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Design flood elevations beyond code requirements and current best practices

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DESIGN FLOOD ELEVATIONS BEYOND CODE REQUIREMENTS AND CURRENT BEST PRACTICES

A Thesis

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To my hardworking & supporting professors

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To my fellow co-workers

&

To my family and friends

&

To those affected by storm surge and hurricanes

&

To all those aided by the results of this research

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ABSTRACT

In the United States, nearly 9 million people, 3.0% of the population, live in areas subject to the 1% annual chance (100-yr) coastal flood hazard. New construction and substantial improvements in coastal high hazard areas require structures to be elevated above the design flood elevation (DFE), without the use of fill (Bellomo et al. 1999). Building code requirements for flood elevation are linked to the National Flood Insurance Program (NFIP) insurance policies, and represent the minimum requirement for building elevation. Current elevation procedures are limited to the 100-year base flood elevation with minimal guidance beyond the 100-year elevation in many locations, which may be of interest to those designing critical facilities and buildings with a longer design life (e.g. institutional buildings). Additional code-plus resources exist to provide best available practices for practitioners; however, gaps still exist that may lead to lower design elevations than warranted for a particular risk level.

In an effort to provide guidance for practitioners, this thesis presents a methodology to address existing gaps in combination in the context of current best practices. A short case study to demonstrate the proposed methodology in comparison to code and best practices is provided. To provide guidance for longer return period flood events, this thesis uses stillwater elevations (SWEL) from flood insurance studies (FIS) to extrapolate flood elevations associated with longer return periods. FIS data are fit using the Huff-Angel and SRCC regression models, resulting in an equation to be used for extrapolating new flood elevations. The results of are evaluated using R^2 values, differences in projected elevations and known elevations for the same return period, and normalized data for the 100-year SWELs. The result of this work is not intended to become integrated into current code or policy regulations in the United States, but rather to provide generalized guidance to aid practitioners in decision making by consolidating current code, best practices, and characteristics of the changing coastal environment.

CHAPTER 1: INTRODUCTION

Financial losses attributed to floods in the U.S. total $2.4 billion annually, and more than 75% of all federal disaster declarations are related to flood events (Taggart and van de Lindt 2009; Li and van de Lindt 2012). Improving the practices and methods used for development within or near floodplains can help to reduce flood-induced losses. In 2000, USGS reported floods during the 20th century were the number-one hazard in terms of loss of life and financial damage (USGS 2000). Three of the thirty-two most significant flood events of the 20th century were a result of hurricane storm surge (Perry 2000). These three events were responsible for greater loss of life than all the rest of the events combined - the 1900 Galveston Hurricane which claimed 6,000+ lives, the worst on record; a 1938 unnamed hurricane responsible for 494 fatalities in the northeast U.S.; and 1969 Hurricane Camille causing 259 casualties (Perry 2000). This report does not include 2005 Hurricane Katrina, where storm surge 28 feet above sea level claimed 1,700+ lives (FEMA 2006). Coastal buildings are the first line of defense against storm surge and coastal flooding. As a means to mitigate the drastic economic losses and loss of life associated with storm surge and coastal flooding events, there is a need to expand our understanding of factors affecting flood elevations and how building design is affected by such factors.

The 1%-annual-chance flood is the basis for current standards, as established by the National Flood Insurance Program (NFIP), which was established through the National Flood Insurance Act of 1968 (Power and Shows 1979). The 1%-annual-chance represents the flood level that has a 1% chance of occurring or being exceeded every year. This elevation standard is used as a means to minimize flood losses by requiring buildings to be elevated at or above a location-specific base flood elevation (BFE) to prevent water from entering a building. The 1% annual chance flood is also referred to as the 100-year flood and the "base flood elevation"

(BFE). The 100-year flood should not be mistaken as a flood that only occurs once every 100 years, but rather has a 1% chance of being equaled or exceeded annually. The 1% annual chance flood was identified in December 1968 at a floodplain management guidelines seminar at the University of Chicago by approximately 50 floodplain management researchers convened by the U.S. Department of Housing and Urban Development (FEMA and Federal Insurance and Mitigation Administration 2002; Sheaffer 2004). The 100-year flood design standard was selected as a compromise between those advocating a 50-year or lesser standard and those advocating a 500-year flood standard (Krimm 2004). Although this decision defined the 100-year flood as the regulatory floodplain, it also largely affected engineering and construction practices in the U.S. by limiting development of risk-based design of the built environment exposed to flood events. Guidance and practices for determining building elevations beyond insurance requirements should have been established for those who wish to exceed the insurance-based NFIP minimum requirements. Unfortunately, the 1% recurrence interval BFE serves as an arbitrary number tied to NFIP insurance rates rather than conveying the real risk associated with flood hazards.

There are three main primary methods used in the U.S. to mitigate flood risk: elevating to the BFE in accordance with community-specific requirements, dry-flood proofing (i.e. preventing water from entering the structure), and wet-flood proofing (i.e. intentionally preparing the structure for flooding) (FEMA 2009; FEMA 2011). Wet and dry flood proofing are not viable options for mitigation in Coastal high hazard areas (i.e. V-zones) because of the hydrodynamic forces that can be imparted on buildings. Coastal high hazard areas are defined as areas subject to high velocity wave action on a community's flood hazard map and that are subject to a breaking wave height equal to or greater than 3-feet (ASCE 2005; ICC 2009). New construction and substantial improvements of structures in V-zones must be elevated on piles or

piers so the lowest horizontal structural member (LHSM) is above the BFE. Structural fill is also not allowed within V-zones (Bellomo et al. 1999).

The BFE serves only as a minimum requirement, representing a standard size event in order to manage flood risk for determining insurance rates, thus treating all communities equally (FEMA and Federal Insurance and Mitigation Administration 2002). The 100-year flood has a 26% (1 in 4) chance of occurring over the life of a 30-year mortgage (FEMA and Federal Insurance and Mitigation Administration 2002). Designing for a 700-year event would reduce this probability to a 4.2% chance of occurring over a 30-year mortgage. ASCE wind requirements call for structures to be designed to the 300, 700, and 1700 year wind according to their classification (ASCE 2010), yet flood design practice only requires structures to be built to the 100-year flood plus any freeboard requirements. Freeboard is defined as additional elevation between the lowest horizontal structural member and the BFE (FEMA 2011). Local jurisdictions can determine a community freeboard requirement; however, in lieu of local requirements, code-required freeboard is the same amount for the entire country. Therefore, a critical structure required to be built to withstand a 1700-year wind speed is only required to design for the 100-year flood plus two feet of freeboard (ASCE 2005; ASCE 2010). This represents a drastic gap between flood design and wind design practices. To design buildings and communities with known levels of risk, more risk-consistent practices should be employed.

As a result of the requirement to design to the 100-year BFE, for the majority of communities, there is no guidance for practitioners desiring to elevate above 100-year BFE to a specified recurrence interval flood. Newer Flood Insurance Studies (FIS) are beginning to provide 500-year stillwater elevations (SWELs), but for the majority of communities, which do not have 500-year SWEL values, FEMA recommends calculating the 500-year SWEL as 1.25 of the 100-year SWEL (FEMA and NAHB Research Center 2010). The rule of 1.25 may

significantly over estimate for some locations, thus causing extra costs to the owner, and significantly underestimate for other locations, causing owners' risk to be significantly higher than their perceived rate of risk. Flood elevation design guidance consistent with wind design requirements and with more reliable accuracy should be readily available for practitioners.

The Coastal Construction Manual (CCM) is currently considered the best flood design practice available by incorporating subsidence effects, sea level rise, and erosion effects with code regulations (FEMA 2011; FEMA 2011). Although the CCM is considered a step-up from code requirements (i.e. code-plus), it also only references the 100-year flood elevation and gaps remain within the recommended practices. Variables neglected by the CCM include vertical datum conversions from NGVD 29 to NAVD 88, adjustments for high tide, and accounting for global sea level rise (GSLR) that occurs from the FIS modeling date until the time construction begins on the structure. Collecting information about flood zones and elevations can be confusing in and of itself with all of the available sources (e.g. American Society of Civil Engineers, the International Code Council, FEMA, CCM), yet information on how BFE and SWEL numbers are determined can be very vague. Aspects of the gap in flood estimation have been called "wider than desirable" and "difficult to obtain" information on the approaches used and problems faced with flood design (Pilgrim 1986, 165S-166S). There is a need for a comprehensive method to estimate the DFE for any annual occurrence probability event that incorporates code requirements, regulations, best available practices, and new data on the changing coastal environment.

In the United States, nearly 9 million people (3.0% of the population) live in areas subject to the 1% annual chance coastal flood hazard (Crowell et al. 2010). A 1997 study found that there were over 6 million residential structures in Special Flood Hazard Areas (FEMA and Federal Insurance and Mitigation Administration 2002), a number which has likely increased in

the past 15 years as more development occurs in these areas. A better understanding of factors affecting floods and further elevation guidance can significantly decrease the risk for these 6 million structures and 9 million people. The NFIP indicates $1 billion in flood damages are avoided annually and structures built to NFIP criteria experience 80% less damage due to reduced occurrences and severity of losses (FEMA and Federal Insurance and Mitigation Administration 2002). Thus, guidance beyond NFIP minimums will serve to further decrease flood damage and create more flood resilient communities.

1.1 Problem Statement

Flood design practitioners are generally limited to information pertaining to the 100-year BFE. Flood code and wind code are not risk consistent with respect to the return period of each hazard's design event, thus lacking uniformity throughout the design process. Code-plus methodologies are available to help practitioners consider the effects of the changing coastal environment, but neglect certain aspects of the DFE process. FEMA provides rule of thumb guidance to estimate the 500-year SWEL, but does not provide guidance to estimate other, longer return period flood elevations. While recent literature (e.g. CCM) provides guidance beyond flood insurance rate map (FIRM) BFEs, the practitioner is severely limited in determining any DFE beyond the 100-year BFE.

1.2 Goals and Objectives

Mitigation and adaptation techniques for flooding in coastal high hazard (V-Zones) is currently limited to elevating structures (Bellomo et al. 1999). Therefore, the goals of this thesis are to improve the understanding of coastal flooding in the changing environment and to provide guidance to practitioners to consider increased mitigation and adaptation for buildings designed in the coastal environment. As a step toward accomplishment of these goals, the following objectives are undertaken:

- To evaluate current code and code-plus guidance regarding building elevation in coastal high hazard areas
- To identify existing gaps in code and code-plus practices and to recommend practices to bridge identified gaps
- To evaluate existing FIS and develop practitioner-oriented guidance for estimating longer return period flood elevations (e.g. 700 and 1700 years) from existing FIS

1.3 Scope of Study

The DFE methodology presented in Chapter 2 is developed through review and synthesis of coastal processes, ASCE, IBC, FEMA regulations, and the CCM (code-plus) requirements. Gaps among these requirements will be noted, and solutions will be presented in the comprehensive methodology. Chapter 2 should be used by practitioners in flood zones that experience erosion, global sea level rise (GSLR), subsidence, and whose flood maps are referenced to NGVD 29. The return period analysis in Chapter 3 will be conducted using FIS and SWEL data from 16 coastal communities distributed along the U.S. Atlantic and Gulf Coasts. Guidelines for estimation of longer return period flood elevations will be presented. ASCE category III & IV structures whose failure could pose a “substantial risk to human life” and a “substantial economic impact” or “could pose a substantial hazard to the community” represent critical facilities such as schools serving as shelters, sewage treatment and power plants, and nuclear hospitals and police stations (ASCE 2005, p. 7; ASCE 2010, p. 2). ASCE code requires Category III & IV structures to be built to the 1700-year wind, yet the maximum elevation required by code is the 100-year BFE plus 2-feet of freeboard (ASCE 2005; ASCE 2010).

It is important to note that data are taken from existing FIS and that flood modeling, historical flood marks, and tidal gage data outside of the FIS are not included in this thesis.

Chapter 3 can be used as guidance in initial development phases for critical facilities which warrant a full probabilistic surge model. Chapter 3 can also be used in communities for schools and government buildings for a more scientifically based process to determine DFE. The findings of this thesis are not intended to be used to change code or insurance requirements, but rather to provide guidance to practitioners wishing to design beyond the 100-year BFE and include changes occurring in the coastal environment.

1.4 Limitations of Study

The extent of this study is limited to those structures located in coastal high hazard zones. Chapter 2 is presented as generalized design recommendations; however, implementation of the methodology requires knowledge of a specific local environment in order to determine effects caused by local characteristics. If the practitioner's locale is not in a special flood hazard area, methodologies for determining wave effects different than presented in Chapter 2 should be used for more reliable results. FIS SWELs are normally reported as a maximum probability elevation for an individual coastal transect; therefore, using an elevation from Chapter 3 for an A-Zone will result in a significantly higher elevation than desired for that given locality. The FIS used for calculations in Chapter 3 only represent the Gulf Coast and Atlantic Coasts and are separated by an average of 206 miles with a shortest distance of approximately 90 miles and a maximum distance of approximately 306 miles between communities. FIS data were limited to those communities that provided a SWEL for the 10% annual chance (10-year), 2% annual chance (50-year), 1% annual chance (100-year) and 0.2% annual chance (500-year) events. The results of Chapter 3 will be more accurate for locations contained within one of the selected FIS. For those areas between selected FIS communities, assumptions must be made outside of the scope of this thesis to produce results for the desired location.

1.5 Organization of the Thesis

This thesis is organized into two separate manuscript chapters, followed by an overall summary and conclusions. Chapter 2 relies on combining code requirements, best practices and developing new recommendations for existing gaps among current methods. Chapter 2 is intended to provide a single source for code and code-plus practices for flood elevation information in Special Flood Hazard Areas. Chapter 3 examines the extrapolation of existing FIS SWELs to estimate flood elevations for the 700 and 1700 year flood in an effort to facilitate risk-consistent development among flood and wind hazards.

1.6 Definitions

There are many terms and variables that are utilized with respect to coastal flood hazards, building elevation design, and the NFIP. The flowing definitions (Table 1.1) are provided to serve as a resource to the reader for use with this thesis.

Table 1.1 Definitions and Variables Used to Describe Coastal Flood Hazards, Building Elevation Design, and the NFIP

Term	Definition
Base Flood Elevation (BFE)	Elevation of flooding, including wave height, having a 1% chance of being equaled or exceeded in any given year; minimum NFIP requirement (ASCE 2005).
Coastal A Zone	Areas where the stillwater depth of the BFE above the eroded ground elevation is greater than or equal to 1.9 ft, sufficient to support a wave height greater than or equal to 1.5 ft, and where conditions are conducive to the formation and propagation of such waves (ASCE 2005).
Coastal High Hazard Area (Includes V-Zones)	Areas designated as subject to high velocity wave action on a community's flood hazard map (V-Zones) (ASCE 2005; ICC 2009); or where the SWEL of the BFE above the eroded ground is greater than or equal to 3.8 ft, sufficient of supporting a wave height equal to or greater than 3ft subject to high-velocity wave action or wave-induced erosion; these include V-Zones (ASCE 2005; ICC 2009). Or, an area within a special flood hazard area extending from offshore to the inland limit of a primary frontal dune along an open coast and any other area that is subject to high velocity wave action (ASCE 2005).

