.0</td>
</tr>
<tr>
<td>( s_{u,7} )</td>
<td>25.9</td>
<td>36.1</td>
<td>15.7</td>
<td>31.9</td>
<td>19.9</td>
</tr>
<tr>
<td>( s_{u,8} )</td>
<td>43.1</td>
<td>62.4</td>
<td>23.8</td>
<td>51.2</td>
<td>35.0</td>
</tr>
</tbody>
</table>

\(^a\) Case A

\(^b\) Case B

\(^c\) Units and Variables: \( E_{\text{surge}} = \) Elevation of hurricane storm surge, m; \( \gamma_{\text{soil}} = \) unit weight of soil, kN/m\(^3\); \( s_{u} = \) undrained shear strength, kPa
dimensions of the floodwall as well as top and bottom elevations of soil strata are shown in Figure 4.4.

Figure 4.4 Critical dimensions of hurricane risk reduction floodwall for reliability analysis.

Hurricane surge elevations were drawn from the dataset that forms the basis of USACE guidance (USACE 2007) for different annual probability of exceedance surge elevations (e.g. 0.2%, 0.5%, 1%, and 2%). The standard deviation for each annual probability of exceedance is the standard deviation provided by the numerical models used to compute surge elevation (USACE 2007). For this chapter, future elevations (present day plus 50-years) that incorporate
estimates of sea-level rise and subsidence are considered. These elevations were determined by adding the estimate for sea-level rise and subsidence to present-day surge estimates (USACE 2007).

4.3.2 Initial Model Runs

Using SLOPE/W (Geo-Slope 2008), simulations were run with all variables set at expected values and with each variable adjusted up and down by one standard deviation while holding all other variables at the expected value (Table 4.1). The factor of safety for each simulation and the associated critical failure surface was noted. Four annual probabilities of exceedance were analyzed, 0.2%, 0.5%, 1%, and 2%, and for each annual probability of exceedance, two cases were considered: Case A and Case B. Case A ignored spatial averaging effects for undrained shear strengths, adjusting the inherent soil variability only by the upper bound of measurement error, and Case B utilized the reduced variability afforded by spatial averaging of undrained shear strengths as adjusted for the upper bound of measurement error (Chapter 3). All other random variables remained the same for both Case A and Case B simulations.

The computed factor of safety for each simulation is presented in Figure 4.5. For design, an $FS_{all}$ based upon historical performance or engineering judgment is used as the metric for unsatisfactory performance. In the case of USACE, $FS_{all}$ for global stability ranges from 1.4 for resiliency checks for mainline risk reduction features (i.e. top of wall or design grade of levee) to 1.5 for the design surge elevation at the 90% confidence interval when used in conjunction with USACE guidelines for establishing design shear strengths from test data (USACE 2008). For reliability analysis, $FS = 1.0$ is typically used as the failure metric.

Figure 4.5 presents the results of the $2k+1$ global stability analysis simulations. For Simulations 0 through 18, all values of $s_u$ were set at mean values and no other variables were
affected by variance reductions. As such, these simulations exhibited no change in $FS$ from Case A to Case B. For Simulations 31 through 34, the CFP was located in boundaries in the upper strata so variations in $s_u$ for the deeper strata did not affect the computed $FS$. Simulations 22, 24, and 28 represented the $\mu - \sigma$ simulation for $s_u$ for Strata 2, 3, and 5, respectively. For these simulations, the low values of $s_u$ for the $\mu - \sigma$ simulation resulted in shifts of the CFP from the mode of El. -24 to other depths, indicating that the $s_u$ in each layer governed the results for each successive simulation. Because of the larger value of $\sigma$ for the $s_u$ simulations, the Case A results

![Figure 4.5](image.png)

Figure 4.5  Computed global stability factor of safety by simulation for each hurricane storm surge exceedance.

produced higher $FS$ than Case B for the $\mu + \sigma$ simulations and lower $FS$ than Case B for the $\mu - \sigma$ simulations. Lastly, the results presented in Figure 4.5 show that the use of mean values of $s_u$
results in much higher factors of safety than USACE $FS_{all}$. None of the simulations produced $FS < 1.0$ and few produced $FS < USACE F_{all}$.

4.3.3 Target Factor of Safety

The next phase of the analysis is the computation of $\sigma_{FS}$, $F_{MLV}$ (or $E[F.S.]$), and $\beta_{ln}$ for each annual probability of exceedance and case. From $\beta_{ln}$, the probability of instability given a specific surge elevation ($P(I|S)$) is estimated using the inverse of the standard normal distribution. The term probability of instability is utilized here in lieu of probability of failure given that slope failure is assumed to be resisted by the floodwall pile foundation. The probability is thus the probability that there will be an additional loading on the floodwall rather than a complete failure of the foundation and a corresponding discounting of all axial and lateral pile capacity above the critical failure plane.

Using the factors of safety computed for each simulation and case, the Taylor Series method was used to compute $\beta_{ln}$, which was then used to compute $P(I|S)$ for use in the final “system” reliability check. Values of $\beta_{ln}$ computed for Case A ranged from a low of 1.36 for the 0.2% exceedance surge elevation to a high of 2.30 for the 2% exceedance surge elevation. For Case B, the values ranged from a low of 2.24 to a high of 3.42. These values are compared to those presented in Table 4.2 (from USACE 1997), which presents $\beta$ (and associated $P_{up}$, $E[F.S.]$) and expected performance level. In the context of Table 4.2, both terms are defined as described in section 4.2. As shown in the table, a $\beta$ of 2.5 is considered below average performance and values of 2.0 and below reflect poor performance (2.0) to imminent instability (1.0). Based on this observation, it is clear that $FS = 1.0$ and $FS_{all}$ could not be used as the metric for computing unbalanced load. Instead, a different factor of safety that achieves a target performance level that could be considered the threshold between a stable and unstable section should be determined.
Table 4.2  Target reliability indices and probability of unsatisfactory performance (from USACE (1997)).

<table>
<thead>
<tr>
<th>Expected Performance Level</th>
<th>$\beta$</th>
<th>$P_{ue}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>High</td>
<td>5.0</td>
<td>0.0000003</td>
</tr>
<tr>
<td>Good</td>
<td>4.0</td>
<td>0.00003</td>
</tr>
<tr>
<td>Above Average</td>
<td>3.0</td>
<td>0.001</td>
</tr>
<tr>
<td>Below Average</td>
<td>2.5</td>
<td>0.006</td>
</tr>
<tr>
<td>Poor</td>
<td>2.0</td>
<td>0.023</td>
</tr>
<tr>
<td>Unsatisfactory</td>
<td>1.5</td>
<td>0.07</td>
</tr>
<tr>
<td>Hazardous</td>
<td>1.0</td>
<td>0.16</td>
</tr>
</tbody>
</table>

For a given annual probability of exceedance hurricane storm surge, the set of Taylor Series simulations used to compute $\beta_{ln}$ each utilize a prescribed value of each random variable (whether that is $\mu$, $\mu + \sigma$, or $\mu - \sigma$). A factor of safety is determined for each simulation, but because of the use of statistical mean values for unit weight and undrained shear strength, the factor of safety that is computed for each iteration is no longer $FS_{scN}$. As such the published value of $FS_{all}$ or $FS_{all} = 1.0$ is no longer a good metric for assessing whether instability exists. Instead, a new threshold, or target, factor of safety ($FS_{target}$) must be determined.

From the combined results of all simulations in the set $\sigma_{FS}$, $F_{MLV}$, and $\beta_{ln}$ are calculated. The method used here develops $FS_{target}$ to be used with a specific set of simulations only. The $FS_{target}$ will vary for different sets of simulations. For this method, once the set of simulations are completed, they are not changed. None of the variables are adjusted and no computed factor of safety for a given simulation is changed. Consequently, $\sigma_{FS}$ does not change, either. A new $FS_{target}$ can be determined, however, by changing the performance level to a $\beta_{ln}$ that can be expected to produce instability. This $FS_{target}$ can then be used to assess which individual simulations in the set under consideration can be said to have UBL. Then those individual simulations are re-analyzed and the USACE methodology employed to determine the UBL.
The first step, however, is the selection of the new target performance level; i.e., target $\beta_{ln}$. This value must produce a reasonable expectation that the performance of the foundation will result in instability. From Table 4.2, values of reliability index of 2.5 reflect fair to excellent performance, which qualitatively translates into a low expectation of instability. This assertion is reinforced by Paikowsky (2004), where for soil structure interaction problems such as redundant pile systems a target $\beta$ of 2.33 was recommended for design. Based upon this, a target reliability index of 2.0 was selected as a reasonable threshold for computing $F_{ub}$. The steps to determine a factor of safety that achieves a low target performance level are:

1. Establish the simulations required for the analysis
2. Perform slope stability analysis for all simulations, documenting each computed factor of safety, and using the Taylor Series method, compute $F_{MLV}$, $\sigma_{FS}$, and $\beta_{ln}$
3. Establish a target $\beta_{ln}$
4. Holding $\sigma_{FS}$ constant at its original value and substituting the new target $\beta_{ln}$ into the equation, solve for a new $F_{MLV}$, which is the new target factor of safety ($F_{S_{target}}$).

This procedure was used to compute $F_{S_{target}}$ for different target $\beta_{ln}$ using the simulations run for each annual probability of exceedance surge elevation and each case. The results are presented in Table 4.3.

To further assess the reasonableness of the selection of a $\beta_{ln}$ of 2.0, the target factors of safety presented in Table 4.3 were compared to the factors of safety presented in Figure 4.5. Using Case A simulations to illustrate the results, a total of 35, 30, 28, and 7 individual simulations would be considered “unstable” using a target $\beta_{ln}$ of 2.0 for 0.2%, 0.5%, 1%, and 2% exceedance, respectively. For $\beta_{ln}$ of 1.5, these numbers reduce to 29, 8, 5, and 4 and for $\beta_{ln} = 1.0$, they reduce to 6, 5, 3, and 2. To produce results that would most closely represent expected
values, either 2.0 or 1.5 would appear to be most appropriate. However, the target $\beta_{th}$ of 2.0 was ultimately selected to provide the best threshold for assessing whether a section is stable or unstable. The corresponding target factors of safety are in bold font in Table 4.3. These were used to compute unbalanced loads.

Table 4.3  Computed target factors of safety for different values of reliability index for case study.