Table 1.1 Continued

Term	Definition
Depth	A depth is the measurement from an object to the ground. Depth is not equivalent to elevation.
Design Flood Elevation (DFE)	Elevation of the design flood, including wave height, relative to the datum specified on a community's flood hazard map (ASCE 2005). This only applies to communities that have elected to exceed NFIP requirements; for a community adopting minimum NFIP requirements, the DFE defaults to the BFE (ASCE 2010).
Design flood protection depth (d_{fp})	The vertical distance between the eroded ground elevation and the DFE. This concept only applies to CCM calculations and it is important to note it is *not* an elevation.
Design stillwater flood depth (d_s)	The vertical distance between the eroded ground elevation (GS) and the stillwater elevation (SWEL) associated with the design flood (FEMA 2011). This concept only applies to CCM calculations and it is important to note it is *not* an elevation.
Diurnal Tide Cycle	One high tide (MHHW) and one low tide (MLLW) per day (U.S. Department of Commerce 2000).
Elevation	An elevation within the context of this thesis is a height always referenced to a recognized vertical datum (e.g. NGVD 29 or NAVD 88). Elevation is *not* equivalent to depth.
Erosion	Refers to the wearing or washing away of coastal lands and commonly refers to horizontal recession of the shore (shore erosion) (FEMA 2011). Erosion can also refer to “seabed erosion” which occurs when sediments are transported offshore, resulting in a lowering of the seabed, which increases local water depths and wave heights reaching the shore (FEMA 2011). Both types of erosion are used when determining the erosion effects from the CCM.
Design Stillwater Flood Elevation (E_{SW})	The design stillwater flood elevation in feet above datum as used in the CCM.
Global (Eustatic) Sea Level Rise (GSLR)	Global sea level, which changes in response to changes in the volume of ocean water and volume ocean basins (Schlumberger 2012). This differs from relative (local) sea level rise, which includes changes in land movement. Global Sea level changes are the same regardless of where measurements are taken (McCue 2010).
Great Diurnal Range (GT)	The difference in height between mean higher high water and lower low water for both the semidiurnal and diurnal cycle (U.S. Department of Commerce 2000)
Lowest Eroded Ground Elevation (GS)	Lowest eroded ground elevation in feet above datum, adjacent to a building, excluding effects of localized scour around the foundation as used in the CCM (FEMA 2011).

Table 1.1 Continued

Term	Definition
Lowest Horizontal Structural Member (LHSM)	For an elevated structure, the lowest beam, joist, or other horizontal member that supports the building is the LHSM (FEMA 2011). Grade beams for vertical foundation members do not count as LHSM (FEMA 2011)
Maximum Breaking Wave (H_b)	The maximum breaking wave is defined as 78% of the stillwater depth (FEMA 2002; FEMA 2011).
Maximum Wave Crest	The wave crest is 70 percent of the breaking wave height (H_b) above the stillwater level (FEMA 2003; FEMA 2011). This can also be calculated as 1.546 times the stillwater depth.
Mean High Water (MHW)	The average of all the high water heights, the maximum height reached by a high tide, observed over the 19- year National Tidal Datum Epoch, and only applies to semi-diurnal cycles (U.S. Department of Commerce 2000).
Mean Higher High Water (MHHW)	The average of the higher of the two high waters of each tidal day for a semi-diurnal cycle, and the single high tide for diurnal cycles observed over the 19 year National Tidal Datum Epoch (U.S. Department of Commerce 2000).
Mean Low Water (MLW)	The average of all the low water heights, the minimum height of the low waters, observed over the 19- year National Tidal Datum Epoch, and only applies to semi-diurnal cycles (U.S. Department of Commerce 2000).
Mean Lower Low Water (MLLW)	The average of the lower of the two low waters of each tidal day for a semi-diurnal cycle, and the single low tide for diurnal cycles over a 19- year tidal datum epoch (U.S. Department of Commerce 2000).
Mean Range of Tide (MN)	The difference in height between the mean high water and the mean low water and only applies to a semidiurnal cycle (U.S. Department of Commerce 2000).
Mean Sea Level (MSL)	The arithmetic mean of hourly tide heights observed and recorded over the 19- year National Tidal Datum Epoch (U.S. Department of Commerce 2000).
National Tidal Datum Epoch	Specific 19-year period adopted by the National Ocean Service (NOS) as the official time segment over which tide observations are computed to obtain mean values (U.S. Department of Commerce 2000).
Semi-diurnal (mixed) Tide Cycle	Two high tides and two low tides per day, with one of the two high tides being higher than the other and one of the two low tides being lower than the other (U.S. Department of Commerce 2000; Zevenbergen et al. 2004).

Table 1.1 Continued

Term	Definition
Still Water Elevation (SWEL)	The base water surface elevation upon which the waves ride; it consists of Mean Sea Level (MSL), astronomic tide, and surge (FEMA 2007). Waves are excluded from the SWEL (FEMA 2003); a SWEL is not a depth.
Subsidence	Subsidence is a lowering of ground elevation that results from a number of natural and human processes: river diversion and damming preventing re-nourishment of soils (Milliman and Haq 1996); withdrawing large amounts of ground water/petroleum from certain types of rocks (Waller 1982; Milliman and Haq 1996); removal of native vegetation (Zektser et al. 2005); and compaction of the Holocene strata (Tornqvist et al. 2008).
Vertical Datum	A zero elevation coordinate surface and methods of determining heights relative to that surface (Vanicek 1991; National Geodetic Survey 2001). Common National Geodetic Vertical Datum of 1929 (NGVD 29), North American Vertical Datum of 1988 (NAVD 88)

CHAPTER 2: A COMPREHENSIVE METHOD TO DETERMINE DESIGN FLOOD ELEVATIONS FOR STRUCTURES IN COASTAL HIGH HAZARD FLOOD ZONES

2.1 Introduction

Financial losses attributed to floods in the U.S. total $2.4 billion annually, and more than 75% of all federal disaster declarations are related to flood events (Taggart and van de Lindt 2009; Li and van de Lindt 2012). Further, nearly 9 million people (3.0% of the U.S. population) live in areas subject to the 1% annual chance (100-yr) coastal flood hazard (Crowell et al. 2010). Improving our understanding of coastal flood elevations and the factors affecting these elevations within the coastal environment can result in improved design and construction practices that are based on flood risk and probabilities of failure. These practices, if properly implemented, have potential to reduce flood-induced losses.

The 1% annual chance flood is the basis for current standards, which attempt to minimize flood losses by requiring buildings to be elevated at or above a base flood elevation to prevent water from entering a building. Many aspects of the gap in flood estimation are “wider than desirable” and it is “difficult to obtain” information on the approaches used and problems faced with flood design (Pilgrim 1986, 165S-166S). In certain situations it may be more cost-effective to elevate a building above floodwaters than to structurally build to protect against the forces of floods. There is a need for a revised, comprehensive methodology to determine a more reliable design flood elevation (DFE) for buildings in coastal communities that is able to be implemented for any flood recurrence interval.

NFIP Flood Insurance Studies (FIS) and FIRMs do not account for the effects of long-term erosion, subsidence, or sea level rise, all of which should be considered when establishing lowest floor elevations in excess of the BFE (FEMA 2011). In addition, for some communities, the BFE is referenced to an outdated vertical datum (NGVD 29) and the highest of tides are not

included in the BFE modeling process. Therefore, the actual risk of flooding is not equivalent to the intended, or design, risk. All of these long-term effects should be considered in order to formulate a DFE that addresses the effects of changing coastal characteristics. Flood research has to date not addressed this issue, and inadequacies in past research have been identified, specifically in the fundamental decisions underlying design solutions (Pilgrim 1986). Furthermore, current flood design methods include the addition of "freeboard", which is defined as additional elevation between the lowest structural member and the BFE, resulting in the DFE, which is current basis for design (FEMA 2011). The amount of freeboard is determined by ASCE standards that are incorporated into the building code or by community adopted freeboard requirements in excess of the national ASCE standards. Freeboard may be assumed to be largely arbitrary given that flood elevations for storms of selected recurrence intervals are calculated for each given locality, yet freeboard required by code is the same amount for the entire country providing an unknown level of protection.

The Coastal Construction Manual (CCM), currently considered the best practice available, provides code-plus design recommendations that include long-term effects of subsidence, sea level rise, and erosion (FEMA 2011; FEMA 2011). However, the CCM does not account for simultaneous high tides and peak surges, or the total amount of global sea level rise, and does not explicitly detail vertical datum transformations. The methodology presented in this chapter builds on the recommendations of the CCM and should be considered and combined with current best practices available to develop a DFE that will reduce the overall risk of flooding, thus reducing flood damage from water penetrating the structure. Flood engineering design and decisions are currently too closely linked with insurance policy decisions. The intent of this chapter is to provide scientifically based recommendations that focus on the actual risk to determine a DFE for the desired level of protection.

In this chapter, current code and best practices are reviewed and analyzed in the development of a comprehensive method that addresses current design gaps. The proposed methodology section introduces the recommended methods combined with current code and practices to arrive at the proposed DFE. In addition, a case study is provided to show the implementation of the proposed DFE methodology. The methodology recommended by this chapter is not intended to apply strictly within the current insurance policy guidelines. That is, the authors are not making a recommendation about what the National Flood Insurance Program or other insurers should accept as the required bottom elevation of a building. The purpose of this chapter is to provide designers with concise additional guidance to develop a building DFE for a given return interval that integrates all known considerations, codes, and practices.

2.2 Current Code and Best Practices Literature

Current building code requirements in the U.S. integrate International Code Council (ICC) codes and American Society of Civil Engineers (ASCE) standards. The International Residential Code (IRC) Section 322.1 General, specifies that, "Buildings and structures located in whole or in part in identified floodways shall be designed and constructed in accordance with ASCE 24" (ICC 2009). The International Building Code (IBC) Section 1612.4 Design and Construction states, "The design and construction of buildings and structures located in *flood hazard areas*, including flood hazard areas subject to high-velocity wave action, shall be in accordance with Chapter 5 of ASCE 7 and with ASCE 24" (ICC 2011).

The standard *ASCE 7 Minimum Design Loads for Buildings and Other Structures* provides minimum load requirements for the design of buildings that are subject to code requirements (ASCE 2010). Chapter 5, *Flood Loads*, of ASCE 7 presents information for the design of buildings and structures in areas prone to flooding including erosion and scour; loads on breakaway walls; hydrostatic loads; hydrodynamic loads; wave loads and breaking wave

loads on vertical walls; and impact loads from debris (ASCE 2010). ASCE 7 covers flood loads to be considered when designing, whereas the *ASCE 24 Flood Resistant Design and Construction* standard addresses elevation requirements when designing for floods.

ASCE 24 puts structures into four risk categories classified by the nature of occupancy (Table 1-1 of ASCE 24). As required by ASCE 24, the height required above the BFE is to be determined by the structure category and the lowest horizontal structural member's (LHSM) orientation to the direction of wave approach, which is presented in ASCE Table 4-1 of Section 4.4, *Elevation Requirements*, and summarized in the bulleted list below (ASCE 2005).

- Elevate to DFE for structure category I (LHSM parallel or perpendicular to wave approach), and category II (LHSM parallel to wave approach only).
- Elevate to maximum of BFE + 1' or DFE for structure category II (LHSM perpendicular to wave approach only), and structure category III and IV (LHSM parallel to wave approach only).
- Elevate to maximum of BFE + 2' or DFE for structure category III and IV (LHSM perpendicular to wave approach only).

Figure 2.1 presents a flow chart that graphically depicts the process of selecting the DFE as required by ASCE 24. Included in the ASCE DFE determination process is an evaluation of community-adopted freeboard and freeboard required by ASCE 24 Table 4-1. Community-adopted freeboard is any amount of freeboard a community requires above the minimum requirements and results in decreased insurance rates due to decreasing the risk of flooding (FEMA 2012). In addition, a community may also be known as an Authority Having Jurisdiction (AHJ) and may provide other guidelines above the 1% annual-chance flood.

FEMA's CCM provides guidance above current code and standard guidance (FEMA 2011; FEMA 2011). The CCM is currently considered the best practice available to designers

and homebuilders. The CCM was recently revised under the guidance of a technical committee of national experts, and provides guidance for incorporating the effects of coastal processes that are not addressed in current codes, standards, or NFIP products. In addition to the FIRM-specified BFE, which does not consider any long term effects, the CCM provides guidance to determine the effects of subsidence (if any) on the site, the most landward expected shoreline of the building over the anticipated life of the building, the lowest expected eroded ground elevation at the base of the building or structure, and the highest expected stillwater depth at the building (FEMA 2011). The CCM provides four ways of determining flood protection elevations (Table 2.1). This chapter considers only Option III, which is the most comprehensive method by accounting for most factors and future conditions that may increase flood risk.

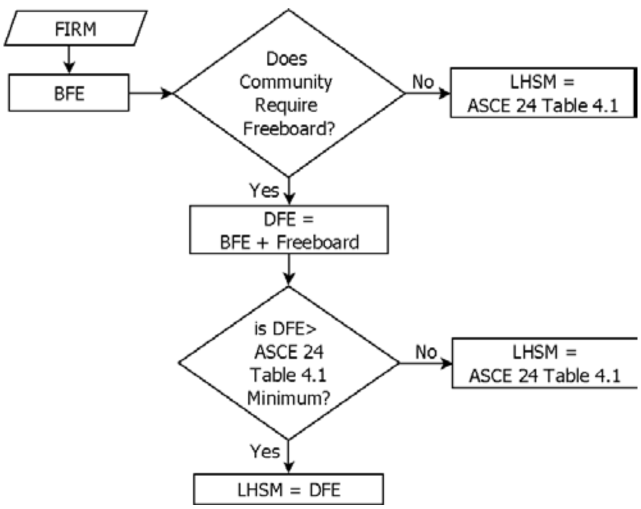


Figure 2.1 ASCE 24 DFE Determination Process

Table 2.1 CCM Building Elevation Options

Option	Description
I.	100-year SWEL. Future Conditions not considered.*
II.	100-year SWEL plus freeboard. Future conditions not considered.*
III.	100-year SWEL. Future Conditions including GSLR and long-term erosion considered for a specified amount of time.*
IV.	500-year wave crest elevation using the AHJ 500-year wave crest. Future conditions not considered.