<table>
<thead>
<tr>
<th>Case</th>
<th>$P_a$</th>
<th>Above Average ($\beta=3.00$)</th>
<th>Below Average ($\beta=2.50$)</th>
<th>Poor ($\beta=2.00$)</th>
<th>Unsatisfactory ($\beta=1.50$)</th>
<th>Hazardous ($\beta=1.00$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>0.20%</td>
<td>2.51</td>
<td>2.32</td>
<td>2.11</td>
<td>1.90</td>
<td>1.68</td>
</tr>
<tr>
<td></td>
<td>0.50%</td>
<td>2.61</td>
<td>2.41</td>
<td>2.19</td>
<td>1.97</td>
<td>1.73</td>
</tr>
<tr>
<td></td>
<td>1%</td>
<td>2.68</td>
<td>2.47</td>
<td>2.24</td>
<td>2.01</td>
<td>1.76</td>
</tr>
<tr>
<td></td>
<td>2%</td>
<td>2.85</td>
<td>2.62</td>
<td>2.38</td>
<td>2.12</td>
<td>1.85</td>
</tr>
<tr>
<td>B</td>
<td>0.20%</td>
<td>2.06</td>
<td>1.92</td>
<td>1.77</td>
<td>1.62</td>
<td>1.46</td>
</tr>
<tr>
<td></td>
<td>0.50%</td>
<td>2.15</td>
<td>2.00</td>
<td>1.84</td>
<td>1.67</td>
<td>1.50</td>
</tr>
<tr>
<td></td>
<td>1%</td>
<td>2.22</td>
<td>2.06</td>
<td>1.90</td>
<td>1.72</td>
<td>1.54</td>
</tr>
<tr>
<td></td>
<td>2%</td>
<td>2.37</td>
<td>2.15</td>
<td>2.01</td>
<td>1.82</td>
<td>1.61</td>
</tr>
</tbody>
</table>

*Case A denotes no variance reduction for spatial averaging; Case B includes variance reduction.*

**4.4 Probabilities of Failure and Reliability Index for Flowthrough**

The demand, or in this instance unbalanced force, is dependent upon the probability of experiencing the hurricane storm surge ($P(S)$) and the $P(I|S)$, computed as described in Section 4.3.3 (page 72). The probability of experiencing the hurricane storm surge is based upon service life and is computed with Equation 4.1.

$$P(S) = 1 - [1 - P_a]^n$$  \hspace{1cm} (4.1)

where:

$P_a =$ annual probability of hurricane surge exceedance (e.g. 0.002, 0.005, 0.01, or 0.02)

$n =$ service life or design life of structure in years

Using the USACE model for flowthrough resistance, unsatisfactory performance occurs if the resistance computed for either Flowthrough Check 1 (passive pile resistance) or
Flowthrough Check 2 (soil flowthrough between pile rows in the direction of loading) is unsatisfactory to resist the demand. Consequently, they will be considered in series. As noted in USACE (2003), soft clays such as those found in the area of this case study exhibit ductile stress-strain behavior. Since the elements considered here are geotechnical and are in the region occupied by soft clay strata, all “system” elements are considered ductile for this analysis. Therefore the probability of unsatisfactory performance, $P_{up}$, can be computed using Equation 4.2. It should be noted, however, that soil stress-strain behavior should be reviewed for site-specific conditions. If stiff clays or other soil types that exhibit brittle behavior are present, the “system” equations will need to be revisited. Equation 4.2 is derived from idealized relationships for series and parallel systems. The two flowthrough mechanisms are modeled as elements in series given that unsatisfactory performance for either limit state translates into unsatisfactory element performance (Figure 4.6).

![Idealized reliability model for global instability and flowthrough limit states.](image)

Figure 4.6  Idealized reliability model for global instability and flowthrough limit states.

$P(S)$ and $P(I|S)$ are modeled as being in parallel with one another because both the given hurricane storm surge elevation must occur and there must be global instability for either flowthrough mechanism to be a concern. Equation 4.2 is then:

$$P_{up} = P(S)P(I|S)[1-(1-P(F1|I\cap S))(1-P(F2|I\cap S))]$$ (4.2)
where:

\[ P(F1|I \cap S) = \text{probability of unsatisfactory performance for flowthrough mechanism 1 (F1)} \]
given a specific hurricane storm surge elevation and global instability

\[ P(F2|I \cap S) = \text{probability of unsatisfactory performance for flowthrough mechanism 2 (F2)} \]
given a specific hurricane storm surge elevation and global instability

Figure 4.7 presents \( P(I|S) \) versus the \( E[F.S.] \) for Case A and Case B. As shown from the figure, designs for infrequent events have a much greater probability of instability when not accounting for spatial correlation. For more frequent events, the difference in results is not as pronounced. As shown in the figure, both cases have the same \( F_{MLV} \), but the variation in computed factor of safety resulting from the use of the higher COV introduces greater uncertainty that drives \( P(I|S) \) higher. When using point estimate methods, the more consistent the value of the performance function (in this case either factor of safety or margin of safety), the lower the computed \( P_{up} \).

![Figure 4.7](Image)

Figure 4.7 Probability of global instability versus expected value of F.S. for different surge exceedances without (A) and with (B) variance reductions for spatial averaging.
To illustrate the difference in consistency introduced by not considering spatial correlation, Figure 4.8 presents the difference in computed factor of safety when adjusting individual critical variables for the 0.2% annual exceedance hurricane storm surge case by +/- 1\(\sigma\). As shown by the figure, for some soil strata the difference in factor of safety is much larger for Case A than Case B. Strata 3 and 4 do not exhibit as much change because the thickness of each stratum is very close to the scale of fluctuation, which translates into a small variance reduction applied to \(s_u\). This means that Case A and Case B are virtually identical for these two strata. For Stratum 6, the variance reduction to \(s_u\) is noticeable, but does not result in large differences in factor of safety even for Case A. However, the small difference exhibited for Case A and the no change exhibited for Case B indicates that the -1\(\sigma\) simulation for Case A puts \(s_u\) near the threshold at which Stratum 6 becomes the controlling stratum for the stability analysis. For the same simulation for Case B, \(s_u\) is sufficiently high that the critical failure surface manifests itself at the boundary between two of the upper strata. Stratum 1 had the largest variability in \(s_u\) and also realized large variance reductions due to the small scale of fluctuation, as shown by the range of \(s_u\) between +1\(\sigma\) and -1\(\sigma\). For Stratum 1, the Case A range is 63.4 kPa (1,324 psf) compared to the Case B range of 31.5 kPa (658 psf). This difference in shear strength translated into very different results in factor of safety. Stratum 2 did not have a drastic change in range of \(s_u\), as shown by the Case A range being 14.9 kPa (312 psf) compared to the Case B range being 8.2 kPa (172 psf). However, the drastic change in factor of safety would seem to indicate the analysis is extremely sensitive to even small changes in \(s_u\) for this stratum, particularly to changes in the -1\(\sigma\) simulation.

Table 4.4 presents the inputs for each stage of the reliability computation and the resulting \(P_{up}\) and \(\beta\) for \(n = 100\) years. When factoring in the probability of the surge elevation
Note: Only parameters with $|\Delta FS| > 0$ shown.

$|\Delta FS| = \text{Parameter}^+ - \text{Parameter}^-$

Figure 4.8 Change in factor of safety due to change in variable by +/- one standard deviation without variance reduction for spatial averaging (A) and with variance reduction (B) for 0.2% surge exceedance.

associated with the annual exceedance, the computed $\beta$ varies from 2.58 to 2.81 for Case A and from 4.07 to 4.69 for Case B. When compared to the performance levels shown in Table 4.2, inclusion of spatial averaging can mean the difference between “Good” performance and “Below Average” for a given reliability analysis.

Table 4.4 Probabilities of failure and reliability indices for flowthrough for a 100-year design life.$^a$

| Case$^b$ | $P_a$ | $P(S)$ | $P(I\cap S)$ | $P(F1|S\cap I)$ | $P(F2|S\cap I)$ | $P_{\beta}$ | $\beta$ |
|---------|-------|---------|--------------|-----------------|-----------------|------------|-------|
| A       | 0.181 | 0.095   | 8.72E-02     | 1.52E-01        | 1.85E-01        | 4.89E-03   | 2.58  |
|         | 0.394 | 0.222   | 4.44E-02     | 2.73E-03        | 1.09E-03        | 3.09E-04   | 3.54  |
|         | 0.634 | 0.395   | 6.64E-02     | 1.95E-01        | 1.73E-01        | 3.38E-03   | 2.71  |
|         | 0.867 | 0.636   | 1.09E-02     | 1.09E-01        | 1.70E-01        | 2.46E-03   | 2.81  |
| B       | 0.181 | 0.095   | 1.27E-02     | 2.73E-03        | 2.73E-03        | 2.34E-05   | 4.07  |
|         | 0.394 | 0.222   | 1.09E-03     | 1.09E-03        | 6.10E-03        | 6.61E-06   | 4.27  |
|         | 0.634 | 0.395   | 8.28E-04     | 5.95E-04        | 5.13E-03        | 1.00E-06   | 4.53  |
|         | 0.867 | 0.636   | 3.09E-04     | 4.56E-04        | 4.76E-03        | 1.40E-06   | 4.69  |

$^aP_a = \text{probability of annual exceedance of surge; } S = \text{probability of exceedance for surge for design life; } I = \text{global instability; } F1 = \text{flowthrough mechanism #1; } F2 = \text{flowthrough mechanism #2}$

$^b$Case A denotes no variance reduction for spatial averaging; Case B includes variance reduction.
The overall $\beta$ for each case investigated for $n = 100$ and $n = 50$ are both presented in Figure 4.9. As shown by the plot, when compared to design life, spatial averaging has a greater impact on $\beta$ for this geological setting. There was, however, a small increase in $\beta$ associated with the shorter exposure period for both Case A and Case B. It is worthy of note, that referring back to Paikowsky (2004), where for redundant pile foundations the recommended $\beta$ was on the order of 2.33, the results associated with not considering spatial averaging effects still produce a level of reliability consistent with that utilized in other soil-structure interaction problems. Furthermore, the results for Flowthrough Checks 1 and 2 before including the probability of experiencing the surge, appear to generally be in line with the range presented for the global stability results presented by El-Ramly et al. (2002). For this study, for example, the 0.2% annual probability of exceedance produced reliability indices on the order of 2.78 and 2.43 for Flowthrough Checks 1 and 2, respectively. The El-Ramly, Morgenstern et al results presented were on the order of 2.32 and 2.42, depending upon the reliability method used.

![Figure 4.9](image-url)

Note: $n = \text{design or service life}$

**Figure 4.9** Overall reliability index for flowthrough for different annual exceedances of hurricane storm surge.
4.5 Conclusions

This chapter developed a method for computing an unbalanced force for use in point estimate simulation methods for reliability analysis of hurricane risk reduction structures subjected to global instability. It also highlighted the importance of accounting for spatial correlation effects in soils through reliability analysis for the flowthrough condition presented in the USACE unbalanced load methodology (USACE 2008). The findings of this study can be summarized in the following:

1. Because factors of safety computed using other than nominal values of geotechnical parameters are much higher than 1.0 (and in several instances higher than $F_{S\text{All}}$), the back-calculation of a target factor of safety using a preset performance level is required to set a new threshold for assessing whether the foundation is globally stable or unstable. Using the new threshold, the unbalanced force necessary to maintain system stability can be computed. A target reliability index of 2.0 is recommended for this purpose. However, factors of safety corresponding to other target performance levels were also presented.

2. Not considering the variance reductions that can be taken due to the effects of spatial correlation on undrained shear strengths results in higher probabilities of global instability for a given hurricane surge exceedance, with a pronounced difference when considering lower annual exceedance probability surge levels. When carried through the full calculation, it also results in a pronounced difference in expected performance levels as measured by the reliability index for flowthrough. Not considering the spatial correlation resulted in reliability indices in the 2.5 to 3.0 range while accounting for spatial correlation resulted in reliability indices greater than 4.0.
3. Compared to spatial correlation effects, varying the design or service life of a structure resulted in little difference in reliability index for the investigated n=50 and n=100 year cases.

4. The analysis presented, especially the method for developing a threshold for assessing the stability of a section when computing other than a nominal factor of safety, will aid in the reliability analysis of structures whose pile foundations are designed to resist global instability and unbalanced loads. The observations about the use of spatial averaging will also aid those interpreting the results of reliability analysis of embankments or foundations with or without spatial averaging effects.
CHAPTER 5: RELIABILITY ANALYSIS OF PILE-FOUNDED HURRICANE RISK REDUCTION STRUCTURES

5.1 Introduction

The development of performance-based design criteria requires an understanding of the reliability and risk of the structures under consideration. Calibration of modern design codes (AISC 1989, ACI 2008, AASHTO 2012) rely upon the quantification of reliability and the quantification of risk is becoming increasingly utilized for communication of expected system performance to facility owners, designers, risk managers, and the general public. Subsequent to 2005, quantification of risk for hurricane risk reduction systems was used by the U.S. Army Corps of Engineers (USACE) to communicate the risk of flooding given different system heights and different annual probability of exceedance hurricane storm surge events (IPET 2009). USACE has been engaged in the quantification of reliability since the 1990s, emphasizing the use of comparative reliability to prioritize infrastructure rehabilitation (USACE 1992, 1995). In risk models developed since 2005 (e.g., USACE Interagency Performance Evaluation Taskforce, IPET (2009)), much attention has been given to hurricane risk reduction levees and geotechnical aspects of I-walls due to their inherent fragility when exposed to extreme events. Pile-founded structures have not received the same level of scrutiny in the context of system analysis given their inherent robustness when compared to levees and I-walls. To provide more quantitative support for reliability model results, additional attention to the reliability of pile-founded structures is warranted, especially those whose pile foundations have been designed to resist global instability.

This chapter presents a methodology for quantifying the reliability of the pile-founded floodwall designed in accordance with the Hurricane and Storm Damage Risk Reduction System (HSDRRS) Design Guidelines (USACE 2008). The HSDRRS Design Guidelines were developed to assure consistency of design for the reconstruction of the Greater New Orleans
HSDRRS in the wake of Hurricane Katrina. The criteria were developed using separate discipline-specific teams of experts in the fields of hydrology and hydraulics engineering, geotechnical engineering, and structural engineering. The criteria included changes in design methodology (e.g. the use of Joint-Probability Method – Optimum Sampling (USACE 2007) for determination of hurricane storm surge elevations, the use of the Spencer (1967) method for global stability analysis, and the development of new unbalanced load methodology (Varuso 2010)) and metrics (factors of safety, load factors, and resistance factors) that historically resulted in acceptable performance. These criteria were reviewed and commented upon by independent experts. However, the time-consuming process of calibration of the criteria to a target reliability index was not performed. Not knowing the level of reliability is an important concern. A higher reliability than intended could introduce increased robustness into the finished HSDRRS structure, which could, in turn, result in dramatic increases in cost affecting the viability of not-yet-funded projects.

The methodology presented in this chapter provides a framework for quantifying reliability of the entire pile-founded structure from the top of the superstructure to the tips of the foundation piles for pile-founded hurricane risk reduction structures subjected to global instability. The proposed method draws upon current U.S. Army Corps of Engineers design practices (USACE 2008), the Rosenbleuth (1975) point estimate method for computing reliability indices, and an event tree framework to provide a technique for computing the overarching system reliability. The method uses designer-friendly software commonly utilized by the design community for pile group analysis and slope stability to develop input to the overarching reliability model. The methodology is illustrated using an example of a pile-founded T-wall subjected to hurricane storm surge hydrostatic loading for four different annual probabilities of exceedance hurricane storm surge elevations. The example draws upon the
This chapter will provide researchers a method by which to combine current design methods with reliability analysis to enable greater participation of design engineers in the computation of structure reliability. The methodology in this chapter can also be used by researchers to review the reliability of structures designed using the 2008 Hurricane and Storm Damage Risk Reduction System (HSDRRS) Design Guidelines (USACE 2008), with an eye toward refinement of deterministic design criteria where warranted. The methodology can also be adapted more generally to other pile-founded structures designed to resist lateral loads through soil-structure interaction (such as retaining walls, bridge abutments, mooring facilities, impact dolphins) for application beyond just hurricane risk reduction structures.

5.1.1 Pile Founded Hurricane Risk Reduction Structures

Pile-founded hurricane risk reduction structures typically consist of a cast-in-place concrete superstructure supported by a pile foundation consisting of any one of several typical pile types including steel pipe piles, steel H-piles, or precast prestressed concrete piles (typically square cross-section). Steel H-piles are common in the Greater New Orleans HSDRRS, especially where resistance to global instability was required. For pile-founded T-walls, the superstructure is as simple as a base slab and cantilever wall stem. For larger pile-founded structures, the concrete superstructure can be quite complex.

Some of the typical loads resisted by hurricane risk reduction structures are presented in Figure 5.1. Hurricane risk reduction structures are typically subjected to loads from the weights of the concrete superstructure, soil bearing directly on the base slab, any scour protection bearing on the base slab, and water (both groundwater on the protected side of the structure and hurricane storm surge on the flood side). The base slab is subjected to uplift pressure from the hurricane
storm surge and design typically assumes two separate distributions: (1) the pervious distribution, which assumes a partially effective sheet pile cut-off wall and a trapezoidal pressure distribution that linearly varies between the protected side water pressure and the flood side water pressure, and (2) the impervious distribution, which assumes a 100% effective sheet pile cut-off wall and a uniform pressure distribution equivalent to the flood side water pressure that extends between the centerline of the cut-off wall and the flood side edge of the base slab. Lateral loads include soil loads, groundwater pressure, hurricane storm surge hydrostatic pressure, hurricane storm surge wave loads, uniform debris loads, wind loads, and, where applicable, aberrant vessel (barge or pleasure-craft) impact forces. Construction live loads and negative skin friction loads on piles are also considered. Global instability is treated as a uniform load applied directly to the superstructure above the bottom of the base slab and directly to the foundation piles from the bottom of the base slab to the critical failure surface at some depth below the structure. Global instability is determined by the current USACE design methodology (USACE 2008), which has been validated with detailed numerical modeling and centrifuge tests (Abdoun and Sasanakul 2007, Varuso 2010).

Figure 5.1 Hurricane risk reduction T-wall and applied loads.
Limit states and resistance to applied loads are reflective of industry standards ((AISC 1989, ASCE 2006, ACI 2008, AASHTO 2012)) as modified or further adjusted by USACE Engineering Manuals, Engineering Technical Letters, Engineering Circulars, and, in the case of the Greater New Orleans HSDRRS, the HSDRRS Design Guidelines (USACE 2008). Given the uncertainty associated with hurricane loadings and the strict performance requirements for public safety, USACE criteria are intended to produce robust and resilient structures through the use of increased load factors, greater reductions for resistance, and higher safety factors for the design case. USACE criteria also include provisions for resiliency of structures when exposed to conditions beyond design conditions.

5.2 Reliability Analysis

Methods available for reliability problems include direct reliability analysis, First Order Second Moment (FOSM), First Order Reliability Methods (FORM), the Rackwitz-Fiessler method, point estimate methods (2k+1, Latin hypercube), and Monte Carlo Simulation (Nowak 1999, Nowak and Collins 2000, Christian 2004). Hurricane risk reduction structures in highly stratified soils can have large numbers of variables. Typical probability distributions for many of the typical variables can be found in the literature and are summarized in Chapter 2. However, some distributions may not be readily available. Furthermore, the computation of unbalanced load and its application to the pile foundation may use analysis techniques that either requires the use of different software packages that preclude development of a simplified model or a single complex finite element modeling software package. To enable greater participation by design professionals in the reliability analysis, emphasis here is on the former technique. Because of the potential limitations of knowledge of probability distributions and the stated purpose of this chapter to utilize design production tools, direct probability methods, FORM, Rackwitz-Fiessler, and Monte Carlo Simulation were not selected for this study. Chapter 4 utilized the Taylor series
(Wolff 1994, Duncan 2000) method (for global instability) and the 2k+1 point estimate method (Rosenbleuth 1975). This chapter extends the application of the 2k+1 point estimate method to all elements of the pile-founded hurricane risk reduction structure and utilizes the Taylor Series method for global instability. This chapter also utilizes the FOSM-Mean Value method to validate point estimate results for elements, which can easily be modeled. The governing equations for these three methods are summarized in Figure 5.2.

\[
\sigma_{FS} = \sqrt{\sum_{i=1}^{N} \left( \frac{FS_i - FS_i^{-}}{2} \right)^2}
\]

\[
COV = \frac{\sigma_{FS}}{FS_{MLV}}
\]

\[
\beta_n = \frac{\ln \left( \frac{F_{MLV}}{1 + COV^2} \right)}{\ln (1 + COV^2)}
\]

\[
COV_{MS} = \left( \prod_{i=1}^{K} \left( 1 + \left( \frac{ms_i^+ - ms_i^-}{ms_i^+ - ms_i^-} \right)^2 \right) \right)^{1/2} - 1
\]

\[
\beta = \frac{1}{COV_{MS}}
\]

\[
a_i = \frac{\partial g}{\partial X_i} \bigg|_{evaluated \ at \ mean \ values}
\]

\[
\text{COV}_{MS} = \text{coefficient of variation for margin of safety}
\]

\[
MS_i^+, MS_i^- = \text{margin of safety computed for variable } i \text{ at } + \text{ one and } - \text{ one standard deviation}
\]

\[
\beta = \text{reliability index}
\]

\[
FS_{MLV} = \text{most likely value of factor of safety computed with all variables equal to mean values}
\]

\[
\sigma_{FS} = \text{standard deviation of factor of safety}
\]

\[
COV_{FS} = \text{coefficient of variation for factor of safety}
\]

\[
FS_i^+, FS_i^- = \text{factor of safety computed for variable } i \text{ at } + \text{ one and } - \text{ one standard deviation}
\]

\[
\beta_n = \text{lognormal reliability index}
\]

(Figure 5.2) Taylor Series (Wolff 1994), Rosenbleuth (1975) 2k+1, and First Order Second Moment (FOSM) – Mean Value (Nowak and Collins 2000) reliability equations.

5.3 Methodology

The method used for this analysis builds upon the general conceptual framework used for risk analysis (Kuijper and Vrijling 1998, IPET 2009). For those methods, the following steps are performed as the first step toward quantifying risk:
1. Identify analysis objectives
2. Develop the system description
3. Compute the failure probability

These three steps represent the reliability analysis component of a larger risk analysis. These are expanded upon in Figure 3 and steps specific to the analysis of a pile-founded floodwall subjected to an unbalanced load are added to address the typical limit states analyzed for hurricane risk reduction structures. From the initial steps described in Figure 5.3, an event tree model (Figure 5.4) is used to model the overall structure “system” reliability.

Figure 5.3   Modeling procedure.

In the context of this chapter, the objective of the analysis is quantification of the structure reliability for specific annual probability of exceedance hurricane storm surge
elevations. Definition of the system in this case involves developing the statistical parameters that define the structural resistance, developing the statistical depiction of the foundation conditions for the specific site of consideration and developing the statistics of the hurricane storm surge loadings. While values of structural material and geometrical properties draw upon published values (Ellingwood et al. 1980, Nowak 1999, Nowak and Szersze 2003, Galambos 2004), such as those presented in the recommendations of Chapter 2, development of the description of the geotechnical components of the system are more involved. Modeling the sources of geotechnical uncertainty and accounting for spatial averaging are important to obtaining a more complete estimate of reliability. Chapter 3 presents the computation of scale of fluctuation using limited, design-quality datasets and Chapter 4 highlights the importance of accounting for spatial averaging effects. If inadequate data exist, consideration should be given to utilizing published values of scales of fluctuation (at least vertical scale of fluctuation) for use

Figure 5.4  Event tree for floodwall system.
in developing spatial averages. Phoon and Kulhawy (1999b), IPET (2009), and Hicks and Samy (2002) provide values of scale of fluctuation for different soil types. However, published values should be weighed against site geology and soil mineralogy to assure appropriate values are selected. Using the recommendations of Chapter 2, existing datasets for hurricane storm surge elevations are utilized by this method.