*Options I-III are only d_s, the design stillwater flood depth, not an elevation or DFE. In order to arrive at DFE add the d_s back to the GS, lowest eroded ground elevation, and then multiply by 1.546 to get to DFE for Coastal High Hazard Zones.

2.3 Gaps in Current Code and Practices

Although the CCM increases the level of protection over the ASCE 24 standard, room for improvement remains. Many of the effective FIS and FIRM elevations, as well as guidance from the CCM, are referenced to NGVD 29, an older vertical datum currently going out of use. While using the correct vertical datum is required for a structure's elevation certificate to be approved, designers and developers should be provided with better guidance between the available vertical datums. Neither ASCE nor the CCM provide any such guidance for converting from NAVD 88- or NGVD 29-referenced elevations. Additionally, the CCM introduces the concept of global sea level rise (GSLR), but does not account for the total level of GSLR. This total level includes increases in sea level during the lifetime of the structure in addition to GSLR that occurs prior to construction. The consideration of high tides should also be included in the design flood elevation. The effects of storm surge occur over an extended duration and may in fact occur simultaneously with the highest tide of the day, therefore increasing the risk of flooding. Finally, FIS have started providing some, yet minimal, information regarding the 500-year flood; however, most coastal communities still have no guidance for designing above the 100-year BFE. Return intervals greater than the 100-year BFE should be considered. These gaps are discussed in further detail in the following sections.

2.3.1 Datum Conversions

A vertical control datum (e.g. NGVD 29 and NAVD 88) is a set of fundamental elevations to which other elevations are referred (Vanicek 1991; National Geodetic Survey 2001). Many FIS, FIRMs, and CCM recommendations are still referenced to NGVD 29, which is going out of use, rather than NAVD 88, which is widely accepted nationwide (Iliffe and Lott 2008). NGVD 29 is the National Geodetic Vertical Datum of 1929, also known as the Sea Level Datum of 1929 until May 10, 1973 (Meyer et al. 2004; National Geodetic Survey 2011).

Thousands of reference benchmarks used for NGVD 29 have been destroyed or affected due to crustal motion, highway construction, postglacial rebound, and subsidence caused by withdrawing underground fluids (Meyer et al. 2004; National Geodetic Survey 2011). Therefore, the reliability of NGVD 29 benchmark elevations will not be consistent with the original elevation mark they were assigned and should not be used with a high degree of accuracy.

NAVD 88 is the North American Vertical Datum of 1988, which was established in June 1991 by a minimum-constraint adjustment of the Canadian-Mexican-U.S. leveling observations, referencing the height of the primary tidal bench mark of the new International Great Lakes Datum of 1985 at Father Point/Rimouski, Quebec, Canada (Meyer et al. 2004; National Geodetic Survey 2011). NAVD 88 offers the advantage of over 625,000 km (approx. 38,835 miles) of leveling compared to 106,724 km of leveling for NGVD 29 (National Geodetic Survey 2011). Distortions between NAVD 88 and NGVD 29 vary from -15.7 inches to +59.05 inches for the conterminous United States (Meyer et al. 2004). Therefore, it is important to ensure that FIS and FIRM elevations are referenced to NAVD 88 in order to account for these differences given that NAVD 88 is currently the most reliable vertical datum.

Although many FIS and FIRMS including recommendations of the CCM are still referenced to NGVD 29, rather than the current NAVD 88, neither ASCE nor the CCM provide any guidance or recommendations for a developer or designer to convert an elevation from NGVD 29 to NAVD 88. If a FIS is referenced to NGVD 29, no datum transformation is mentioned, even on FIS with recent effective dates. Some FIS referencing NAVD 88 provide a small section on the differences in values, but rarely provide the methodology used to derive the values. The only guidance for converting to NAVD 88 is in Appendix B of FEMA's *Guidelines and Specifications for Flood Hazard Mapping Partners* (FEMA 2003). The issue is this

information is provided to those developing new flood maps and is not readily available to designers and practitioners who need to perform a datum conversion for older maps still referenced to NGVD 29. Although this issue has not been discussed in the literature, the authors have first-hand observations of practicing engineers, academics, and many others ignoring this transformation because they are not familiar with the need for datum transformation. It is of utmost importance to recognize that the correct elevation associated with the correct vertical datum is required for approval of the elevation certificate. If the elevation certificate is not approved, then the building will not be permitted for occupancy or use.

One should recognize that not all datum conversions will result in an additive value to the NGVD 29 elevation, but that some can be subtracted. As an example, at Marco Island in Collier County, Florida, to arrive at the NAVD 88 elevation, there is a -1.30 foot difference from the NGVD 29 elevation (FEMA 2009). Many datum transformations in Florida are exceptions where one might subtract from the NGVD 29 elevation to arrive at the NAVD 88 elevation.

2.3.2 Global Sea Level Rise

While GSLR projections are important for considering future effects, GSLR which has occurred after the SWEL modeling date but prior to construction is important as well for determining the total starting SWEL and eventually the DFE. While subsidence affects the elevation of the structure due to a ‘sinking’ effect, GSLR affects the elevation of the flood by raising the normal water levels along the coast. By the year 2100, FEMA projects that the area normally inundated by the 100-year flood (19,500 square miles) will increase to 23,000 square miles with a 1-foot relative sea level increase and to 27,000 square miles with a 3 foot relative sea level increase (FEMA and FIA 1991). This is expected to increase the number of households in the floodplain by 2100 from 2.7 million (as of 1991) to 5.7 million with a 1-foot sea level increase and 6.8 million with a 3-foot sea level increase (FEMA and FIA 1991). GSLR effects

raise the SWEL used in FIRM BFE calculations and the E_{SW} used in CCM calculations (FEMA 2011). An increase to the SWEL and E_{SW} directly increase the maximum wave crest, thus increasing the risk of a structure to flooding. GSLR raises overall flood levels and increases flood risk.

The CCM includes the effects of sea level rise over the useful life of the structure, but it does not account for GSLR that occurs between the effective SWEL modeling date and the start of construction. When a new FIS is completed, it is not immediately effective, as the adoption process takes some time, even years. Therefore, it is important to use the modeling date of the SWEL, as this provides the date that the BFE was computed. If a community publishes a newly effective FIS with the same SWEL from an older modeling date, then the community has not accounted for the all effects of GSLR. For example, if an FIS was published in 2010 and uses the same SWEL that was modeled in 1970, then forty years of GSLR have not been accounted for. SWELs that remain unchanged from one effective FIS to the next are not uncommon, thus neglecting changes in sea level over that time period may have considerable effects. GSLR that has already occurred but is unaccounted for must be considered through the use of recorded historical GSLR if the modeling date is prior to 1990. For modeling dates after 1990, the projected annual rate for GSLR assuming a linear distribution of 0.03 feet per year (0.36 inches per year) is recommended (Rahmstorf 2007; DeMarco et al. 2012).

Future effects of GSLR are included in the CCM and are based on projections. Current climatological science projects increases in GSLR in the range of 0.5 to 1.4 meters (1.6' – 4.6') from 1990-2100, with 1 meter (3.3') being the most likely (Rahmstorf 2007; DeMarco et al. 2012). The Louisiana Coastal Protection and Restoration Authority (CPRA) Technical Report recommends that CPRA staff assume that Gulf sea-level rise will be 1 meter, 3.3' by 2100, with a bounding range of 0.5 – 1.5 meters (1.6' – 4.9') (DeMarco et al. 2012). A 1-meter increase in

sea level will put an additional 14 million people at risk for flooding by 2100, and by 2080, sea level rise will cause nearly five-times as many people to be flooded than those flooded during a typical year from storm surge (Nicholls et al. 1999). Failure to acknowledge or account for an increase in GSLR has the potential to greatly increase design flood damage and underestimate the risk associated with changing coastal characteristics. Accounting for GSLR is not currently required by the building code.

2.3.3 Consideration of Tides

Given the long duration of storm surge events, it is very likely that a surge event will occur simultaneously with a high tide, therefore directly increasing the likelihood of flooding. In order to provide for maximum protection against flood inundation, the DFE should be referenced to the highest tide, mean higher high water (MHHW), in the event that a surge event occurs during high tide. In the event that a DFE is referenced to MSL and peak storm surge occurs during the highest normal tide of the day (MHHW), the DFE may not accurately account for the actual risk. According to the USGS, worst-case scenario surge events occur when high tide and peak storm surge occur concurrently (Perry 2000). It is the assertion of the authors that these are not truly worst-case scenarios, but rather that they should be anticipated scenarios and included in the design process.

FEMA's *Guidelines and Specifications for Flood Hazard Mapping Partners* [February 2007] explicitly states "if the surge duration is short – such as may be typical for hurricanes in northern latitudes – this approximation [linearly adding the high tide to predicted storm surge levels to determine SWEL] is inadequate" (FEMA 2007, D.2.4-23). In this case, FEMA recommends assuming there is an equal probability that peak surge will occur during high tide or low tide and to take the mean high and mean low as representative values to be used in the frequency analysis as 50% of the corresponding total water elevations (FEMA 2007). Designing

from these recommendations potentially increases the risk of flood inundation due to underestimating total surge levels in the event that peak surge does occur during high tide. In contrast, when speaking of Hurricane Sandy's (2012) storm surge, storm surge modeling expert Rick Luettich stated that "its [Hurricane Sandy's] effects will extend longer than a single tidal cycle" (Drye 2012). In order to be considered a best practice, high tide should be recognized as a naturally occurring cycle taking place once or twice daily, therefore high tides should be included in SWELs as an effort to mitigate risk through designing against all conditions.

Tides are not always included in SWEL modeling. If they are not included, a significantly low calculation of the FIS SWEL will result; if tides are included and referenced to MHHW this chapter recognizes this as a best practice. It can be determined whether or not the effects of tides are included in the determination of the FIS SWEL by reviewing the community's technical support data notebook (TSDN). In addition to determining whether or not tides are included it is also important to determine which tidal datum the SWEL references.

There are 5 main tidal datum based on two types of cyclical periods that are summarized in Table 1.1. For the proposed methodology, it is recommended to use the MHHW tidal datum when determining a new SWEL to account for the possibility of peak storm surge events occurring during the MHHW levels; demonstrated by Figure 2.2. Including MHHW levels increases the SWEL, which increases the maximum wave crest and the chance for flooding.

To illustrate the severity of the storm surge durations the following storms and locations were used in Figure 2.2: Hurricane Isaac (2012) at Bay Waveland, Mississippi; Hurricane Ike (2008) at Galveston, Texas; and Hurricane Ivan (2004) at Dauphin Island, Alabama. Time 0 on Figure 2.2 represents landfall for the eye of the hurricane. The following three examples are cases of recently observed tidal data during surge events referenced to MHHW, demonstrating how high surge levels remain above the highest predicted tide for a given time during hurricanes.

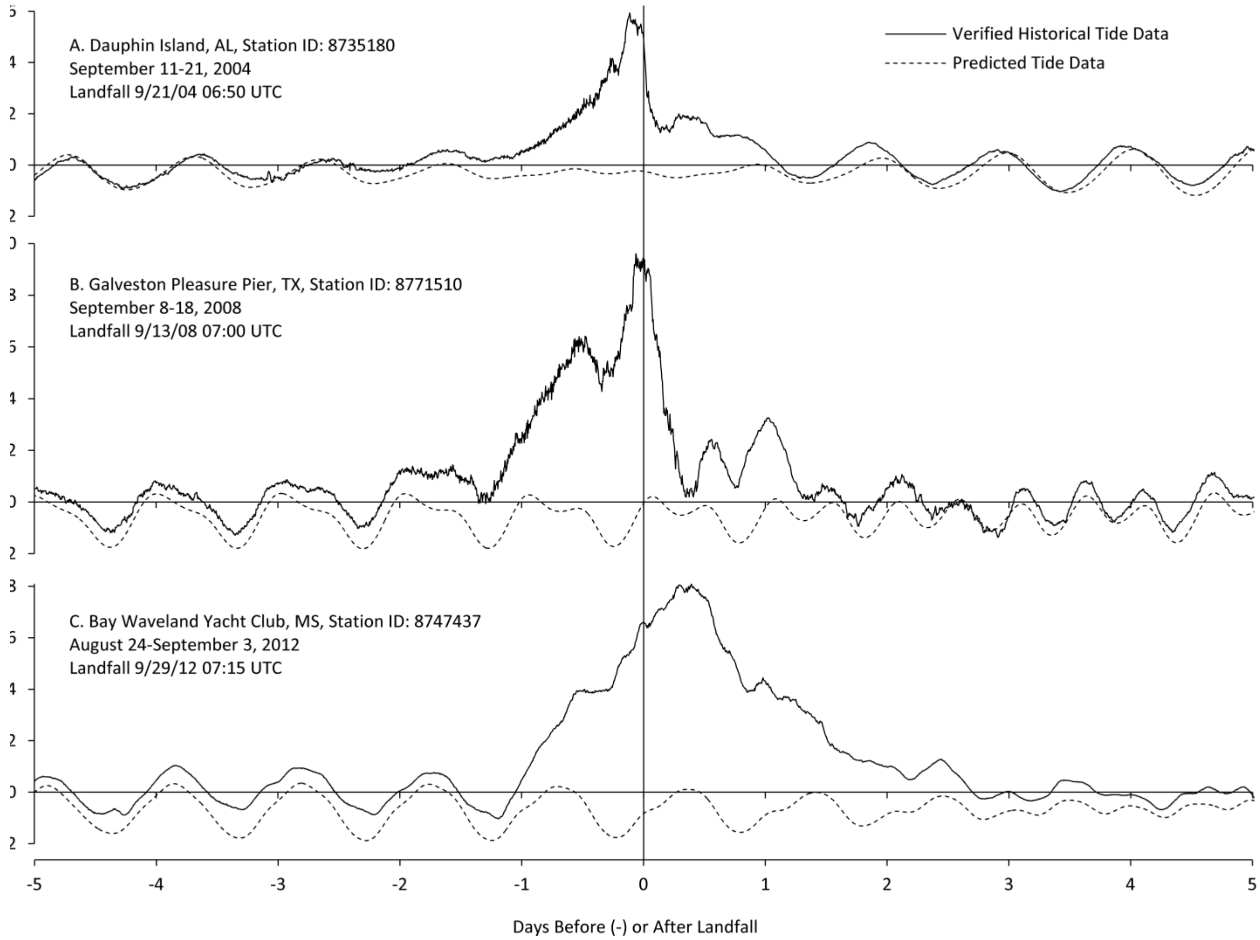


Figure 2.2 Tidal Gauge Data Collected by NOAA Co-Ops Buoys During Hurricanes (A) 2004 Ivan, (B) 2008 Ike, and (C) 2012 Isaac

In all three cases, the peak surge occurred almost simultaneous with the normal predicted high tide. The maximum surge elevation above the predicted high tide level was 6.19 feet for Hurricane Isaac, 10.2 feet for Hurricane Ike, and 7.99 feet for Hurricane Isaac. The maximum surge levels above the normal GT (MHHW to MLLW range) associated with the selected storms are 459% above the 1.74 foot GT for Waveland, MS. (Currents, 2012a); 500% above the 2.04 foot GT for Galveston, TX. (NOAA Tides & Currents 2012); and 515% above the 1.20 foot GT for Dauphin Island, AL. (NOAA Tides & Currents 2012).