Depending upon the objective of the reliability analysis, either a single probability of exceedance or multiple probabilities of exceedance hurricane surge events may be selected. In many cases, the 2%, 1%, and 0.2% annual probability of exceedance events are of the most interest.

Computation of the probability of failure is accomplished through the reliability model. Performance functions here use the models of resistance used for structural design codes (AISC 1989, ACI 2008) and the resistance model for global instability (USACE 2008, Varuso 2010). To satisfy the objective of providing the opportunity for design professionals to participate in the reliability modeling, two common software packages (SLOPE/W (Geo-Slope 2008) and GROUP (ENSOFT 2006)) are used in conjunction with the 2k+1 point estimate method (Rosenbleuth 1975) and Taylor Series method (Wolff 1994).

For this method, special attention is given to treatment of piles as structural elements if there is global instability. If there is instability, the piles are considered unsupported from the base slab to the critical failure plane (CFP) – the failure plane that produces the lowest deterministic factor of safety in a limit equilibrium analysis. In that instance, column action must be considered. For the structural axial compression capacity of steel piles, the Bjorhovde (1972) Curve 2 relationships are used. As noted by Ellingwood et al. (1980), the coefficient of variation (COV) for the ratio $P_{cr}/F_{y}A_{g}$ (critical buckling load/(static yield stress x gross column...
area) varies depending upon the slenderness ratio. If there is no global instability, piles are considered fully supported along their length.

As a simplification, only applicable interaction equations such as those with combined axial and bending stress in columns or combined shear and tensile breakout of tension anchors in the concrete base slab are used in the performance functions. In these instances, only maximum forces are used concurrently. For piles exposed to unbalanced loads, the assumption of no contribution of geotechnical capacity above the CFP supports this assumption, as the axial force in the pile will be constant in this zone. The method described in Chapter 4 is used to determine a target factor of safety for assessing whether global instability exists in the context of reliability analysis. Global stability analysis and the steps for determining unbalanced load (UBL), CFP, the vertical uniform load from the flood side natural ground at the floodwall to the CFP \( f_{ub} \), and the uniform load bearing directly on the flood side row of piles \( F_p \), are described in detail in Chapter 4.

Element reliabilities are then computed and combined into the system model developed using the event tree method (Christian 2004, USACE 2006). The floodwall is exposed to hurricane storm surge. If there is global instability, the system must survive flowthrough or the system exhibits unsatisfactory performance. If the system survives flowthrough, the stem must survive to transmit hurricane storm surge loads to the remaining elements. If the stem does not survive, then the system exhibits unsatisfactory performance. Similarly, the piles must survive to transmit loads to the base slab, which ties all elements together. For the loads to be fully transmitted, the base-to-pile connection must remain intact. If the base-to-pile connection survives, the base slab must also survive to exhibit satisfactory performance. Lastly, the base slab must remain within serviceable movement limits to complete the satisfactory performance. Since all probabilities must sum to unity and there is only one path to survival for this event tree,
the net probability of unsatisfactory performance is the summation of the individual element probabilities.

Idealized series and parallel system equations (Nowak and Collins 2000) are used. To account for elements that may behave somewhere between the idealized systems, system reliability should be bracketed by developing results for the element in series and developing a second set of results for the element in parallel. This approach can readily provide a range of possible results. An example of an element that could fall into this category is the pile foundation. If there are more than two rows of piles, there is a possibility of load redistribution should one row of piles fail.

5.4 Case Study

5.4.1 Element Reliabilities

To illustrate the methodology and to provide data on the reliability of a structure designed using the HSDRRS Design Guidelines (USACE 2008), an example using a hurricane risk reduction floodwall founded on three rows of steel HP14x89 piles is used. This case study assumes that all elements were constructed to the dimensions, lines, and grades shown on the plans and specifications. The floodwall superstructure is presented in Figure 5.5. Only the hydrostatic component of hurricane storm surge was considered for this example (i.e. no wind, waves, or impact forces). As such, due to symmetry of all elements, a 2-D, plane-strain modeling approach was used.

All design parameters were identified and hurricane storm surge statistics were obtained from USACE (2007). Annual probabilities of exceedance of 2%, 1%, 0.5%, and 0.2% were selected for use and hurricane storm surge elevations are treated as random variables. A mean value of surge elevation and a standard deviation model error were obtained from the storm surge models. For this effort, future elevations (present day plus 50-years) incorporating
estimates of sea-level rise and subsidence were used. These elevations were determined by adding the estimate for sea-level rise and subsidence to present-day surge estimates (USACE 2007).

Figure 5.5  Pile-founded hurricane risk reduction floodwall for example.

The ground geometry presented in Chapter 4 and the site geology analyzed in Chapter 3 were used to establish foundation conditions and spatial averages. The statistics of the structural parameters were obtained from the literature presented in Chapter 2. Since SLOPE/W and GROUP were utilized, the variables, constants, and moving constants were assigned based on the recommendations of Chapter 2. The example site contains soils that are treated as soft clays (using the definition in Terzaghi et al. (1996)). All site topography and concrete exterior dimensions were treated as constants. Soil strata limits were also treated as constants. All random variables were assumed independent and uncorrelated. Where data was unavailable on
professional factors or other parameters, a value of 1.0 was assumed. A complete listing of
geotechnical characterization of the project site, tabulation of model constants, and model
structural and hydraulic variables are presented in Tables 5.1, 5.2, and 5.3, respectively.

Table 5.1 Statistical characterization of project site\textsuperscript{a}.

<table>
<thead>
<tr>
<th>Variable</th>
<th>( \mu )</th>
<th>( \mu + \sigma )</th>
<th>( \mu - \sigma )</th>
</tr>
</thead>
<tbody>
<tr>
<td>( \gamma_{\text{soil,1}} )</td>
<td>0.104</td>
<td>0.120</td>
<td>0.088</td>
</tr>
<tr>
<td>( \gamma_{\text{soil,2}} )</td>
<td>0.085</td>
<td>0.097</td>
<td>0.073</td>
</tr>
<tr>
<td>( \gamma_{\text{soil,3}} )</td>
<td>0.110</td>
<td>0.115</td>
<td>0.105</td>
</tr>
<tr>
<td>( \gamma_{\text{soil,4}} )</td>
<td>0.108</td>
<td>0.116</td>
<td>0.100</td>
</tr>
<tr>
<td>( \gamma_{\text{soil,5}} )</td>
<td>0.112</td>
<td>0.119</td>
<td>0.105</td>
</tr>
<tr>
<td>( \gamma_{\text{soil,6}} )</td>
<td>0.105</td>
<td>0.111</td>
<td>0.099</td>
</tr>
<tr>
<td>( \gamma_{\text{soil,7}} )</td>
<td>0.114</td>
<td>0.125</td>
<td>0.103</td>
</tr>
<tr>
<td>( \gamma_{\text{soil,8}} )</td>
<td>0.119</td>
<td>0.123</td>
<td>0.115</td>
</tr>
<tr>
<td>( s_u,1 )</td>
<td>0.901</td>
<td>1.230</td>
<td>0.572</td>
</tr>
<tr>
<td>( s_u,2 )</td>
<td>0.270</td>
<td>0.356</td>
<td>0.184</td>
</tr>
<tr>
<td>( s_u,3 )</td>
<td>0.301</td>
<td>0.489</td>
<td>0.113</td>
</tr>
<tr>
<td>( s_u,4 )</td>
<td>0.278</td>
<td>0.358</td>
<td>0.198</td>
</tr>
<tr>
<td>( s_u,5 )</td>
<td>0.447</td>
<td>0.577</td>
<td>0.317</td>
</tr>
<tr>
<td>( s_u,6 )</td>
<td>0.562</td>
<td>0.664</td>
<td>0.460</td>
</tr>
<tr>
<td>( s_u,7 )</td>
<td>0.541</td>
<td>0.666</td>
<td>0.416</td>
</tr>
<tr>
<td>( s_u,8 )</td>
<td>0.900</td>
<td>1.069</td>
<td>0.731</td>
</tr>
</tbody>
</table>

\textsuperscript{a}Variables and Units:

\( \gamma_{\text{soil,i}} \) = unit weight of soil stratum \( i \), in pcf.
\( s_u,i \) = undrained shear strength of soil stratum \( i \), psf.
\( \mu \) = mean value
\( \sigma \) = standard deviation

Table 5.2 Model constants.

<table>
<thead>
<tr>
<th>Constant</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit Weight of Storm Surge: ( \gamma_{\text{surge}} ) (kcf)</td>
<td>0.063</td>
</tr>
<tr>
<td>Unit Weight of Protected Side Groundwater: ( \gamma_{\text{GW T}} ) (kcf)</td>
<td>0.0624</td>
</tr>
<tr>
<td>Elevation of Flood Side Embankment at Floodwall: ( E_{\text{oil,FS}} ) (ft, NAVD 88)</td>
<td>5</td>
</tr>
<tr>
<td>Elevation of Protected Side Embankment at Floodwall: ( E_{\text{oil,PS}} ) (ft, NAVD 88)</td>
<td>4.5</td>
</tr>
<tr>
<td>Elevation of Protected Side Scour Protection: ( E_{\text{conc,PS}} ) (ft, NAVD 88)</td>
<td>5</td>
</tr>
<tr>
<td>Elevation of Top of Floodwall Stem: ( E_{\text{Top Wall}} ) (ft, NAVD 88)</td>
<td>14</td>
</tr>
<tr>
<td>Elevation of Top of Floodwall Base Slab: ( E_{\text{Top Base}} ) (ft, NAVD 88)</td>
<td>4</td>
</tr>
<tr>
<td>Elevation of Bottom of Floodwall Base Slab: ( E_{\text{Bott Base}} ) (ft, NAVD 88)</td>
<td>1</td>
</tr>
<tr>
<td>Elevation of Protected Side Groundwater: ( E_{\text{GW T}} ) (ft, NAVD 88)</td>
<td>1</td>
</tr>
<tr>
<td>Overall Width of Floodwall Base Slab: ( \text{Width}_{\text{Base}} ) (ft)</td>
<td>14</td>
</tr>
<tr>
<td>Thickness of Floodwall Stem: ( \text{Width}_{\text{Stem}} ) (ft)</td>
<td>2</td>
</tr>
<tr>
<td>Width of Floodwall “Heel”: ( \text{Width}_{\text{Heel}} ) (ft)</td>
<td>2</td>
</tr>
<tr>
<td>Distance from Flood Side Face of Base Slab to Cut-off Sheeting: ( \text{Width}_{\text{Sheets,FS}} ) (ft)</td>
<td>4</td>
</tr>
<tr>
<td>Unit Width of Floodwall for Plane Strain Model: ( \text{unit width, b} ) (ft)</td>
<td>5.583</td>
</tr>
<tr>
<td>Coefficient of At-Rest Pressure: ( K_o )</td>
<td>0.8</td>
</tr>
</tbody>
</table>
Table 5.3  Model structural and hydraulic variables$^a$.

<table>
<thead>
<tr>
<th>Variable</th>
<th>Description</th>
<th>Nominal Value</th>
<th>$\lambda$</th>
<th>COV</th>
<th>$\mu$</th>
<th>$\sigma$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\gamma_{\text{con}}$ (kcf)</td>
<td>Unit weight of concrete</td>
<td>0.15</td>
<td>1</td>
<td>0.15</td>
<td>0.002</td>
<td></td>
</tr>
<tr>
<td>$E_{\text{steel}}$ (ksi)</td>
<td>Young’s modulus, steel</td>
<td>29,000</td>
<td>0.993</td>
<td>0.034</td>
<td>28797</td>
<td>979.098</td>
</tr>
<tr>
<td>$f_c'$ (ksi)</td>
<td>Compressive strength, concrete</td>
<td>4</td>
<td>1.235</td>
<td>0.145</td>
<td>4.94</td>
<td>0.7163</td>
</tr>
<tr>
<td>$f_y, \text{rebar}$ (ksi)</td>
<td>Yield stress, reinforcing steel</td>
<td>60</td>
<td>1.145</td>
<td>0.05</td>
<td>68.7</td>
<td>3.435</td>
</tr>
<tr>
<td>$F_y, \text{HP}$ (ksi)</td>
<td>Yield stress, H-piles</td>
<td>50</td>
<td>1.06</td>
<td>0.06</td>
<td>53</td>
<td>3.18</td>
</tr>
<tr>
<td>$F_{y, \text{HPSHEAR}}$ (ksi)</td>
<td>Yield stress, H-pile shear</td>
<td>50</td>
<td>1.1</td>
<td>0.11</td>
<td>55</td>
<td>6.05</td>
</tr>
<tr>
<td>$F_y, \text{TC}$ (ksi)</td>
<td>Yield stress, tension connectors</td>
<td>50</td>
<td>1.06</td>
<td>0.06</td>
<td>53</td>
<td>3.18</td>
</tr>
<tr>
<td>$F_u$ (ksi)</td>
<td>Ultimate stress, steel</td>
<td>65</td>
<td>1.101</td>
<td>0.051</td>
<td>71.565</td>
<td>3.64982</td>
</tr>
<tr>
<td>$A_{s, \text{stem}}$ (in$^2$)</td>
<td>Area of main reinforcement, stem</td>
<td>0.44</td>
<td>1</td>
<td>0.015</td>
<td>0.44</td>
<td>0.0066</td>
</tr>
<tr>
<td>$A_{s, \text{base top}}$ (in$^2$)</td>
<td>Area of upper layer of base reinforcement</td>
<td>1.58</td>
<td>1</td>
<td>0.015</td>
<td>1.58</td>
<td>0.0237</td>
</tr>
<tr>
<td>$d_{\text{stem}}$ (in)</td>
<td>Effective depth, stem reinforcement</td>
<td>19.625</td>
<td>0.99</td>
<td>0.04</td>
<td>19.4288</td>
<td>0.77715</td>
</tr>
<tr>
<td>$d_{\text{base, top}}$ (in)</td>
<td>Effective depth, base top reinforcement</td>
<td>31.5</td>
<td>0.99</td>
<td>0.04</td>
<td>31.185</td>
<td>1.2474</td>
</tr>
<tr>
<td>$d_{\text{base, bot}}$ (in)</td>
<td>Effective depth, base bottom reinforcement</td>
<td>23.5</td>
<td>0.99</td>
<td>0.04</td>
<td>23.265</td>
<td>0.9306</td>
</tr>
<tr>
<td>$B = b_f, \text{HP}$ (in)</td>
<td>Pile width</td>
<td>14.695</td>
<td>1</td>
<td>0.05</td>
<td>14.695</td>
<td>0.73475</td>
</tr>
<tr>
<td>$d_{\text{HP}}$ (in)</td>
<td>Depth of H-pile</td>
<td>13.83</td>
<td>1</td>
<td>0.05</td>
<td>13.83</td>
<td>0.6915</td>
</tr>
<tr>
<td>$S_{x, \text{HP}}$ (in$^3$)</td>
<td>Section modulus, H-pile strong axis</td>
<td>131</td>
<td>1</td>
<td>0.05</td>
<td>131</td>
<td>6.55</td>
</tr>
<tr>
<td>$I_{x, \text{HP}}$ (in$^4$)</td>
<td>Moment of inertia, H-pile strong axis</td>
<td>904</td>
<td>1</td>
<td>0.05</td>
<td>904</td>
<td>45.2</td>
</tr>
<tr>
<td>$t_w, \text{HP}$ (in)</td>
<td>Thickness of H-pile web</td>
<td>0.615</td>
<td>1</td>
<td>0.05</td>
<td>0.615</td>
<td>0.03075</td>
</tr>
<tr>
<td>$A_g, \text{HP}$ (in$^2$)</td>
<td>Gross area, H-pile</td>
<td>26.1</td>
<td>1</td>
<td>0.05</td>
<td>26.1</td>
<td>1.305</td>
</tr>
<tr>
<td>$r_x, \text{HP}$ (in)</td>
<td>Radius of gyration, H-pile strong axis</td>
<td>5.88</td>
<td>1</td>
<td>0.05</td>
<td>5.88</td>
<td>0.294</td>
</tr>
<tr>
<td>$r_y, \text{HP}$ (in)</td>
<td>Radius of gyration, H-pile weak axis</td>
<td>3.53</td>
<td>1</td>
<td>0.05</td>
<td>3.53</td>
<td>0.1765</td>
</tr>
<tr>
<td>$t_{T C}$ (in)</td>
<td>Width, tension connector</td>
<td>2</td>
<td>1</td>
<td>0.05</td>
<td>2</td>
<td>0.1</td>
</tr>
<tr>
<td>$t_{T C}$ (in)</td>
<td>Thickness, tension connector</td>
<td>0.625</td>
<td>1</td>
<td>0.05</td>
<td>0.625</td>
<td>0.03125</td>
</tr>
<tr>
<td>$PF \text{ RC Flex}$</td>
<td>Professional factor, concrete flexure</td>
<td>1</td>
<td>1.02</td>
<td>0.06</td>
<td>1.02</td>
<td>0.0612</td>
</tr>
<tr>
<td>$PF \text{ RC Shear}$</td>
<td>Professional factor, concrete shear</td>
<td>1</td>
<td>1.075</td>
<td>0.1</td>
<td>1.075</td>
<td>0.1075</td>
</tr>
<tr>
<td>$PF \text{ HP Col. Tens.}$</td>
<td>Professional factor, H-pile tension</td>
<td>1</td>
<td>1</td>
<td>0</td>
<td>1</td>
<td>0</td>
</tr>
<tr>
<td>$PF \text{ HP Col.}$</td>
<td>Professional factor, H-pile compression only</td>
<td>1</td>
<td>1.03</td>
<td>0.05</td>
<td>1.03</td>
<td>0.0515</td>
</tr>
<tr>
<td>$PF \text{ HP Flex.}$</td>
<td>Professional factor, H-pile flexure only</td>
<td>1</td>
<td>1.02</td>
<td>0.08</td>
<td>1.02</td>
<td>0.0816</td>
</tr>
<tr>
<td>$PF \text{ HP Combined}$</td>
<td>Professional factor, H-pile combined axial (compression or tension) and flexure</td>
<td>1</td>
<td>1.02</td>
<td>0.1</td>
<td>1.02</td>
<td>0.102</td>
</tr>
</tbody>
</table>

$^a$Variables defined in Chapter 2. Subscripts HP (H-Piles), TC (Tension Connectors) added to differentiate elements.

Global stability analysis and flowthrough element reliability were computed as described in Chapter 4. Using that method, UBLs were computed for multiple individual simulations. Pile group analysis was performed using GROUP and was included with the results of the stability analysis for post-processing. Individual element reliabilities were computed for stem flexure, stem beam shear, base slab flexure (positive and negative moment), base slab beam shear, pile-
to-base slab connection (punching shear of vertical pile reaction through projected area of pile top through the base slab; pullout of pile due to tension, and axial tension of the tension connectors), pile geotechnical capacity, pile beam-column capacity (combined axial and flexure), pile structural beam shear capacity, flowthrough due to global instability, and pile passive flowthrough resistance. As an added measure to compare results, stem reliability was also computed using First Order Second Moment (FOSM).

5.4.2 System Reliability

After computation of element reliabilities, system reliability was assessed using the event tree method of Figure 5.4. Because it does not typically control the design of pile-founded hurricane risk reduction structures, this model neglected the seepage and piping limit states and focused solely on external and internal structural stability. For computation of reliability for seepage and piping limit states, the method used by IPET can be utilized (IPET 2009). All limit states were considered in series and the model was developed using the assumption of no correlation among variables or elements. Two separate runs were performed to bracket the series versus parallel system assumption for the two protected side pile rows. In the former case, the piles were all considered in series with one another and in the latter, the two protected side pile rows were considered to be in parallel. Probabilities of exceedance used in the computation were based upon a structure life equal to 100 years.

5.5 Results and Discussion

A comparison of element reliabilities for shear and flexure limit states using the 2k+1 method (Rosenbleuth 1975) and FOSM is presented in Table 5.4. As shown by the table, compared to FOSM, the 2k+1 produces lower values of element reliability, producing reliability indices on the order of 0.400 less than the corresponding FOSM indices for shear. For flexure, however, the individual element reliabilities computed using 2k+1 were closer to those computed
by FOSM, showing a difference of approximately 0.170. These results indicate that the point estimate approximation provides as reasonable an approximation of element reliability as FOSM.

Table 5.4  Stem reliability indices ($\beta$) using FOSM and point estimate methods.

<table>
<thead>
<tr>
<th>$P_e$</th>
<th>Shear FOSM</th>
<th>2k+1 FOSM</th>
<th>Flexure FOSM</th>
<th>2k+1 FOSM</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.20%</td>
<td>7.467</td>
<td>7.083</td>
<td>7.882</td>
<td>7.718</td>
</tr>
<tr>
<td>0.50%</td>
<td>7.724</td>
<td>7.322</td>
<td>9.937</td>
<td>9.775</td>
</tr>
<tr>
<td>1.00%</td>
<td>7.879</td>
<td>7.467</td>
<td>10.866</td>
<td>10.700</td>
</tr>
<tr>
<td>2.00%</td>
<td>8.009</td>
<td>7.589</td>
<td>11.292</td>
<td>11.121</td>
</tr>
</tbody>
</table>

A complete tabulation of element reliabilities is presented in Table 5.5. As shown by Table 5.5, the reliability of the structural elements designed in accordance with USACE criteria (USACE 2008) is in many cases much higher than the soil-structure interaction elements. To illustrate this, the geotechnical axial limit state for pile 3 produced $\beta = 2.674$. For the structural combined axial and bending limit state, a value of $\beta = 18.690$ was computed. Overall, shear governed over flexure for combined action for this case study, and geotechnical elements governed over structural elements. Based on these results for this case study, a sensitivity analysis that targets the “safe” limit states could be run to justify refining the model.