Figure 2.3 shows the duration of the surge elevations from Figure 2.2 as the percentage of a 24-hour day for three intervals of the maximum surge level: 40%, 60%, and 80%. A value on

the Y axis above 100% indicates that the indicated level of surge was experienced at the station location for greater than one 24-hour day.

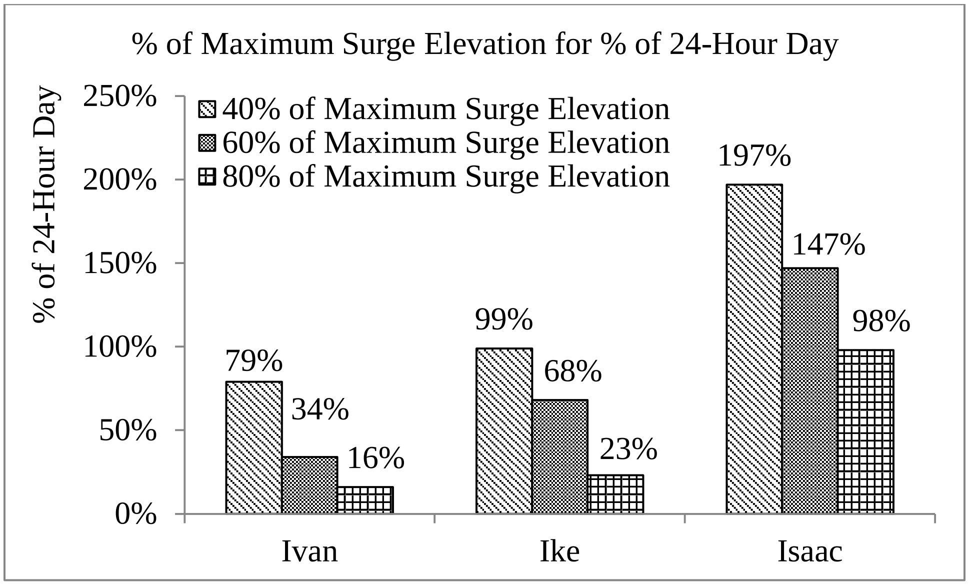


Figure 2.3 Percentages of the Maximum Surge Elevation Shown as a Percentage of a 24-hour Day.

For the selected hurricanes, 40% of the maximum surge elevation occurs simultaneously with the predicted high tide for 79% to 197% of a 24-hour day; 60% of the maximum surge elevation occurs simultaneously with the predicted high tide for 34% to 147% of a 24-hour day; and 80% of the maximum surge elevation occurs simultaneously with the predicted high tide for 16% to 98% of a 24-hour day. These statistics verify the need to include high tide effects with an estimated storm surge level by demonstrating through past events the high likelihood of elevated surge levels occurring simultaneously with the predicted high tide. Additionally, two of the three examples (‘A’ Ivan and ‘C’ Isaac) are only indicative of a diurnal tidal cycle, one high and low per day, representing a lower likelihood of simultaneous occurrence than a mixed or semi-diurnal cycle (‘B’ Ike). Thus, if the tidal cycle is mixed or semi-diurnal, two high and low tides per day, the possibility of simultaneous high tides and peak surge levels is doubled on the average.

As demonstrated above in Figure 2.2 and 2.3, storm tides can be abnormally high for a duration extending across multiple high tides thus increasing the opportunity for the two to occur simultaneously. Therefore, incorporating MHHW rather than a weighted average of high and low tides accounts for the highest risk associated with natural tidal cycles given at least one high and low tide per day tides and simultaneous peak storm surge elevations. To prevent damage by elevating above flood waters, making sure MHHW is included in the DFE is imperative.

2.3.4 Flood Return Intervals

Within the expected 60 year lifespan of a typical building, defined as an ASCE risk category II structure (ASCE 2010), the probabilities of the 100-, 500- and 700-year events occurring during a 60-year useful life are 45%, 11%, and 8% respectively (Friedland and Gall 2012). This level of risk associated with flood return periods is inconsistent with wind design risk levels. ASCE 7 requires the following design levels for wind events (ASCE 2010):

- Risk category I structures, defined as those that represent a low risk to human life in the event of failure, to be designed for wind speeds that correspond to the 300-year mean recurrence interval (MRI)
- Risk category II structures, defined as all buildings and structures except those listed as I, III, and IV, shall be designed for wind speeds that correspond to the 700-year MRI
- Risk category III and IV structures, defined as buildings and structures which pose a substantial hazard to the community and substantial risk to human life, shall be designed for wind speeds that correspond to the 1700-year MRI

While the minimum design standard for wind is the 300-year MRI, the maximum design standard for flood is the 100-yr MRI. As hurricanes are a hazard with coupled risk (i.e. wind and coastal flooding), this incredible imbalance of risk is not in line with current optimized risk-based design.

FEMA provides minimal guidance for designing above the 100-year BFE. A minority of FIS have started to provide a 500-year SWEL, but if this is not available then the only guidance for design beyond 100-yr elevations is provided by FEMA as rule of thumb to approximate the 500-year Stillwater elevation (SWEL) as 1.25 times the 100-year Stillwater elevation (FEMA 2002; FEMA and NAHB Research Center 2010). In addition, the CCM mentions that an AHJ may specify a non-100-year frequency-based DFE to be used (FEMA 2011). There is no real guidance for developers to use when trying to design for flood events with longer return periods.

2.4 Proposed Design Practices

The proposed methodology to determine a DFE for the LHSM combines ASCE, CCM, and recommended practices for bridging the gaps in current code and practices. The methodology may be used in conjunction with any selected storm recurrence interval and is not restricted to the 100-year storm. The proposed practice presents new information in addition to referencing guidance from other sources such as the CCM.

The equations proposed to calculate the recommended DFE reflected in column five are presented in Equations 2.1 through 2.12. Each of the variables is discussed in further detail in the following subsections, including appropriate usage and origin or derivation of the associated equations.

$$SWEL_{88} = SWEL_{FIS} +/- Conversion \tag{2.1}$$

$$SWEL_{PRIOR} = (Date_{cons} - Date_{model}) * Historic_{GSLR} + SWEL_{88} \tag{2.2}$$

$$SWEL_{GSLR} = SWEL_{PRIOR} + (GSLR \times LIFE) \tag{2.3}$$

$$SWEL_{MHHW} = GT/2 + SWEL_{GSLR} \tag{2.4}$$

$$SWEL_{MHHW} = (MN/2 - GT/2) + SWEL_{GSLR} \tag{2.5}$$

$$Ground_{SUB} = Ground_{Exist} - (Rate_{SUB} * LIFE) \tag{2.6}$$

$$E_{landward} = E_{Long\text{-}term} \times LIFE \tag{2.7}$$

$$Drop = D_{Profile} \times E_{landward} \tag{2.8}$$

$$GS = Ground_{SUB} - Drop \tag{2.9}$$

$$d_{proposed} = SWEL_{MHHW} - GS \tag{2.10}$$

$$Proposed\ d_{fp} = ((d_{proposed} * 0.78)*0.7) + d_{proposed} \tag{2.11}$$

$$DFE_{PROPOSED} = Proposed\ d_{fp} + GS \tag{2.12}$$

where:

$SWEL_{FIS}$ is the elevation taken from FIS

Conversion = the output offset between NGVD 29 and NAVD 88 from the vertical transformation program. Note: an actual elevation may be given and used as '$SWEL_{88}$' rather than an offset equation.

$SWEL_{88}$ = the adjusted elevation from the datum conversion now referenced to NAVD 88

$SWEL_{PRIOR}$ = SWEL resulting from GSLR amount occurring between effective SWEL model date and construction start date

$Date_{cons}$ = the date construction is to begin on the structure

$Date_{model}$ = the modeling date of the current SWEL as published in the FIS

$Historic_{GSLR}$ = the published historical rate of GSLR; or amount of rise as a number over the given time period. If $Date_{model}$ is after 1990, $Historic_{GSLR}$ rate = GSLR

$SWEL_{GSLR}$ = the total combined effects of GSLR that occur prior to construction of the structure and over the structure's useful life

GSLR = the projected global sea level rise rate over the specified time frame

LIFE = the useful life of the structure in years

$SWEL_{MHHW}$ = the SWEL referenced to Mean Higher High Water tidal datum

GT = the great diurnal tide range (MHHW-MLLW)

MN = Mean Range of Tide (MHW-MLW)

$Ground_{SUB}$ = New ground elevation after accounting for effects of subsidence

$Ground_{Exist}$ = existing ground elevation

$Rate_{SUB}$ =Rate of subsidence

$E_{landward}$ = the amount of feet the shoreline translates horizontally towards the building

$E_{Long\text{-}term}$ = the long-term landward erosion rate of the shoreline; CCM recommends using a minimum rate of 1.0 ft/yr (FEMA 2011);

Drop = the amount in feet that the ground elevation drops at the seaward row of piles

$D_{Profile}$ = the calculated eroded dune profile; CCM uses a vertical to horizontal ratio of 1:50 (FEMA 2011)

GS = eroded ground elevation; need to reference NAVD 88 for the proposed methodology.

$d_{proposed}$ = the design stillwater depth

0.78 = the coefficient for the maximum breaking wave height of a depth-limited wave

0.70 = the coefficient of the percent of the breaking wave height that lies above the SWEL

Proposed d_{fp} = the Proposed design flood protection depth

$DFE_{PROPOSED}$ = the Design Flood Elevation resulting from the proposed methodology

2.4.1 Datum Conversion

The FIS and FIRM can be used to determine which vertical datum is used. If the FIRM and FIS reference NAVD 88, this section may be skipped and SWEL calculations can be resumed with Section 2.4.2. FEMA FIS can be used to determine the desired return period flood SWEL from the generally included 10-, 50-, and 100-year storms, with newer FIS including 500-year storms. Other recurrence intervals may be used provided they are approved by an AHJ. FIS include transect locations and data providing the approximate location of the elevations. The transect closest to the site of the structure with the SWEL corresponding to the desired storm frequency in the FIS summary of transects table will yield the most accurate results.

Programs such as Corpscon, VERTCOM or VDatum may be used for vertical datum transformations from NGVD 29 to NAVD 88. Of the coastal FIS evaluated for this thesis, most are still referenced to NGVD 29. The latitude, longitude, SWEL, and horizontal projection used for the structure's geographic location are typically necessary for such vertical datum transformations. To perform the vertical datum transformation, the SWEL elevation is the input variable, NGVD 29 is the input datum, and NAVD 88 is the output vertical datum. The transformation results in a new elevation referenced to NAVD 88, hereafter indicated as $SWEL_{88}$. If the vertical transformation program used produces an output showing a vertical offset rather than an elevation, Equation 2.1 can be used to arrive at the new $SWEL_{88}$.

2.4.2 Effects of GSLR

To account for the total GSLR, it may first be necessary to include GSLR between the effective SWEL model date and construction start date. When using the effective date of the FIS, it is necessary to review prior versions of the FIS and determine the modeling date for the current SWEL ($Date_{model}$). A difference in the modeling date and the effective date of the FIS is not uncommon. Therefore, using the modeling date accounts for the total amount of sea level rise determined from the date of modeling until the date of construction and should be added to the SWEL. If the modeling date is prior to 1990, then documented historical rates of GSLR should be used to better represent actual conditions. After 1990, $Historic_{GSLR}$ (Equation 2.2) is estimated based on the recommendation of using a linearly projected rate of 0.03 ft/yr. To determine the new SWEL resulting from GSLR that has occurred from the modeling date of the FIS SWEL until construction begins, Equation 2.2 should be used. The result, $SWEL_{PRIOR}$, should be added to the GSLR that is expected to occur over the useful life of the structure (Equation 2.3).

To determine GSLR over the useful life of the structure, the process proposed in the CCM is recommended (FEMA 2011, 8-13). As recommended in the CCM (pg 8-13), the 100-year SWEL should be increased by the product of the GSLR rate (0.03 feet per year) and the expected useful life of the structure (FEMA 2011). The result should be combined with $SWEL_{PRIOR}$ (Equation 2.3) to obtain $SWEL_{GSLR}$, which includes all effects of GSLR after the modeling date of the SWEL.

2.4.3 Tidal Influence

If possible, determine if the SWEL modeling is calibrated to MHHW, if it does not reference MHHW, determine which tidal datum the SWEL references. On occasion a tidal datum may be noted in the Flood Insurance Study, but of the observed FIS, most mention only astronomical tide calibration in the modeling process, which includes a weighted average of high and low tides (FEMA 2007). If not found in an FIS, the technical support data notebook (TSDN) can be reviewed to determine how the astronomic tide is calibrated for the modeling process. In order to design for the highest possible flood elevation associated with a surge event, it is proposed that the SWEL accounts for the highest predicted tide for a single day; known as Mean Higher High Water (MHHW). To account for high tides it is necessary to use the great diurnal tide range, GT, or the mean range of tide, MN, of the nearest tidal station from tidesandcurrents.noaa.gov. Step 2.4.3, Account for Tides, will use only one of the following options below: for a MSL SWEL step 2.4.3.1 is recommended, for a MHW SWEL step 2.4.3.2 is recommended, and for a MHHW SWEL it is recommended to proceed to step 2.4.4.

2.4.3.1 Mean Sea Level (MSL)

If a SWEL is only referenced to MSL (no tides) it is recommended to add the difference between MHHW and MSL to the SWEL to provide a more effective SWEL. To adjust from

MSL to MHHW, Equation 2.4 can be used. The GT is the difference between MHHW and MLLW for both semidiurnal (mixed) cycles and diurnal cycles.

2.4.3.2 Mean High Water (MHW)

In the context of a semidiurnal (mixed) tide cycle, MHW is the average of the lower of two daily high tides observed over a tidal epoch. A diurnal cycle is not referenced to MHW, its high tide is known as MHHW. If a SWEL is referenced to MHW, it is recommended to be changed to reference MHHW to be considered a best practice. In order to adjust a SWEL from MHW to MHHW, Equation 2.5 can be used to arrive at the desired elevation.

At the conclusion of step 2.4.3, the result $SWEL_{MHHW}$, comes from either step 2.4.3.1 or 2.4.3.2 and should be carried through the remainder of the methodology. At the end of this step $SWEL_{MHHW}$ is still an elevation. The goal of step 2.4.3 is to have all SWEL's referenced to MHHW in order to achieve the best-recommended practice.

2.4.4 Determine Lowest Eroded Ground Elevation (GS) and Erosion Effects

In the CCM there is a methodology to determine the lowest eroded ground elevation at the seaward row of columns/ piles. The lowest eroded ground elevation is necessary if attempting to determine the maximum wave crest of depth-limited breaking waves. GS determination is a four-step process as explained by methods in the CCM.

2.4.4.1 Lower Existing Ground Elevation by Amount of Subsidence Over Life of Structure

To lower the existing ground elevation by the amount of subsidence, it is recommended to subtract the product of the documented rate of subsidence and the useful life of the structure from the existing ground elevation; CCM pg 8-8 (FEMA 2011). See Equation 2.6.

2.4.4.2 Landward Movement of Sand Dune

Determine how far the seaward toe of the dune translates horizontally toward the building: CCM pg 8-14 (FEMA 2011). Equation 2.7 may be used to find $E_{LANDWARD.}$

2.4.4.3 Eroded Ground Elevation at Seaward Row of Piles

Finding the amount the ground will drop at the seaward row of piles considering the slope of the eroded dune is recommended because it affects the overall SWEL and wave effects; view CCM pg. 8-14 (FEMA 2011). Dune erosion characteristics are covered in chapter 3 and 8 of the CCM. Equation 2.8 can be used to find 'Drop'.

2.4.4.4 Determine GS

The eroded ground elevation can be found using the existing ground elevation and the calculated drop from step 2.4.4.3 above; see CCM pg 8-14. Equation 2.9 can be used to find GS.

2.4.5 Find the Design Stillwater Depth, d_s.

The design stillwater depth is the difference in $SWEL_{MHHW}$, found in step 2.4.3, and the GS, found in step 2.4.4.4. This difference as presented in the CCM is defined as the final design stillwater *depth*, $d_{S,}$ before wave calculations are included. The proposed methodology refers to this difference as $d_{proposed}$ given there are differences in the proposed methodology and the CCM. It is important to note that this step is necessary to define the flood *depth* rather than the *elevation* in order to determine the maximum depth-limited wave crest in step 2.4.6. At the beginning of this step, 2.4.5, $SWEL_{MHHW}$ and GS are both elevations, but the difference between the two, $d_{proposed}$, in Equation 2.10 is a *depth* to be used in step 2.4.6.

2.4.6 Maximum Wave Crest

Using $d_{proposed}$ from step 2.4.5, the maximum wave crest can be found using techniques in the CCM (proposed Equation 2.11). The result is the design flood protection *depth* from the proposed method, indicated by the variable '*proposed* d_{fp}' or 'd_{fp}' for the CCM (FEMA 2011).

2.4.7 Design Flood Elevation

The result of step 2.4.6, *proposed* d_{fp}, is a flood *depth*, which is recommended to be converted back into an elevation in order to get an approved certificate of elevation. Using the

lowest eroded ground elevation, GS, from the method provided in the CCM (step 2.4.4.4), add to *Proposed* d_{fp} from step 2.4.6, to get an elevation referenced to NAVD 88 (Equation 2.12). It is recommended that all BFEs and DFEs are rounded to the nearest whole foot as are done on FIRMS. At the completion of step 2.4.7 the result is the proposed DFE: $DFE_{PROPOSED}$.

2.5 Summary of Code, Code-Plus and Recommended Practices

Table 2.2 summarizes ASCE, IBC & IRC code requirements, CCM best practices, recommended practices to close gaps within the CCM recommendations, and the input required for each computation. The final row of the table provides a DFE for code requirements and the recommended practices, and provides a design flood protection depth (d_{fp}) for the CCM practices. Slight alteration to the d_{fp} by adding to the lowest expected ground elevation will yield a DFE for the CCM. Code requirements are summarized in column three in addition to the freeboard amounts associated with each ASCE category and the respective LHSM orientation to the direction of wave approach. Column four describes the practices provided by the CCM. Column five combines CCM recommendations with proposed practices to address the gaps discovered within the CCM. The end result of column five is the proposed DFE.

2.6 Case Study

A case study for St. Johns County, Florida, is used to demonstrate the differences among SWEL elevations from the three methods summarized in Table 2.2. The case study provides all of the necessary information and characteristics a practitioner would need to follow the proposed methodology presented in Section 2.4. In addition, calculations are shown in the case study. The differences in the elevation requirements for code, CCM practices, and the proposed method are show in Figure 2.4 and discussed in detail. DFEs for the case study are rounded to the nearest foot as done in practice and for BFE values specified on FIRMs.

Table 2.2 Code, Best Practices, and Recommended Practices Summary

Variables	Input Data	IBC*, IRC*, and ASCE [1]	Coastal Construction Manual (CCM)	Recommended Best Practices
Vertical Datum	• Vertical datum for SWEL • Difference in vertical datum	Specified on community's FIRM	Specified on community's FIRM	1. Before continuing, make sure all elevations reference NAVD 88.
Global (Eustatic) Sea Level Rise (GSLR)	• Published GSLR rate • Building lifetime • SWEL • Model date of effective SWEL • Date of construction	--	Add the product of the GSLR rate and the building lifetime to the effective SWEL. [CCM Fig 8-3]. This results in E_{SW}.	2. Include GSLR from modeling date of effective SWEL through date of Construction. This will increase the SWEL 3. Include CCM direction.
Tides	• Are tides included in the FIS modeling? • If so, to which tidal datum is it referenced?	--	--	4. Include the highest tide (MHHW) with DFE. This increases the SWEL (ds for CCM). Resulting in $d_{PROPOSED}$: best practice SWEL depth.
Subsidence	• Published rate of subsidence • Building lifetime • Existing ground elevation	--	Lower the existing ground elevation by the product of the subsidence rate times the building lifetime. [CCM Fig 8-3]	5. Follow CCM recommendations
Erosion (landward)[4]	• Long-term erosion rate • Building lifetime • Historical landward shoreline	--	Measure landward from historical shoreline by the product of the erosion rate and the building lifetime. [CCM Fig 8-3]	6. Follow CCM recommendations
Lowest expected ground (due to erosion)	• Expected landward shoreline • Eroded dune profile from storm erosion model or stable bluff profile using available guidance and data	--	Using the most landward expected shoreline, calculate an eroded dune profile. Lower the ground elevation calculated from subsidence by eroded dune amount at the seaward row of piles, The result from steps 2-4 is GS.	7. Follow CCM recommendations
Maximum Wave Crest Calculation	• For IRC, IBC, ASCE: • FIS • For CCM: • d_s • $0.78d_s$= max breaking wave ht • 0.7(max breaking wave ht)	Included in BFE calculation	Calculate: $d_s = E_{SW} - GS$ (EQ 8.1 CCM). E_{SW} is calculated in step 2, and GS is derived in steps 2-4. Multiply d_s by 0.78, resulting in the breaking wave height; then multiply this height by 0.7 and add back to the d_s giving you the d_{fp}	8. Multiply Best Practices stillwater depth, $d_{PROPOSED}$, by 0.78,= breaking wave height (H_b); then multiply by 0.70 and add to $d_{PROPOSED}$ = *Proposed* d_{fp}: $((d_{PROPOSED} * 0.78)*0.7) + d_{PROPOSED}$ = *Proposed* d_{fp}
Bottom of LHSM Elevation Requirements (IRC, IBC, ASCE, CCM) OR, LHSM depth (CCM): High Hazard Areas- V-Zones	• For IRC, IBC, ASCE: • BFE • Orientation of LHSM • Community adopted freeboard • d_{fp}	Category I structure: DFE [2, 3] Category II Structure: higher of DFE [2] or of BFE+ 1' [2, 3] Category III & IV Structures: Higher of BFE + 1' or DFE[2]; or Higher of BFE + 2' or DFE[3]	Use the design flood protection depth, d_{fp}, calculated above in step 5. OR Add d_p back to GS for elevation referenced to original datum of FIS (DFE).- not an official CCM recommendation	9. Using *Proposed* d_{fp}, add GS from step 6 (calculated in step 4 of CCM method) to arrive at LHSM elevation referenced to NAVD 88. *Proposed* d_{fp} + GS = $DFE_{PROPOSED}$

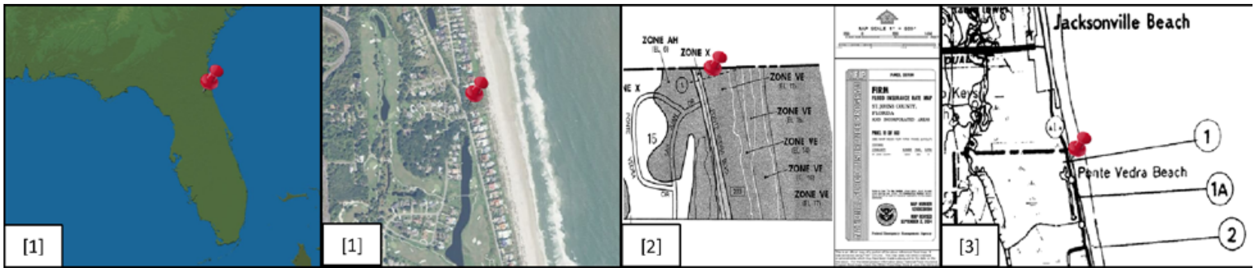
* IBC 1612.4 & IRC R322.1 Floodway construction must be in accordance with ASCE 24 (ICC 2009; ICC 2011).

1- Use in conjunction with Table 4-1 of ASCE 24.

2- LHSM is parallel to wave approach, where LHSM is ≤ 20 degrees from the direction of approach (ASCE 2005; ICC 2009).

3- LHSM is perpendicular to wave approach, where LHSM > 20 degrees from direction of approach (ASCE 2005; ICC 2009).

4- It is recommended that a minimum rate of 1.0 ft/year is used unless durable shoreprotection or erosion-resistant soil is present (FEMA 2011)



Transect 1 [2,3] SWEL = 11.3 FT NGVD29 [3]; BFE @ structure [2] = 13' NGVD29; Ground Elev. @ structure: 6.09' NGVD 29= 5.0' NAVD 88 Lat: 30.253711 Long: -81.381118 [4] Modeling Date = March 1999 [3]; Date of Construction: 9/3/2012 Community Freeboard = +1 FT [5]	Category II Residential Structure; LHSM Perpendicular to wave app. Useful life: 60 years (9/3/2072) Subsidence: 0.02 inches/yr [6] Projected GSLR: 0.03 ft/ yr [7, 8] Historic GSLR: 0.08 in/yr [6] Eroded Dune Prof (v:h): 1:50 [9] GT: 4.95 FT [10] Landward erosion: 1 ft/yr [11]	TABLE 9 - TRANSECT DESCRIPTIONS Transect: 1 Location: Ponte Vedra Beach, Atlantic coastline, FDEP Monument R-1 [3]	Elevation (feet NGVD29) 100-Year Stillwater[1]: 11.3 Maximum 100-Year Wave Crest[2]: 17
Input	ASCE, IBC, IRC	Coastal Construction Manual	Recommended Practices
SWEL	11.3 feet NGVD 29	11.3 feet NGVD 29	11.3 feet NGVD 29
Vertical Datum (Eq. 2.1)	--	--	1. NAVD 88 Output: 10.21 feet
Global (Eustatic) Sea Level Rise (Eq 2.2 & Eq 2.3)	--	E_{SW} = (0.03)*(60) + 11.3 = 13.1 feet	2. (0.03)*(13. 5 yr) + 10.21 = 10.62 3. 10.62 + (0.03*60) = 12.42 feet NAVD88
Tides (Eq 2.4, OR Eq 2.5 if necessary)	--	--	4. (4.95)/(2) + 12.42' = 14.90' NAVD 88
Subsidence (Eq 2.6)	--	Ground Drop = 6.09' -(0.02/12)*60 = 5.99' NGVD 29 : new ground elev	5. Ground Drop = 5'- (0.02/12)*60 = 4.9' NAVD 88 : new ground elev
Landward Erosion (Eq 2.7)	--	(1.0)*(60) = 60 foot setback from shoreline (landward movement)	6. (1.0)*(60) = 60 foot setback from shoreline (landward)
Lowest Expected Ground Elevation: due to erosion (Eq 2.8 & Eq 2.9)	--	(60)*(1/50) = 1.2 feet GS = 5.99' – 1.2' = 4.79' NGVD 29	7. (60)*(1/50) = 1.2 feet GS = 4.9' – 1.2' = 3.7' NAVD 88
Maximum Wave Crest Elevation (Eq 2.10 & Eq 2.11)	--	d_s 13.1 – 4.79 = 8.31'= 8.3' (8.3'*0.78*0.7)+8.3= 12.83'	8. $d_{Proposed}$ 14.90 – 3.7 =11.2' (11.2*0.78*0.7)+11.2= 17.32'
Bottom of LHSM Elevation Requirement (Eq 2.12)	BFE = 13 +1 = 14' NGVD 29 OR DFE = 13 + 1= 14' NGVD 29 OR DFE = 12+1 = 13' NAVD 88	D_{fp} =12.83' (depth above ground) OR DFE= (12.83'+4.79') = 17.62 18' NGVD 29 OR 15.95' = 16' NAVD 88	9. DFE= 17.32' + 3.7' = 21.02 = 21' NAVD 88

[1] (USDA 2012) [2] (FEMA 2012) [3] (FEMA 2011) [4] (iTouchMap.com 2012) [5] (Department of Public Works 2001) [6] (Postal et al. 2010) [7] (Rahmstorf 2007) [8] (DeMarco et al. 2012) [9] (FEMA 2011) [10] (NOAA and NOS 2012) [11] (Foster et al. 2000)

Figure 2.4 Case Study – St. Johns County, Florida

The case study produces the following results: ASCE DFE = 13' NAVD 88 (14' NGVD 29); CCM DFE= 16' NAVD 88 (18' NGVD 29); and the proposed method DFE = 21' NAVD 88. Most notably, the DFE for the proposed method is 61.5% (8.0') greater than the comparable ASCE NAVD 88 required standard. When referencing NAVD 88, the CCM practices are 23%

(3.0’) higher than the required ASCE standard and the proposed methods remain 31% (5’) higher than the best available current practices.