A comparison of system reliability for different annual probability of exceedance hurricane storm surge elevations is presented in Figure 5.6. The figure shows plots of results when (1) the two protected side (compression) pile rows are considered to be in series and (2) when those same to pile rows are considered to act in parallel. As shown by the figure, the 0.2% exceedance hurricane surge produced the lowest reliability for both cases, producing reliability somewhere between 2.583 and 3.364. The 0.5% exceedance also produced slightly lower values than the 1% and 2% exceedance cases. It should be noted that the largest number of cases having unbalanced loads was the 0.2% exceedance level, with the 6 total iterations exhibiting global instability, followed by the 0.5% with 2 iterations exhibiting global instability. With this
Table 5.5   Element reliability indices ($\beta$) for a 100-year design life.a.

<table>
<thead>
<tr>
<th>ELEMENT</th>
<th>SYMBOL</th>
<th>2%</th>
<th>1%</th>
<th>0.5%</th>
<th>0.2%</th>
</tr>
</thead>
<tbody>
<tr>
<td>Global Instability</td>
<td>I</td>
<td>3.424</td>
<td>3.146</td>
<td>2.707</td>
<td>2.236</td>
</tr>
<tr>
<td>Flowthrough 1</td>
<td>F1</td>
<td>3.316</td>
<td>3.241</td>
<td>3.064</td>
<td>2.779</td>
</tr>
<tr>
<td>Flowthrough 2</td>
<td>F2</td>
<td>2.593</td>
<td>2.567</td>
<td>2.506</td>
<td>2.434</td>
</tr>
<tr>
<td>Pile - Geotechnical Axial - Pile 1</td>
<td>PGA1</td>
<td>6.799</td>
<td>7.229</td>
<td>5.946</td>
<td>3.384</td>
</tr>
<tr>
<td>Pile - Geotechnical Axial - Pile 2</td>
<td>PGA2</td>
<td>7.602</td>
<td>8.770</td>
<td>5.570</td>
<td>3.167</td>
</tr>
<tr>
<td>Pile - Geotechnical Axial - Pile 3</td>
<td>PGA3</td>
<td>6.006</td>
<td>6.234</td>
<td>4.971</td>
<td>2.674</td>
</tr>
<tr>
<td>Pile - Structural - CBF - Pile 1</td>
<td>PCBF1</td>
<td>68.143</td>
<td>53.013</td>
<td>19.987</td>
<td>15.741</td>
</tr>
<tr>
<td>Pile - Structural - CBF - Pile 2</td>
<td>PCBF2</td>
<td>163.565</td>
<td>159.479</td>
<td>228.073</td>
<td>52.838</td>
</tr>
<tr>
<td>Pile - Structural - CBF - Pile 3</td>
<td>PCBF3</td>
<td>87.948</td>
<td>102.086</td>
<td>73.167</td>
<td>18.690</td>
</tr>
<tr>
<td>Pile - Structural - Shear - Pile 1</td>
<td>PSV1</td>
<td>7.591</td>
<td>7.597</td>
<td>7.594</td>
<td>7.596</td>
</tr>
<tr>
<td>Pile - Structural - Shear - Pile 2</td>
<td>PSV2</td>
<td>7.614</td>
<td>7.584</td>
<td>7.585</td>
<td>7.595</td>
</tr>
<tr>
<td>Pile - Structural - Shear - Pile 3</td>
<td>PSV3</td>
<td>7.565</td>
<td>7.569</td>
<td>7.570</td>
<td>7.573</td>
</tr>
<tr>
<td>Pile - Base - Punch Shear - Pile 1</td>
<td>PBPV1</td>
<td>8.384</td>
<td>8.404</td>
<td>8.424</td>
<td>8.448</td>
</tr>
<tr>
<td>Pile - Base - Punch Shear - Pile 2</td>
<td>PBPV2</td>
<td>8.406</td>
<td>8.374</td>
<td>8.349</td>
<td>8.292</td>
</tr>
<tr>
<td>Pile - Base - Punch Shear - Pile 3</td>
<td>PBPV3</td>
<td>8.168</td>
<td>8.152</td>
<td>8.133</td>
<td>8.092</td>
</tr>
<tr>
<td>Pile - Base - Pullout - Pile 1</td>
<td>PBPO1</td>
<td>56.607</td>
<td>52.505</td>
<td>24.938</td>
<td>28.267</td>
</tr>
<tr>
<td>Pile - Base - Pullout - Pile 2</td>
<td>PBPO2</td>
<td>162.773</td>
<td>100.571</td>
<td>28.969</td>
<td>10.967</td>
</tr>
<tr>
<td>Pile - Base - Pullout - Pile 3</td>
<td>PBPO3</td>
<td>75.241</td>
<td>89.709</td>
<td>90.897</td>
<td>43.653</td>
</tr>
<tr>
<td>Pile - Base - Anchor Struc - Pile 1</td>
<td>PBAS1</td>
<td>10.768</td>
<td>10.768</td>
<td>10.768</td>
<td>10.768</td>
</tr>
<tr>
<td>Pile - Base - Anchor Struc - Pile 2</td>
<td>PBAS2</td>
<td>10.746</td>
<td>10.768</td>
<td>10.768</td>
<td>10.768</td>
</tr>
<tr>
<td>Pile - Base - Anchor Struc - Pile 3</td>
<td>PBAS3</td>
<td>10.768</td>
<td>10.768</td>
<td>10.768</td>
<td>10.768</td>
</tr>
<tr>
<td>Pile Group Lat Movement</td>
<td>PGLM</td>
<td>19.499</td>
<td>35.018</td>
<td>80.327</td>
<td>24.499</td>
</tr>
<tr>
<td>Pile Group Vert Movement</td>
<td>PGVM</td>
<td>48.962</td>
<td>92.831</td>
<td>129.271</td>
<td>49.265</td>
</tr>
<tr>
<td>Base Flex+</td>
<td>BF+</td>
<td>10.219</td>
<td>10.284</td>
<td>10.325</td>
<td>10.303</td>
</tr>
<tr>
<td>Base Flex-</td>
<td>BF-</td>
<td>11.249</td>
<td>11.249</td>
<td>11.249</td>
<td>11.212</td>
</tr>
<tr>
<td>Base Shear+</td>
<td>BV+</td>
<td>7.114</td>
<td>7.094</td>
<td>7.088</td>
<td>7.054</td>
</tr>
<tr>
<td>Base Shear-</td>
<td>BV-</td>
<td>7.655</td>
<td>7.655</td>
<td>7.655</td>
<td>7.611</td>
</tr>
<tr>
<td>Stem Flex</td>
<td>SF</td>
<td>11.121</td>
<td>10.700</td>
<td>9.775</td>
<td>7.718</td>
</tr>
<tr>
<td>Stem Shear</td>
<td>SV</td>
<td>7.589</td>
<td>7.467</td>
<td>7.322</td>
<td>7.083</td>
</tr>
</tbody>
</table>

$\alpha$ = Annual Probability of Exceedance

fact, the trend in the figure indicates the introduction of increased variability in the margin of safety that translates into lower reliability. Also worthy of note is that for 3 out of 4 of the cases analyzed, the system reliability index is between 4.266 and 4.686.

To put these results in perspective, Table 5.6 presents target reliabilities for different structures of different design lives. As shown by the table, the design condition for pile-founded hurricane risk reduction structures has a system reliability at or above the target reliability indices presented in Table 5.6, even with the higher probability of experiencing the hurricane storm surge in a 100-year period than the 50-year life presented in the table. Although the 0.2% annual probability of exceedance demonstrates a significant drop in reliability for the
compression piles in series condition, this exceedance is a resiliency check whose performance requirement is structural survival. Because hurricane risk reduction system elevations (in this case the Greater New Orleans system) are selected on the basis of overtopping rates for the design exceedance and checked against a higher exceedance surge elevation (in this case 0.2%), higher exceedance events would result in the system likely being overtopped prior to arrival at a dangerously low reliability value for structural integrity (USACE 2007).

![System reliability index for different annual exceedances of hurricane storm surge.](image)

Figure 5.6  System reliability index for different annual exceedances of hurricane storm surge.

For some elements, the disparity between typical reliability values presented in Table 5.6 and element reliabilities presented in Table 5.5 are quite large, especially for structural elements. The floodwall structural elements were even larger than the target reliability indices presented in Table C.1.3.1a of ASCE 7-10 (ASCE 2010), which presents target reliability indices varying from 3.5 to 4.5 for Risk Category IV structures (which would be closest in performance requirements to hurricane risk reduction structures) depending upon the type of failure anticipated. The exception to the substantial disparity lay with the geotechnical elements,
however. The element reliability for global instability and flowthrough are much closer to the values in Table 5.6 than expected. For geotechnical elements, such as piles, the target reliability index can be lower than that for structural elements. Paikowsky (2004) recommended the use of a target reliability index of 2.33 for redundant piles (defined as 5 or more piles per pile cap) and target reliability index of 3.0 for non-redundant piles. Since this floodwall in actuality contains 27 piles, the value of 2.33 would be appropriate for comparison.


<table>
<thead>
<tr>
<th>Structural Type/Member, Limit State</th>
<th>Target Reliability Index ($\beta_{target}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Metal structures for buildings (dead, live, and snow loads)</td>
<td>3.0</td>
</tr>
<tr>
<td>Metal structures for buildings (dead, live, and wind loads)</td>
<td>2.5</td>
</tr>
<tr>
<td>Metal structures for buildings (dead, live, and earthquake loads)</td>
<td>1.75</td>
</tr>
<tr>
<td>Metal connections for buildings (dead, live, and snow loads)</td>
<td>4 to 4.5</td>
</tr>
<tr>
<td>Reinforced concrete for buildings (dead, live, and snow loads)</td>
<td></td>
</tr>
<tr>
<td>- Ductile failure</td>
<td>3.0</td>
</tr>
<tr>
<td>- Brittle failure</td>
<td>3.5</td>
</tr>
<tr>
<td>Structural Steel</td>
<td></td>
</tr>
<tr>
<td>- Tension member, yield</td>
<td>3.0</td>
</tr>
<tr>
<td>- Beam in flexure</td>
<td>2.5</td>
</tr>
<tr>
<td>- Beam in shear</td>
<td>3.0</td>
</tr>
<tr>
<td>- Column, intermediate slenderness</td>
<td>3.5</td>
</tr>
<tr>
<td>Reinforced Concrete</td>
<td></td>
</tr>
<tr>
<td>- Beam in flexure</td>
<td>3.0</td>
</tr>
<tr>
<td>- Beam in shear</td>
<td>3.0</td>
</tr>
<tr>
<td>- Tied Column, compressive failure</td>
<td>3.5</td>
</tr>
<tr>
<td>Girder Bridge</td>
<td></td>
</tr>
<tr>
<td>- AASHTO LRFD</td>
<td>3.5</td>
</tr>
<tr>
<td>- Redundant (span &lt; 100 ft)</td>
<td>2.5 to 2.7</td>
</tr>
<tr>
<td>- Non-redundant (span &lt; 100 ft)</td>
<td>3.5</td>
</tr>
<tr>
<td>Offshore Structures</td>
<td>2.5</td>
</tr>
<tr>
<td>Bridges (Canada)</td>
<td>2.00 to</td>
</tr>
<tr>
<td>Bridge Foundations</td>
<td>3.75$^b$</td>
</tr>
<tr>
<td>- Redundant piles (5 or more piles per pile cap)</td>
<td>2.33</td>
</tr>
<tr>
<td>- Non-redundant piles (4 or fewer piles per pile cap)</td>
<td>3.00</td>
</tr>
</tbody>
</table>

$^a$Design life for all features except bridges is 50 years. Design life for bridges is 75 years.

$^b$Function of many factors (see Allen 1992)

These element reliability indices indicate the separate safety factors incorporated into the separate disciplines’ criteria and methods may actually be producing a more conservative than
necessary design, when considering a floodwall designed using steel H-piles subjected to hydrostatic hurricane storm surge and global instability. Based upon the prescribed minimum floodwall stem thicknesses presented in USACE (2008) and the minimum base slab thickness dictated by pile embedment and tension connector lengths, the hurricane storm surge hydrostatic load is likely to produce similar results over a range of what could be considered the “typical” T-wall. However, element reliability for different pile types may actually be lower, especially for pile types that may not have the reserve flexural capacity that is often observed in steel H-piles. Other loading conditions including waves, debris impact, small vessel impact and settlement-induced loads on piles should be analyzed to develop a better understanding of the reliability of hurricane risk reduction structures under the broader range of possible loads and combinations of loads before developing recommendations for changes in criteria or policy. Other pile-founded hurricane risk reduction structures should also be analyzed, including both different types of structures and different T-wall configurations (different heights, different pile types, different embankment configurations, and different foundation conditions). To do this, the methodology presented in this dissertation can be adapted, with some computational effort, to accommodate a range of different problems.

5.6 Conclusions

This chapter developed a complete framework that can be used to quantify the reliability of pile-founded hurricane risk reduction structures subjected to global instability through the use of designer-friendly methods and tools. A pile-founded floodwall from a real-world project site was analyzed using the newly-developed framework to quantify reliability for a floodwall designed using the criteria in USACE (2008). The results were compared to other industry target reliability indices and point to the need for further investigation into the reliability of different types of structures designed using USACE (2008).
The conclusion drawn for the modeling framework is that point estimate methods used in conjunction with event tree methods can be used to compute the system reliability of pile-founded hurricane risk reduction structures. These methods make use of commercially available design software to develop input for the point-estimate and event tree reliability simulation model used to quantify element and system reliability.

Conclusions drawn from the case study that investigated a pile-founded T-wall subjected to global instability and hydrostatic hurricane storm surge loadings (excluding wave load, impact load, or settlement induced loads) include:

1. For a floodwall stem exposed to hydrostatic loads from hurricane storm surge, reliability indices computed using point estimate methods were slightly lower than those computed using FOSM. This seems to indicate that point estimate results are somewhat more conservative than FOSM.

2. Using this application of point estimate and event tree methods and the hydrostatic loads associated with hurricane storm surge, the element and system reliabilities computed compared well with recommended values presented in the literature. For structural elements the values computed were well above the published targets. Geotechnical element reliabilities were lower than structural elements, but were still above target values presented in literature.

3. System reliability for the pile-founded floodwall computed for the hydrostatic storm surge load case was above targets used by other codes.

4. Based on the results presented herein, further investigation into possible refinement of the HSDRRS Design Guidelines is warranted. Further research into setting target reliability is recommended and validation of the simplified methodology using advanced modeling techniques is warranted.
CHAPTER 6: CONCLUSIONS AND RECOMMENDATIONS

6.1 Introduction

6.1.1 Summary

The problem addressed by this dissertation is the quantification of reliability provided by designs using the criteria in the 2008 edition of the Hurricane and Storm Damage Risk Reduction System (HSDRRS) Design Guidelines (USACE 2008). Given that the criteria were established using expert judgment and deterministic criteria that historically resulted in successful designs, the level of reliability for structures designed utilizing the HSDRRS Design Guidelines is unknown. Therefore, primary goals of this dissertation were to develop a simplified, interdisciplinary designer-friendly methodology for the quantification of the reliability of pile-founded hurricane risk reduction structures designed using the current HSDRRS criteria and to apply that methodology to quantify the reliability of a representative structure in southeast Louisiana for comparison with target reliabilities used by other codes. These goals were achieved through accomplishment of the dissertation objectives, as presented in Chapters 2 through 5 and summarized here.

6.1.2 Objective 1: To Perform an Examination of the 2008 HSDRRS Design Guidelines.

Chapter 2 summarizes a review of the sources that formed the basis of the 2008 HSDRRS Design Guidelines and supporting literature upon which the methods and deterministic requirements have been modeled. Notable observations include the use of JPM-OS for development of hurricane storm surge elevations (USACE 2007, IPET 2009, Resio et al. 2009), the use of Spencer’s Method (Spencer 1967) for assessment of global stability, the use of the p-y approach (Matlock 1970, Reese et al. 1974, 1975, Reese and Welch 1975) for designing foundations to resist global instability or the use of the subgrade modulus approach (Terzaghi 1955, Broms 1964) when there is no global instability, and the use of the API method (API
1986) for determining adhesion for side friction resistance on piles provided by cohesive soils.

Structural resistance is computed using standard methods. Chapter 2 also described the elements of the T-wall structure (stem, base slab, foundation piles, cut-off sheet piles, and embankment), and described the load path for transmitting hurricane storm surge loadings through the structure to the underlying foundation.

6.1.3 Objective 2: To Develop the Dataset for Critical Variables from the Existing Body of Literature.

Chapter 2 identifies parameters required for design and reliability analysis. Chapter 2 also examines the literature to determine the state-of-the-art in existing datasets used by other researchers and used in the calibration of other design codes. The culmination of Chapter 2 is the presentation of recommended values to be used in the development of a reliability model.

6.1.4 Objective 3: To Assess the Capability of Using a Design-Level Dataset for Determining Spatial Correlation of Geotechnical Parameters.

Chapter 3 provides a critical link between values found in the literature and the quantification of geotechnical uncertainty for a given project site. Sources of geotechnical uncertainty are identified: (1) the inherent variability of the soil, (2) measurement error, (3) statistical uncertainty, and (4) transformation model uncertainty (Christian et al. 1994, Phoon and Kulhawy 1999a, IPET 2009). The methodology for determining scale of fluctuation for undrained shear strength presented by Jaksa, Kaggwa et al. (2000) is extended in Chapter 3 to UU and UC lab test data obtained through the testing of undisturbed borings, in lieu of CPTs. Variance reduction techniques employed by VanMarcke (1977) and Phoon and Kulhawy (1999b) are also applied. Finally, adjusted values of COV are presented based upon published values for measurement error (unit weight and undrained shear strength) and the computed variance reductions (undrained shear strength only).
6.1.5 Objective 4: To Develop a Simulation Modeling Methodology for the Reliability Analysis, to Develop a Method for Setting a Threshold for Global Instability in Reliability Analysis, and to Validate the Need to Use Spatial Averaging.

The state of the art for reliability analysis methods is examined in Chapters 4 and 5. Methods examined include those presented by Christian (2004) for geotechnical and, by extension, soil-structure interaction reliability problems including direct reliability analysis, First Order Second Moment (FOSM), First Order Reliability Methods (FORM), point estimate methods, and Monte Carlo Simulation. Because the simulation modeling methodology is intended to be “designer-friendly,” point estimate methods are selected for use in computing the reliability of individual floodwall elements and limit states, with an order-of-magnitude check of the floodwall stem reliability performed using FOSM. The Taylor series (Wolff 1994, Duncan 2000) method and the 2k+1 (where k = number of variables) point estimate method (Rosenbleuth 1975) are selected, with the former method applied to the global stability element of the analysis and the 2k+1 applied to all other elements. In Chapter 4, analysis of the Flowthrough limit states demonstrate the importance of accounting for spatial averaging effects when computing reliability of soil-structure interaction problems by both including and neglecting the autocorrelation effects in undrained shear strengths.

6.1.6 Objective 5: To Construct the Remainder of the Simulation Model for the Test Case and to Quantify Its Reliability.

In Chapter 5, the reliability model presented in Chapter 4 is expanded to the remaining elements and limit states of the entire test case structure. The event tree method (Christian 2004, USACE 2006) is used to compute “system” reliability for the entire structure. For the overall model, idealized series and parallel system equations (Nowak and Collins 2000) are used and all elements (and element properties) are assumed to be uncorrelated. Reliability of the floodwall “system” is computed for four annual probabilities of exceedance hurricane storm surges (0.2%,
0.5%, 1%, and 2%) and the results are compared to target reliabilities used by other organizations.

6.2 Conclusions and Discussion

6.2.1 Conclusions

This dissertation presented a framework for quantifying reliability of pile-founded hurricane risk reduction structures and used that framework to quantify the reliability of a representative structure for comparison with other industry target reliabilities. Each chapter in the body of this dissertation resulted in contributions to the body of knowledge.

Chapter 2 performed the first step of identifying design parameters associated with the methods and tools used in conjunction with USACE (2008). In Chapter 2, research that supported the updating and calibration of reinforced concrete, structural steel, and bridge design codes was reviewed. The uncertainty in the applicable design parameters was identified, parameters were identified for treatment as random variables and constants, and recommendations were made for values of COV to be used in conjunction with reliability analysis. The overarching conclusion presented in this chapter was:

- For production-oriented modeling, using existing datasets and limiting reliability models to specific annual probability of exceedance hurricane storm surge events is recommended (such as the one identified in USACE 2007). This approach helps reduce the computational burden associated with additional surge and wave modeling or additional data reduction. Consideration should be given to model uncertainty and recommendations are provided for statistical parameters that might not otherwise be included in existing datasets.

Chapter 3 identified sources of geotechnical uncertainty and quantified the variability of soil unit weight, \( \gamma_{soil} \), and undrained shear strength of clay, \( s_u \). Chapter 3 contributed insight into
the suitability for using test data from unconsolidated-undrained triaxial compression (UU) tests and unconfined compression tests (UC) using samples obtained from undisturbed borings obtained as part of design-quality field investigations in reliability analysis. It also presented data on scale of fluctuation for Holocene and Pleistocene clays. Conclusions drawn from this effort are specific to geological settings with highly stratified soils originating from a combination of fluvial, deltaic, and coastal deposits, similar to Southeast Louisiana (LGS 2008) and include:

- The inherent variation in $\gamma_{soils}$, even in the highly-stratified soils of Southeast Louisiana, is low relative to the variation of $s_u$, which is consistent with the findings in the literature. Even with limited datasets, the sample COV will likely be close to $COV_{sw} = 10\%$ found in the literature.

- The inherent variation in $s_u$ can result in values of $COV_{sw}$ approaching the upper value of $62\%$ found in the literature.

- Before computing scale of fluctuation, it is important to review the data carefully and use caution when removing data outliers. Depending upon the size of the dataset, the computed value can be sensitive, especially if the data point is near the upper or lower spatial bounds of the dataset where data trends can be more drastically impacted.

- Even with the limited dataset and assumptions required to fill data gaps, values computed for vertical scale of fluctuation computed for individual soils reaches were between 0.85 and 4.88 m, which is consistent with the values in the literature between 1 and 6 m. This suggests that limited datasets can provide reasonable estimates of $\delta_v$ for use in computing variance reductions. However, the fact that the lower bound value occurs in a soil reach containing peat, silt, and intradelta sands interspersed in clay layers suggests that soil stratification should be reviewed thoroughly before proceeding with analysis.
• The limited dataset produced horizontal scales of fluctuation between 91 and 290 m, which were within the range of 40 to 300 m presented in the literature. However, given the large number of assumptions required to populate the data gaps, the results may contain unacceptable bias. The level of subsurface definition required and the objective for using the scale of fluctuation should guide the level of accuracy required when planning field investigations. As shown by the drastic reductions in variance that can be computed when scale of fluctuation is low and averaging distances are high, the impact on reliability analysis results could be significant if inappropriate results are applied.