FEMA’s recommendation to multiply the 100-year SWEL by 1.25 to arrive at an approximate 500-year SWEL yields a 20’ NAVD 88 DFE (22’ NGVD 29) at the location of the case study (FEMA 2002). The approximated NAVD 88 500-yr DFE from FEMA is 5% (1’) less than the 100-year DFE resulting from the proposed method. In addition, the St. Johns County FIS (from the case study) provides a 500-year SWEL, but no 500-year wave crest (FEMA 2011). Using the same methods in the FIS and case study to determine the 100-year wave crest, the approximate 500-year wave crest based on the given 500-year SWEL, 12.2’ NGVD 29 (FEMA 2011), is equal to 18.86’ NGVD 29 = 17.17’ NAVD 88; a 19’ and 17’ DFE, respectively. The proposed method 21’ DFE that considers future conditions is still 23.5% (4’) higher than the approximated 500-year 17’ NAVD 88 DFE using data from the FIS.

Although the proposed method DFE provided in the case study is already higher than the other options, it is important to note in the first step, datum conversion, that there is actually a negative conversion. The shift from NGVD 29 to NAVD 88 actually reduced the SWEL by 1.09 feet, though this is not always the case. In Grand Isle, Louisiana there is a positive change of 0.14 feet between the vertical datum (USACE 2009). Therefore, if this same study were performed for Grand Isle, LA the SWEL would increase by +1.23 feet (1.09 + 0.14) for the datum conversion alone when compared with the given case study.

Given the statistical differences among the DFEs, it is important to note that including the future effects of geological processes can reduce the likelihood of flooding. Not including future effects of coastal geological processes can significantly increase the risk of flooding. For example, including just the effects of subsidence and GSLR over the expected useful life for the structure (60 years) at the site of the case study effectively reduces the FIRM DFE (14’ NGVD)

by 13.6% or 1.9 feet. The overall difference in accounting for all future effects, as the proposed method does, and not accounting for any future effects, like the current design standard, is a difference of 61.5% (8') in the DFE at the end of the structure's useful life.

2.7 Conclusion

The underlying premise of flood protection in coastal zones is to eliminate flood waters from infiltrating the structure altogether. Design Flood Elevations are the absolute minimum elevation for new construction, and are determined according to NFIP and ASCE standards. Current codes and standards (e.g. ASCE, IBC, and IRC) do not include methodologies for designing above the 100-year DFE, nor do they consider the effects of characteristics of the changing coastal environment. In contrast, the CCM includes the effects of future coastal processes, although it does not address all of the necessary aspects, including vertical datum conversions, global sea level rise that occurs prior to construction of the building and the effects of high tides during peak surge intervals.

This chapter presents a comprehensive methodology to determine the DFE if the design SWEL is known that addresses these remaining challenges and combine the current best available practices to provide a comprehensive guide for practitioners wishing to evaluate each source of flood risk. The methodology was developed based on an evaluation of coastal phenomena such as tides, sea level rise and subsidence. These phenomena are recognized as physical processes, and it is the authors' contention that they should be explicitly accounted for in the determination of flood elevation. The case study that implemented this methodology for St. Johns County, Florida, resulted in a DFE that was 61.5% (8') higher than a DFE calculated following current code requirements and 31% (5') higher than would be calculated using the CCM.

Code revision should better educate practitioners for flood considerations. Designers are aware of code requirements but they might not even be aware of best available practices such as the CCM. In addition, best practices should be incorporated into code requirements with modifications to those that neglect certain aspects. To account for subsidence, code revision should provide subsidence rate maps, including developing maps for areas that don't yet have one. To incorporate MHHW, code revision should provide information about including tides given that tidal data is readily available but not addressed. Tides should be accounted for in design, and more explicit information about the consideration of tides should be included in FIS. Code and best practices neither explicitly state nor provide any guidance to the designer for converting between vertical datums. Code should provide guidance in converting between vertical datum including recommending such conversion programs in addition to highlighting the importance of referencing the correct elevation to the correct vertical datum.

There is not adequate information available for practitioners about flood practices and requirements. Practitioners are restricted to elevating to the 100-year BFE with occasional 500-year SWEL guidance. Only the 100-year BFE is a code requirement, but even if provided with a 500-year SWEL there is no intermediate guidance. In addition, it should be recognized that the 100-year BFE is only a *minimum* design requirement, and not a best practice. It seems that most homeowners want to only build to the minimum required elevation (the BFE), yet they should be better informed by developers of the risk associated with designing only to the absolute minimum requirements.

This chapter is not suggesting the presented methods become requirements, but it does focus on estimating the actual 100-year BFE which considers future effects of the coastal processes during the lifetime of the structure. It should also be noted that comprehensive methods can both over-account and under-account for future effects. Policy and insurance

related issues should be separated from the requirements of engineering design. Engineering design should be focused on protecting human-life and structures, not to determine insurance rates. Insurance rates may still be tied to elevation, but it should reference the best available practice; elevate below the best practice and pay a higher rate, elevate at or above the best practice elevation and pay a lower premium.

2.8 Acknowledgements

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CHAPTER 3: ESTIMATING COASTAL FLOOD ELEVATIONS FOR RETURN PERIODS EXCEEDING CURRENT NFIP GUIDANCE USING EXISTING FIS

3.1 Introduction

Determination of the design flood elevation (DFE) for buildings constructed in the floodplain is currently tied to policy requirements of the National Insurance Flood Program (NFIP) 100-year flood. Additional protection is provided by freeboard, which is intended to compensate for factors that can result in greater flood depths (FEMA 2011). Freeboard, however, is an arbitrary value that is expected to compensate for multiple known and unknown factors. In applying freeboard, protection beyond the 100-year event is provided, but there is no indication of the failure probability that results from the application of freeboard. Additionally, for the practitioner wishing to estimate DFEs for failure probabilities beyond the 100-year event, information is very limited. Thus, those wanting to plan for a higher level of risk may struggle to find methods that can even estimate the DFE for these events. There is therefore a need to develop a methodology to estimate longer return period flood elevations using information readily available to practitioners.

Current codes, standards, and even best practices are based on the 100-year flood for all classifications of structures. This indicates a lack of understanding of the risk associated with designing to the 100-year flood. Within the expected 60-year lifespan of a typical residential structure, defined as a risk category II in ASCE 24 (ASCE 2010), the probabilities of the 100-, 500- and 700-year events occurring at least once are 45%, 11%, and 8% respectively (Friedland and Gall 2012). Furthermore, the 100-year flood has a 26% (1in 4) change of occurring over the life of a 30-year mortgage (FEMA and Federal Insurance and Mitigation Administration 2002). This level of risk (1% annual probability of failure) is much higher than the acceptable risks of failure for other hazards. According to the ASCE 7 commentary Table C.1.3.1a, the highest

acceptable annual probability of failure for load conditions (excluding earthquake hazards) is 0.0125% (ASCE 2010). This discrepancy in acceptable risk is especially critical in coastal flood areas, where waves and hydrodynamic forces can easily destroy buildings when the design flood elevation is exceeded. The acceptable level of risk for flooding should be lower by providing guidance on how to determine flood elevations of longer return periods.

FEMA provides flood insurance studies (FIS), which detail the methods and data used to map the 100-year base flood elevation (BFE) on flood insurance rate maps (FIRMs). At a minimum, the FIS provides a 100-year stillwater elevation (SWEL) which serves as the basis for delineating BFE values on a FIRM. The FIS may also provide the 10- and 50-year SWELs, with newer FIS providing a 500-year SWEL. Although more communities are providing 500-year SWELs, the corresponding annual probability of exceedance for the 500-year event is 0.2%, still significantly higher than the target acceptable limits. Therefore, additional guidance is needed for practitioners wishing to estimate flood elevations for the more commonly accepted risk of failure values. This chapter will analyze FIS SWEL values to estimate SWELs for longer return periods.

Various statistical distributions have been used to fit extreme values in hydrology. These methods include but are not limited to log normal, Gumbel, Beta-P, log-log or Huff-Angel, and semi-log or SRCC (Angel et al. 1992; Needham 2010). Huff-Angel is a log-log regression technique that is used to estimate rainfall frequencies and extremes for the Midwestern United States (Angel et al. 1992), and the Southern Regional Climate Center (SRCC) regression method is log-liner and has been implemented to estimate rainfall frequencies and extremes for the south-central states (Faiers et al. 1997). The ‘fitness’ of a method to a dataset can be determined using the Kolmogorov-Smirnov (KS) statistic, with a lower KS value representing a better fit (Keim and Faiers 2000; Needham 2010). The values for extreme events produced by Huff-

Angel and SRCC have been shown to fall within the lower extreme of Gumbel (Wilks and Cember 1993; Needham 2010) and the upper extreme of Beta-P (Needham 2010). Therefore, Huff-Angel and SRCC will be used to fit regression equations to SWEL data obtained from FIS for select communities along the Atlantic and Gulf Coasts.

3.1.1 Background on NFIP and the 100-year flood

In 1965, Congress directed the Department of Housing and Urban Development (HUD) to conduct a comprehensive study of flood insurance following significant losses from Hurricane Betsy (Power and Shows 1979). Marion Clawson, then secretary of HUD, completed the study in 1966 that recommended a federally sponsored flood insurance program (Power and Shows 1979). The National Flood Insurance Act of 1968 (NFIA) followed, which established the NFIP with the purpose of (Power and Shows 1979):

- Better indemnifying individuals for flood losses through insurance (FEMA and Federal Insurance and Mitigation Administration 2002);
- Reducing future damages through state and community floodplain management regulations (FEMA and Federal Insurance and Mitigation Administration 2002); and
- Reducing federal expenditures for disaster assistance flood control (FEMA and Federal Insurance and Mitigation Administration 2002).

The National Flood Insurance Act of 1968 explicitly intended for the NFIP to be "a program of flood insurance with large-scale participation of the Federal Government and carried out to the maximum extent practicable by the private industry" (Office of the General Counsel 1997, 1).

When the National Flood Insurance Program (NFIP) was mandated to map all the floodplains in the U.S., it became necessary to determine a standard "size" event in order to assess and manage flood risk for determination of insurance rates, thus treating all communities

equally (FEMA and Federal Insurance and Mitigation Administration 2002). Approximately 50 researchers in the field of floodplain management were identified and invited to participate in a floodplain management guidelines seminar at the University of Chicago that was held December 16-18, 1968 (Sheaffer 2004). The 100-year BFE was established at the recommendation of these 50 research experts who were convened by the U.S. Department of Housing and Urban Development (HUD) to determine a standard to be used as the "basis for risk assessment, insurance rating, and floodplain management for the NFIP" (FEMA and Federal Insurance and Mitigation Administration 2002, 5). The 100-year flood was a compromise between a 50-year or lesser standard and a 500-year flood standard (Krimm 2004). Thus the 100-year flood standard, or the flood with a 1%-annual-chance of occurrence, was adopted by the Federal Insurance Administration (FIA) and was used to identify floodplains (Krimm 2004).

In the 1960's, both the USACE and the Tennessee Valley Authority (TVA) agreed upon the 100-year flood as a uniform standard for flood protection (Robinson 2004). On September 10, 1971, the NFIP specifically tied the regulatory requirements of the program to the 100-year flood standard (Robinson 2004). In May 1972, the U.S. Water Resources Council issued final guidelines for evaluating flood hazards that recommended agencies use the 100-year flood as the "basic flood" (Robinson 2004, 3). The 1973 enactment of PL 93-234 (1972 Act, as amended) affirmed the use of the 1% or greater chance of flooding (100-year flood) as the "base flood standard" (Reilly 2004, 14). In 1974 amendments to the NFIA, the 100-year flood was specifically mentioned in the NFIP legislation for the first time (Robinson 2004), and on October 26, 1976, the term "base flood" and "base flood elevation" were introduced to phase out the misleading term of the "100-year flood" (Robinson 2004, 5).

On May 24, 1977 President Jimmy Carter issued Executive Order (EO) 11988-*Floodplain Management*, which specifically defines the floodplain as the base flood in terms of

the 100-year flood (Reilly 2004; Robinson 2004). The sufficiency of the 100-year flood was reviewed in 1973 by the Senate Committee on Banking, Housing, and Urban Affairs, and again in 1981 by the Office of Management and Budget (OMB) as part of the President's 1981 Task Force on Regulatory Relief. In both of these reviews, the 1-percent-annnual-chance standard was deemed "reasonable and consistent with national objectives in reducing flood losses" (FEMA and Federal Insurance and Mitigation Administration 2002, 5). A 1983 report by the Presidential Task Force on Regulatory Relief concluded that no better alternatives to the standard were available and that there was no justification for the expense of converting to another standard (Foundation 2004).

Congress passed the Flood Disaster Protection Act of 1973, which provided strong incentives for community and individual participation in the flood insurance program by introducing broad financial penalties for failure to participate (Power and Shows 1979). Section 102(a) and 102(b) of this act were intended to prevent lending by savings and loan institutions if an identified flood-prone community did not participate in the flood insurance program (Power and Shows 1979). "Section 1315 of the 1968 Act prohibits FEMA from providing flood insurance unless the community adopts and enforces floodplain management regulations that meet or exceed the floodplain management criteria established in accordance with section 1361(c) of the Act" (FEMA and Federal Insurance and Mitigation Administration 2002, 12). Changes made to the program in 1969 and 1973 dramatically increased participation, increasing the number of participating communities from four in 1969 to 15,898 in 1978, leaving only 2,928 (as of 1978) officially identified flood-prone communities that had yet to join (Power and Shows 1979).

3.1.2 Probability of Exceedance

The probability of an annual maxima-specific event occurring during the lifetime of a building is known as the probability of exceedance. Table 3.1 below demonstrates the probability of exceedance for mean recurrence interval (MRI) events compared to the expected useful life of a building. A recurrence interval is the reciprocal of probability (Markowitz 1971).

Table 3.1 Probability of Exceedance for Events During Expected Useful Life (Friedland and Gall 2012)

Expected useful life (years)	Probability of specified event occurring during expected useful life			
	100-year event[a]	500-year event	700-year event[b]	1,700-year event[b]
Normal life (at least 60 years)	45%	11%	8%	3%
Long life (60–120 years)[c]	60%	16%	12%	5%

[a]Current design standard for flood hazards (+ freeboard).

[b]Current design standard for wind hazards (1,700-year event for critical facilities, 700-year event for noncritical facilities).

[c]90-year useful life used in probability calculations.

The probability of exceedance may be calculated using Equations 3.1 and 3.2:

$$P_{not} = (1 - \frac{1}{xx})^n \quad (3.1)$$

$$P_{occur} = 1 - P_{not} \quad (3.2)$$

where:

P_{not} = the probability of an event not occurring in 'n' years,

xx = the return period of the event in years,

n = number of years the event does not occur, and

P_{occur} = the probability of the event occurring at least one time in 'n' years.

3.1.3 Return Periods and Annual Maxima

When using the term '500-year flood' or '100-year flood', it is easy to perceive this event occurring only once every 500 years or once every 100 years, when in actuality, the flood has a

0.2% chance or 1.0% chance of occurring annually, respectively (Philippi 1994). Therefore, an October 26, 1976 Final Rule introduced the terms "base flood" and "base flood elevation" to phase out the misleading term of the "100-year flood" (Robinson 2004, 5).

New FEMA FIS generally include flood elevations in four intervals: 10-percent-annual chance (10-year flood), 2%-annual-chance (50-year flood), 1%-annual-chance (100-year flood or BFE), and 0.2%-annual-chance (500-year flood). The flood elevation value associated with these return periods for coastal areas are typically only stillwater elevations (SWELs), and not the BFE, which includes wave effects. The methodology for determining SWELs associated with the return period is specified in the FIS in the Coastal Analysis subsection under the Engineering Methods section (FEMA 2000). From the SWEL, the BFE can then be calculated and reported along with a 100-year maximum wave crest, the methodology for which can be found in Wave Height Analyses subsection under the Engineering Methods section.

Factors affecting the calculation of the SWEL for various return periods are the computer model used and the five storm surge parameters: storm intensity (central pressure depression), radius of maximum winds from storm center, forward speed of storm, direction of approaching storm, and frequency of the storm occurrence (FEMA 2000). The SWEL directly affects the maximum crest of a breaking wave. Most wave calculations calculate the maximum breaking wave height as 78% of the SWEL, and the maximum wave crest as 70% of the maximum breaking wave height above the SWEL, which can also be calculated as 1.546 times the SWEL (FEMA 2003; FEMA 2011).

Additional factors affecting a storm surge elevation, which in turn affect the flood return period, are subsidence, sea level rise, tides, erosion, and datum conversion. These are discussed in Chapter 2 and covered by the CCM (FEMA 2011; FEMA 2011). Although these factors

affect the overall BFE, their long-term consequences are not currently reflected in FIS SWEL and BFE determinations.

3.1.4 Current Coastal Flood Design Practices

ASCE 7-10, *Minimum Design Loads for Buildings and Other Structures* and ASCE 24-05, *Flood Resistant Design and Construction*, break down structures into four classifications based on their hazard to human life; ASCE 7 and 24 categories are generally the same but not exactly. The generalized categories are as follows:

- *Category I:* "Buildings and other structures that represent a low risk to human life in the event of failure" (ASCE 2005; ASCE 2010, 2).
- *Category II:* "All buildings and other structures except those listed in Risk Categories I, III, and IV" (residential structures are included here) (ASCE 2005; ASCE 2010, 2).
- *Category III:* "Buildings and other structures, the failure of which could pose a substantial risk to human life; buildings and other structures, not listed in risk category IV, with potential to cause a substantial economic impact and/or mass disruption of day-to-day civilian life in the event of failure" (ASCE 2005; ASCE 2010, 2).
- *Category IV:* "Buildings and other structures designated as essential facilities; buildings and other structures, the failure of which could pose a substantial hazard to the community" (ASCE 2005; ASCE 2010, 2).

Elevation requirements for the lowest horizontal structural member (LHSM) for Coastal High Hazard Zones are summarized in Table 3.2, where BFE is the base flood elevation including wave height that has a 1% chance of being equaled or exceeded in any given year, the minimum NFIP requirement; and DFE is the design flood elevation including wave height on a community's flood hazard map where communities have elected to exceed NFIP requirements, otherwise the DFE defaults to the BFE (ASCE 2005). Identification of a flood elevation with a

desired MRI is only the initial step; other factors include tides, datum conversion, erosion, and global sea level rise. Chapter 2 provides further guidance on design flood elevation determination.

Table 3.2 Summary of ASCE 24 Elevation Requirements for Lowest Horizontal Structural Member (ASCE 2005)

Structure Category	Minimum Elevation of LHSM	
	Parallel to Wave Approach[1]	Perpendicular to Wave Approach[2]
I	DFE	DFE
II	DFE	Greater of BFE + 1 ft or DFE
III	Greater of BFE + 1 ft or DFE	Greater of BFE + 2 ft or DFE
IV	Greater of BFE + 1 ft or DFE	Greater of BFE + 2 ft or DFE

[1]LHSM is oriented parallel to the direction of the wave approach, where parallel shall mean less than or equal to 20 degrees (0.35 radians) from the direction of approach (ASCE 2005).
[2]LHSM is oriented perpendicular to the direction of wave approach, where perpendicular shall mean greater than 20 degrees (0.35 radians) from the direction of approach (ASCE 2005).

As shown in Table 3.2, all structure categories are subject to the 100-year BFE with the addition of freeboard required either by ASCE or the community. Post-BFE methods include the addition of "freeboard", defined as extra elevation between the lowest horizontal structural member and the BFE, resulting in the DFE (FEMA 2011). Freeboard is an arbitrary value given that flood elevations for storms of selected recurrence intervals are calculated for each given locality, yet freeboard required by code is the same amount for the entire country providing an unknown level of protection. Furthermore, freeboard doesn't account for varying coastal bathymetry, which is the characteristics of beds or floors of bodies of water (NOAA 2013), affecting the magnitude of surge buildup and wave heights for a given locality and return period.

FEMA Flood Insurance Studies (FIS) provide 100-year stillwater elevations (SWEL) and BFEs for all communities participating in the NFIP, and provide 10-year, and 50-year SWELs and BFEs for most NFIP communities. Recently, FEMA has included a 500-year SWEL for

limited communities which can lead to an approximate calculation of a 500-year BFE wave crest, but most communities have no guidance for designing above the 100-year BFE. The only guidance for design beyond 100-year elevations is a rule of thumb provided by FEMA to approximate the 500-year stillwater elevation (SWEL) as 1.25 times the 100-year stillwater elevation (FEMA and NAHB Research Center 2010). In addition, FEMA has included shaded areas on Flood Insurance Rate Maps (FIRM) designating areas that are outside of the 500-year floodplain. Information and guidance to estimate flood levels beyond the 100-year BFE is scarce and much further investigation is warranted.

3.2 Methodology

As a means to estimate flood levels beyond the 100-year BFE, the methodology in this chapter uses SWEL values from the FIS of thirteen coastal communities along the Gulf and Atlantic Coasts. Huff-Angel and SRRC regression methods are first used to extrapolate a 500-year SWEL from the FIS provided 10-, 50-, and 100-year SWEL values. These values, hereafter known as 500-year projected values, are then compared to the actual 500-year FIS values to determine the more suitable regression method. Using the selected regression method, the 700- and 1700-year SWELs are estimated by extrapolating the 10-, 50-, 100-, and 500-year known FIS SWELs for each community.

3.2.1 Data Collection

Using FEMA's Map Service Center, sixteen community FIS were initially collected for the Gulf Coast and Atlantic Coast. FIS were collected ranging from South Padre Island, Texas to Seabrook, New Hampshire. Using a rough coastline distance, an average of 206 miles separates each FIS location with a minimum distance of approximately 90 miles between the two closest communities, and a maximum of approximately 306 miles separating the two farthest apart communities.

The 16 coastal communities were evaluated for integration into the analysis based on multiple characteristics:

- Communities directly located on the coast with beachfront development with elevations near sea-level, while avoiding anomalies.
- More recent SWEL modeling dates and effective FIS dates than adjacent communities.
- Communities with FIS SWELs associated with the four return intervals (10-year, 50-year, 100-year, and 500-year flood).

Unfortunately, not all sixteen FIS met the above criteria. Much of the Louisiana coastline was designated as an anomaly given the fragmented marsh-wetland present along the coast. Louisiana only provides two areas with beachfront development for this study: Cameron and Grand Isle. In addition, areas in the most north-eastern portion of the Atlantic Coastline offered minimal communities with the desired characteristics. In New Hampshire and Maine, the coastal landscape is dominated with coastal bluffs, resulting in minimal beachfront development subject to coastal flooding. Table 3.3 summarizes the selected coastal communities, organized from the most south-western location to the most north-eastern location. FIS that did not include all four return periods (Holly Beach, LA; Grand Isle, LA; and Naples, FL) were not further evaluated, leaving 13 FIS for the analysis. Appendix A includes a description of the origin of the transects used in the analysis.

3.2.2 Data Compilation

The SWEL data from the remaining 13 FIS were input into an Excel Spreadsheet. Some FIS SWELs for individual return periods were reported as a range. In this case, the larger of the two numbers was used given that a SWEL is the baseline for determining a BFE. Taking into account that the BFE is the minimum design requirement, this makes the larger of the SWELs desired. After all SWEL information was entered into Excel, the data were condensed to only

include transects with unique values, removing all repeated values. Some communities provided multiple transects with repetitive SWELs, which were reduced to a single transect to prevent any effects of weighting of the findings. For the 13 FIS, 117 unique transects remained. When multiple transect flood elevations were present for a community, these values were averaged to obtain a single transect representative of each community; these averages are shown in Table 3.4.

Table 3.3 Summary of Selected Flood Insurance Studies

State	County	SWEL Model Date	FIS Effective Date	All 4 Return Periods?
TX	Cameron	Oct. 1996	03/09/1999	Yes
TX	Galveston	July 2001	12/06/2002	Yes
LA	Cameron	Dec 1989	04/16/1991	No
LA	Jefferson	Aug 1991	03/23/1995	No
MS	Harrison	Sep. 2007	06/16/2009	Yes
FL	Bay	April 1998	06/02/2009	Yes
FL	Pinellas	Sep. 2002	08/18/2009	Yes
FL	Collier	11/30/1996	11/17/2005	No
FL	Martin	Aug 1997	10/04/2002	Yes
FL	St. Johns	March 1999	06/18/2011	Yes
SC	Charleston	Sep. 1989	11/17/2004	Yes
NC	Carteret	N/A	07/02/2004	Yes
VA	Virginia Beach	March 1993	05/04/2009	Yes
NJ	Ocean	Jan 1996	09/29/2006	Yes
MA	Dukes	March 1991	07/06/2010	Yes
NH	Rockingham	Dec 1983	05/17/2005	Yes

Table 3.4 Average FIS Transect Data by Recurrence Interval (ft)

State	County	10	50	100	500
TX	Cameron	7.5	10	10.6	11.7
TX	Galveston	6.5	11.2	13.6	15.8
MS	Harrison	5.9	14.9	17.8	23.4
FL	Bay	3.6	6.4	10.1	10.4
FL	Pinellas	4.5	7.8	9.4	12.4
FL	Martin	4.4	6.2	8.6	8.8
FL	St. Johns	4.8	7.7	11.1	11.7
SC	Charleston	9.3	11.4	12.1	13.6
NC	Carteret	4.3	7.7	11.2	12.5
VA	Virginia Beach	5.5	7.1	7.7	9.1
NJ	Ocean	5.9	7.5	8.5	11.5
MA	Dukes	4.2	5.7	6.8	9.1
NH	Rockingham	8.3	8.9	9.2	9.8

3.2.3 Estimated 500-Year SWEL Values

Using the resulting transect data, two scatter-plots for each individual community were created first using only the 10-, 50-, and 100-year SWEL. A trendline was fit to the three annual-chance flood quantiles: 10%, 2%, and 1% chance to determine how well the Huff-Angel and SRCC model extrapolated the 500-year SWEL in comparison to the given FIS 500-year SWEL. The trendline does not represent the exact data of the selected series, but it depicts trends of the input data or forecasts for future data (Microsoft 2010). This analysis serves a dual purpose in 1) providing a method to determine a 500-year SWEL for communities without such provided value and 2) to validate the methods used to determine a 700 and 1700 year SWEL.

Regression techniques are used to estimate quantile values using Huff-Angel (Angel et al. 1992) in the Midwestern United States, and SRCC techniques (Faiers et al. 1997) in the South-Central states. The Weibull plotting position is used for both Huff-Angel and SRCC methods (Faiers et al. 1997). For the Huff-Angel method, the x-axis which shows the return periods, and the Y-axis which shows the SWEL, are formatted as a log-log scale (Angel et al. 1992). The trendline for the given SWEL quantiles provided in the FIS are formatted as a power regression trend. For the SRCC method, a semi-log (log-linear) regression approach is used; the y-axis, return periods, remains formatted as linear and the x-axis is formatted as log (Faiers et al. 1997). The SRCC trendline is expressed as a logarithmic regression trend.

The R-squared value is also reported for each equation. The R-squared value, or the coefficient of determination, is known as the proportion of variability in a data set that is accounted for by the statistical model (Steel et al. 1960). R^2 generally represents the goodness of fit (Colin Cameron and Windmeijer 1997). In Excel R-squared represents how closely the estimated values for the trendline match the actual data and ranges from 0 to 1 (Microsoft 2010).

The trendline is most reliable as R^2 approaches or is 1 (Microsoft 2010). R^2 is calculated as (Henry 2001; Microsoft 2010):

$$R^2 = 1 - \frac{SSE}{SST} \qquad (3.3)$$

where

SSE is the sum of squares of residuals

SST is the total sum of squares.

Table 3.5 provides the projected 500-year SWEL with the actual FIS 500-year SWEL for the Huff-Angel and SRCC methods. SRCC 500-year SWEL projected values compared with the FIS 500-year SWEL range from -11.5% to 30.8% greater than the FIS 500-yr SWEL; differences from Huff-Angel vary from -4.5% to approximately 81.5% greater than the FIS 500-yr SWEL. The SRCC average is 8.79% greater than the FIS 500-yr SWEL; the Huff-Angel average is 33.94% greater than the FIS 500-yr SWEL. This analysis demonstrates that the SRCC method produces better extrapolation results than Huff-Angel for the 500-year return interval.

Table 3.5 Differences Between Projected 500-year SWEL and FIS SWEL

		SRCC		Huff-Angel	
State	County	Percent Greater than FIS[1]	Difference in Feet[1]	Percent Greater than FIS[1]	Difference in Feet[1]
TX	Cameron	0.111111	1.3	0.188034	2.2
TX	Galveston	0.044304	0.7	0.468354	7.4
MS	Harrison	0.125856	2.9	0.815068	19.0
FL	Bay	0.307692	3.2	0.826923	8.6
FL	Pinellas	0.024194	0.3	0.298387	3.7
FL	Martin	0.227273	2.0	0.443182	3.9
FL	St. Johns	0.219616	2.6	0.54371	6.4
SC	Charleston	0.035242	0.5	0.079295	1.1
NC	Carteret	0.218166	2.7	0.674978	8.4
VA	Virginia Beach	0.021978	0.2	0.087912	0.8
NJ	Ocean	-0.11497	-1.3	-0.04555	-0.5
MA	Dukes	-0.07692	-0.7	0.021978	0.2
NH	Rockingham	0.0	0.0	0.010204	0.1
	Avg	0.087965		0.339421	

[1] A positive difference indicates projected values are greater the FIS values and a negative difference indicates projected values are less than FIS values.