• The presence of an existing manmade embankment can potentially skew computed scales of fluctuation, as shown by the higher values of vertical scale of fluctuation compared to values when not considering all data above El. -1.2, which is consistent with the findings of Koutnik (2012).

• If soil stratification is such that stratum thicknesses are near or less than the lower bound values of \( \delta_v \) found in the literature, computation of \( \delta_v \) is not recommended as the averaging distance would result in no variance reduction.

• For many typical hurricane risk reduction floodwall reaches, the length of floodwall may not approach published values of \( \delta_h \), especially the upper bound of 300 m. The use of published values of \( \delta_h \) is recommended for use in computing variance reductions unless sufficient undisturbed borings and CPTs are available to provide a complete dataset at a sampling interval that adequately represents all soil strata and supports computation of scale of fluctuation without need to fill data gaps through interpolation, simulation, or grouping of strata.

Chapter 4 presented the first elements of the reliability analysis framework, developed a method to incorporate the USACE (2008) model for floodwall resistance to global instability
when subjected to hydrostatic hurricane storm surge loading into reliability simulations, and confirmed the importance of utilizing variance reductions resulting from spatial correlation of undrained shear strengths. Conclusions drawn from this chapter include:

- Because factors of safety computed using mean values of geotechnical parameters are likely to be much higher than unity and allowable factors of safety used for design, the back-calculation of a target factor of safety is required to determine the unbalanced force necessary to maintain system stability. This is possible because the standard deviation of the factor of safety for a given simulation remains stable for that simulation when using the Taylor series method. A target reliability index of 2.0 is recommended for use in developing the target factor of safety.

- Not considering the effects of spatial correlation of undrained shear strengths results in higher probabilities of global instability for a given hurricane surge exceedance, with a pronounced difference when considering lower annual exceedance probability surge levels. When carried through the full calculation, it also results in a pronounced difference in expected performance levels as measured by the reliability index for flowthrough. Not considering the spatial correlation resulted in reliability indices in the 2.5 to 3.0 range while accounting for spatial correlation resulted in reliability indices greater than 4.0.

Chapter 5 developed the full framework for quantifying the reliability of pile-founded hurricane risk reduction structures subjected to global instability. The framework utilized designer-friendly methods and software to produce inputs to point estimate reliability simulations. The resulting element reliabilities can then be used to develop probabilities of unsatisfactory performance for inclusion into an event tree framework that uses idealized parallel and series system assumptions to compute a system reliability. This framework was then used to
 quantify the reliability of a representative structure subjected to hurricane storm surge hydrostatic loading (no wave loading, no impact loading, and no settlement-induced loading) for comparison with published target reliability indices. Conclusions drawn from this effort for the analyzed loading conditions include:

- Element and system reliabilities compared well with recommended values presented in the literature. Element reliability of the floodwall stem was computed using both point estimate methods and FOSM. The results were comparable with the point estimate methods producing lower values of element reliability.

- For a floodwall stem exposed to hydrostatic loads from hurricane storm surge, reliability indices computed using point estimate methods were slightly lower than those computed using FOSM. This seems to indicate that point estimate results are somewhat more conservative than FOSM.

- Using this application of point estimate and event tree methods and the hydrostatic loads associated with hurricane storm surge, the element and system reliabilities computed compared well with recommended values presented in the literature. For structural elements the values computed were well above the published targets. Geotechnical element reliabilities were lower than structural elements, but were still above target values presented in literature.

- System reliability for the pile-founded floodwall computed for the hydrostatic storm surge load case was above targets used by other codes.

- Based on the results presented herein, further investigation into possible refinement of the HSDRRS Design Guidelines is warranted. Further research into setting target reliability is recommended and validation of the simplified methodology using advanced modeling techniques is warranted.
6.2.2 Discussion

6.2.2.1 Model Advantages and Limitations

This study was initiated with the intention of developing a “designer-friendly” methodology for quantifying the reliability of a pile-founded hurricane risk reduction structure. With that in mind, point estimate methods were selected as being the most adaptable to the tools typically used by designers, including classical techniques and the use of software packages such as SLOPE/W (Geo-Slope 2008) and GROUP (ENSOFT 2006). This approach has the advantage of not requiring a detailed knowledge of probability distributions and limits the number of iterations in the simulation to a manageable number while allowing the same variations of each random variable to be run through the multiple numerical models. Monte Carlo Simulation (MCS) was considered for use for the global stability analysis, given the capabilities of SLOPE/W. However, the need for iterating to obtain the critical failure plane and the unbalanced load necessitated the more “controlled” methodology afforded by the point estimate methods. Latin hypercube sampling was also considered, but this would have rendered the method computationally inefficient and required an inordinate amount of time for pre-processing and post-processing results.

More rigorous single-model analysis techniques that utilize unified finite element models are in existence, such as the FLAC model employed by Won et al. (2011). Incorporating MCS into that sort of model could result in more refined results, but more research into the capabilities and limitations of FLAC would need to be conducted before embarking upon such an effort. Given that FLAC is currently used primarily to solve specialized, complex problems and is not currently considered a design production tool, its application might best be suited for validating and calibrating the methodology developed in this study.
As shown by the comparison of the FOSM results to the $2k+1$ results for the floodwall stem element reliability in Chapter 5, the element and system reliability computed should be considered an approximation of the actual reliability. Based on that comparison, it would appear that this method results in a conservative approximation, given that for shear and flexure the results for the $2k+1$ method were lower than those presented for FOSM.

Idealized models for series and parallel elements in the floodwall “system” were used in this method. Actual behavior is expected to be somewhere between those two idealizations. This was addressed by developing an upper and a lower bound for computed reliability. This technique permits the use of the simplifying assumptions while still acknowledging the potential introduction of error into the analysis. As shown by the analysis results, however, the difference between one assumption over the other only becomes pronounced as unbalanced loads are present in a greater number of iterations, such as the 0.2% annual probability of exceedance hurricane storm surge produced. For the case study presented, the design hurricane storm surge (1% exceedance) and the higher frequency hurricane storm surge (2% exceedance) did not exhibit a difference in results.

As noted by USACE (USACE 2007) for the hurricane storm surge models that draw upon inputs from a variety of other numerical models for implementation of the JPM-OS method (WAMBDI-Group 1988, Thompson and Cardone 1996, Smith et al. 2001, Luettich and Westerink 2006, USACE 2007), uncertainty is introduced through the modeling process. For structural reliability, this uncertainty is captured through the professional factor (P) (Ellingwood et al. 1980). The tabulations in Chapter 2 capture P values found in the literature, but unfortunately even limit states obtained from classical methods were not readily available. In these cases and in the case of SLOPE/W and GROUP results, P was assumed equal to 1.0. Given the results of the FOSM analysis performed for the floodwall stem, the value of P can
have significant impacts to the results of the reliability analysis of a given element. Further research should be conducted to better quantify the professional factor for those cases.

Lastly, as presented in Chapter 4, the presence or absence of an unbalanced load is contingent upon the selection of the target reliability index chosen to represent imminent failure. For this method a value of 2.0 has been selected. Further research is warranted into whether a different value may better reflect the possibility that instability may actually induce an unbalanced load.

6.2.2.2 Implications of Case Study Results

The case study analyzed a floodwall subjected to hurricane storm surge loads and dead loads only. Wave loads, impact loads, and wind load were not considered. Additionally, the results presented reflect the reliability of a structure built as designed, with no construction-related deviations and no degradation due to environmental factors through the life of the structure explicitly modeled.

As shown in Chapter 5, element reliabilities were typically well above 4.5 for all annual probability of exceedance \( (P_a) \) hurricane storm surges analyzed. Exceptions included the reliability index associated with the probability of global instability (which ranged from 3.424 for the 2\% \( P_a \) to 2.236 for the 0.2\% \( P_a \)), Flowthrough Mechanism 1 (which ranged from 3.316 for the 2\% \( P_a \) to 2.779 for the 0.2\% \( P_a \)), Flowthrough Mechanism 2 (which ranged from 2.593 for the 2\% \( P_a \) to 2.434 for the 0.2\% \( P_a \)), and the pile geotechnical axial resistance for the 0.2\% \( P_a \) (which ranged from 3.384 for the flood side pile to 2.674 for the protected side pile).

6.3 Recommendations for Further Study

Based upon the results of the case study and the limitations of the methodology, further study is warranted in several areas:
1. Research into what constitutes an appropriate target reliability for a given system, including performance-based or consequence-based targets.

2. Calibration/validation of the assumptions for the system model through finite element modeling and, if possible, Monte Carlo Simulation.

3. Development of Professional Factors for limit states unavailable in literature and for capturing the uncertainty in computational methods used by USACE (2008), including design method and numerical modeling techniques.

4. Development of updated statistical data for reinforced concrete based upon data developed through construction of the HSDRRS.

5. Development of a pushover analysis model to capture degree of redundancy and load sharing for better identification of parallel or series action for individual elements.

6. Development of a larger database of scales of fluctuation for soil types in areas where hurricane risk reduction systems are to be evaluated. This includes the use of CPTs calibrated by lab tests on undisturbed samples.

7. Refinement of the methodology to provide the capability to assess the implications of construction errors or structural degradation with time.

8. Study of additional test cases, to include different soil types, different pile types, different pile configurations, and different structure types and comparison with published target reliabilities or a new target reliability established specifically for hurricane risk reduction structures.

9. Perform comprehensive study to assess whether the criteria presented in the HSDRRS Design Guidelines are providing the intended level of reliability or whether revisions are required to make the criteria more conservative or less conservative.
10. Investigate the reliability of an entire protection system, which may consist of different sub-systems and compare the results to established targets.
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VITA

Christopher L. Dunn, P.E., was born in Greensburg, Louisiana, to Leslie L. Dunn and Lois S. Dunn. Christopher has two older sisters, Sherry and Brenda. Christopher graduated as valedictorian from Oak Forest Academy in Amite, Louisiana, and subsequently attended Louisiana State University, where he met his wife, Kelly, a native of Metairie, Louisiana. Christopher graduated with a bachelor’s degree in civil engineering in 1998 and began working as a structural engineer for URS Corporation in Metairie, Louisiana. In 2000, Christopher left URS for a position in Structures Branch in the New Orleans District of the U.S. Army Corps of Engineers, where his efforts have included the leadership of design efforts for structural features of the West Bank and Vicinity component of the Greater New Orleans Hurricane and Storm Damage Risk Reduction System (HSDRRS). Christopher began his graduate education in 1999 and received his master’s degree in civil engineering in 2001. In 2002, Christopher became a father and welcomed his first daughter Maggie, followed by two other daughters, Abby in 2005, and Molly in 2010. Christopher is a registered professional engineer in Louisiana and is currently the Deputy Chief of Structures Branch at the New Orleans District of the U.S. Army Corps of Engineers. Christopher is honored to be a member of the team rebuilding the risk reduction systems in southeast Louisiana. He is proud of the efforts of his colleagues, superiors, and subordinates whose families have given up much to support his colleagues in their execution of the monumental task of designing and constructing the HSDRRS in the years following Hurricane Katrina.