Two additional scatter-plots for each individual community were created using the 10-, 50-, 100-, and 500-year SWEL (Appendix B). A trendline was fit to the four annual-chance flood quantiles: 10%, 2%, 1%, and 0.2% chance. The 500-year SWEL on the trendline from the resulting regression equations was used to demonstrate the differences from the FIS 500-year SWEL. The results of this analysis also show that SRCC results in lower difference (Table 3.6).

Table 3.6 Differences Between the Regressed 500-year SWEL Values and the FIS 500-year SWEL

		SRCC		Huff-Angel	
State	County	Percent Greater than FIS	Difference in Feet	Percent Greater than FIS	Difference in Feet
TX	Cameron	0.025641	0.3	0.042735	0.5
TX	Galveston	0.044304	0.7	0.107595	1.7
MS	Harrison	0.035959	0.8	0.172945	4.0
FL	Bay	0.076923	0.8	0.173077	1.8
FL	Pinellas	0.008065	0.1	0.072581	0.9
FL	Martin	0.056818	0.5	0.102273	0.9
FL	St. Johns	0.049041	0.6	0.108742	1.3
SC	Charleston	0.013216	0.2	0.020558	0.3
NC	Carteret	0.057881	0.7	0.146037	1.8
VA	Virginia Beach	0	0.0	0.021978	0.2
NJ	Ocean	-0.0282	-0.3	-0.01085	-0.1
MA	Dukes	0.021978	0.2	0.010989	0.1
NH	Rockingham	0	0.0	0	0.0
	Avg	0.027817		0.074513	

The SRCC 500-year regression fit values range from -2.8% to 7.7% greater than the FIS 500-yr SWEL, and Huff-Angel ranges from approximately 1.1% to 17.3% greater than the FIS 500-year SWEL. The SRCC average difference among normalized is 2.8% of the 100-yr SWEL, and the Huff-Angel average normalized difference is 7.5% of the 100-year SWEL.

The above analysis was conducted to determine how well the regression equation obtained from using the 10-, 50-, 100- and 500-year values would match the known 500-year value from the FIS. Results indicate an accurate fit that can potentially yield dependable extrapolation values. The summarized statistics further verify that SRCC is superior to Huff-

Angel and validates the ability of the regression-extrapolation method to produce results consistent with known values.

3.2.4 Estimated 700- and 1700-Year SWEL Values

Using the trendlines obtained through regression of the 10-, 50-, 100- and 500-year values (Appendix B), the 700- and 1700-year SWEL values were extrapolated (Table 3.8). Huff-Angel has been shown to produce excessively large values with outlier data such as the 500-year storm, while the semi-log approach of the SRCC method reduced these large estimates for events of longer recurrence intervals (Faiers et al. 1997).

In addition, further research has shown the SRCC linear regression method to produce a line of best fit for return periods associated with extreme surge levels better than Huff-Angel, Gumbel, and Beta-P distribution methods (Needham 2010). Comparison of the four methods using the Kolmogorov-Smirnov (KS) statistic demonstrates that the SRCC method out-performs the Huff-Angel method for large magnitude events (Keim and Faiers 2000; Needham 2010). Therefore, based on previous research performed by others and the discovered results, the SRCC regression model is recommended for extrapolating elevations for selected recurrence intervals.

Furthermore, comparison of R^2 values for the Huff-Angel method and the SRCC method will generally identify the method best suited for the data set. Table 3.7 provides a summary of the R^2 values for each regression equation using the 10, 50, 100 and 500-year SWEL.

The SRCC regression method has a higher R^2 value for 11 of the 13 communities, implying a better fit for the data than Huff-Angel. All evaluations have indicated that SRCC is better able to fit the FIS data and is recommended for the extrapolation of FIS values.

Table 3.8 presents results of the averaged transect data and the extrapolated 700 and 1700 year SWEL values for the thirteen coastal communities using the SRCC method. These values are considered reliable estimates for each individual community. The 700 and 1700-year SWEL

values can be estimated with more confidence when a 500-year SWEL is provided because an average error of 9.26% difference exists between a projected 500-year SWEL when only given a 10-, 50-, and 100-year SWEL value and the known FIS 500-year SWEL.

Table 3.7 Summary of SRCC and Huff-Angel Regression Equations and R^2-Values

		SRCC		Huff-Angel	
State	County	Regression Equation	R^2 Value	Regression Equation	R^2 Value
TX	Cameron	Y=1.0673*LN(x)+5.4049	0.9473	$y=6.0753*x^{0.1128}$	0.9166
TX	Galveston	Y=2.4103*LN(x)+1.5104	0.9615	$y=4.2235*x^{0.2287}$	0.9126
MS	Harrison	Y=4.458*LN(x)-3.4991	0.9814	$y=3.1218*x^{0.3493}$	0.8979
FL	Bay	Y=1.8478*LN(x)-0.244	0.8527	$y=2.102*x^{0.283}$	0.8524
FL	Pinellas	Y=2.0282*LN(x)-0.1123	0.9987	$y=2.6496*x^{0.2594}$	0.9661
FL	Martin	Y=1.1959*LN(x)+1.9072	0.8851	$y=3.0508*x^{0.1862}$	0.8564
FL	St. Johns	Y=1.8309*LN(x)+0.8959	0.8889	$y=3.0039*x^{0.2359}$	0.8872
SC	Charleston	Y=1.0994*LN(x)+6.9232	0.9917	$y=7.6086*x^{0.0969}$	0.9768
NC	Carteret	Y=2.1905*LN(x)-0.4258	0.9237	$y=2.4638*x^{0.283}$	0.8947
VA	Virginia Beach	Y=0.9186*LN(x)+3.4381	0.9984	$y=4.1869*x^{0.1284}$	0.985
NJ	Ocean	Y=1.4504*LN(x)+2.167	0.9723	$y=3.8667*x^{0.1736}$	0.9967
MA	Dukes	Y=1.2627*LN(x)+1.0726	0.9854	$y=2.6541*x^{0.1994}$	0.9974
NH	Rockingham	Y=0.3849*LN(x)+7.4107	0.9995	$y=7.5341*x^{0.0426}$	0.9988
		Average	0.9528		0.9337

Table 3.8 Summary of Information used to Extrapolate 700 and 1700-year SWEL Values

				Average FIS Transect Flood Elevations by Recurrence Interval (ft)				Extrapolated Flood Depths by Recurrence Interval (ft)	
State	County	Equation	R^2	10	50	100	500	700	1700
TX	Cameron	y=1.0673*LN(x)+5.4049	0.9473	7.5	10	10.6	11.7	12.7	14.1
TX	Galveston	y=2.4103*LN(x)+1.5104	0.9615	6.5	11.2	13.6	15.8	17.3	19.4
MS	Harrison	y=4.458*LN(x)-3.4991	0.9814	5.9	14.9	17.8	23.4	25.7	29.7
FL	Bay	y=1.8478*LN(x)-0.244	0.8547	3.6	6.4	10.1	10.4	11.9	13.5
FL	Pinellas	y=2.0282*LN(x)-0.1123	0.9987	4.5	7.8	9.4	12.4	13.2	15
FL	Martin	y=1.1959*LN(x)+1.9072	0.8551	4.4	6.2	8.6	8.8	9.7	10.8
FL	St. Johns	y=1.8309*LN(x)+0.8959	0.8889	4.8	7.7	11.1	11.7	12.9	14.5
SC	Charleston	y=1.0994*LN(x)+6.9232	0.9917	9.3	11.4	12.1	13.6	14.1	15.1
NC	Carteret	y=2.1905*LN(x)-0.4258	0.9237	4.3	7.7	11.2	12.5	13.9	15.9
VA	Virginia Beach	y=0.9186*LN(x)+3.4381	0.9984	5.5	7.1	7.7	9.1	9.5	10.3
NJ	Ocean	y=1.4504*LN(x)+2.167	0.9723	5.9	7.5	8.5	11.5	11.7	13.0
MA	Dukes	y=1.2627*LN(x)+1.0726	0.9854	4.2	5.7	6.8	9.1	9.3	10.5
NH	Rockingham	y=0.3849*LN(x)+7.4107	0.9995	8.3	8.9	9.2	9.8	9.9	10.3

The data in Table 3.8 indicate an average increase in 500-year SWELs over 100-year SWELs of 1.17, rather than the recommended 1.25. This value ranges from 1.02 to 1.35. These community specific values should be used with a greater level of certainty than FEMA's 1.25 rule of thumb. For best results for a community not included in the analysis, a separate analysis of the data from the respective community's FIS should be executed. These values will be more reliable for community specific data, and can be used with a greater level of certainty.

3.2.5 Normalized Data

There may be circumstances where a generalized estimate of longer return period flood depths is desired and a community specific analysis is not warranted. This section describes normalization of all data presented in Table 3.8 to provide generalized guidance. The SWEL values are ambiguous when compared with other communities; therefore the data is normalized to assign the same weight to a known value. Given that the 100-year flood is the basis for code requirements, the data is normalized to the 100-year flood, making this value equal to '1' for all communities. By normalizing the data, elevations associated with the other return periods are expressed as percentage of the 100-year flood. With the normalized 100-year SWEL being equal for all communities, the percentages of a SWEL associated with alternative recurrence intervals allows for comparison across different communities. Reporting results of the normalized data allows for better comparison and understanding of the resulting data.

Normalized data for the 10-, 50-, 100-, 500-, 700-, and 1700-year SWELs (Table 3.8) were put into a graph formatted with normal linear axes. The data is no longer input as single scatter points; it is shown by a line representing the SWEL values for each individual community. All data lines should come to a point of intersection located at (100, 1), representing the point of normality for the data set. The graph is manipulated to shift this point of intersection to make the vertical value of 'one' be equal to zero on the horizontal axis. This sets the 100-year

SWEL, which is the national basis for flood design, equal to zero thus allowing the 500-, 700-, and 1700-year SWEL to be expressed as a percentage, shown by the y-axis values, of the known 100-year SWEL.

The range of the extrapolated normalized SWEL values needed to be bounded and reported with some level of confidence to be of use for practitioners. The mean of the normalized data for each of the six recurrence intervals was calculated and then used to determine the standard deviation for each recurrence interval. Through use of the standard deviation, a 95% confidence interval for the SWEL values was determined by taking +/- two standard deviations from the mean for the normalized data of each recurrence interval. Values above zero on the horizontal axis for recurrence intervals less than the 100-year adjusted axis and values below zero on the horizontal axis for recurrence intervals greater than 100-years are not possible; therefore they must be adjusted to only equal zero. This was done for the 50-year value plus two standard deviations, and the 500-year value minus two standard deviations. As shown in Figure 3.1, the result was a mean distribution of the normalized data, bounded by an upper and lower limit expressed as a percentage of the 100-year SWEL.

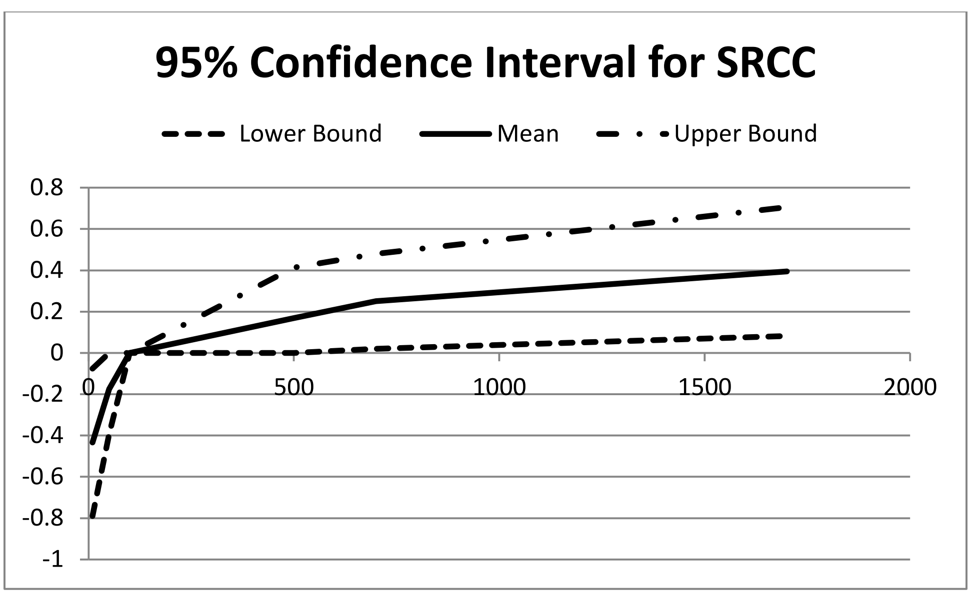

Figure 3.1 Extreme SWEL Values Normalized About the 100-year SWEL

Tabular results of Figure 3.1 are provided in Table 3.9. For the 95% confidence interval, the mean 700-year SWEL is 24.8% greater than the 100-year SWEL. The 700-year SWEL upper limit is 48% greater than the 100-year SWEL, and the 700-year lower bound is 1.5% greater than the 100-year SWEL. The 1700-year mean is 38.9% above the 100-year SWEL and is bounded on the upper limit by 70.8% of the 100-year SWEL, and bounded on the lower limit by 6.9% above the 100-year SWEL.

Table 3.9 Generalized 700- and 1700-Year Estimate Guidelines - 95% Confidence Interval

	700-Year	1700-Year
Lower Bound	1.5%	6.9%
Mean	24.8%	38.9%
Upper Bound	48.0%	70.8%

Values indicate increase over 100-year SWEL

3.3 Conclusion and Recommendations

This study utilized SWEL data for transects from FIS for thirteen coastal communities along the Atlantic and Gulf Coasts. A single transect of averaged SWEL values for each community was derived from existing FIS data for the 10-, 50-, 100-, and 500-year storm. Huff-Angel and SRCC regression methodologies were used to fit a trendline to the 10-, 50-, and 100-year SWEL to project a 500-year SWEL, which was compared with the actual 500-year SWEL given in the FIS. Results that showed differences in the projected value and actual value measured in feet, and as a percentage greater than the 100-year SWEL, indicated the SRCC method best extrapolates values for longer return periods. The SRCC method produced values closer to the actual 500-year SWEL in all but two of the thirteen communities, for which the SRCC underestimated the actual value. This application of the SRCC method can be used to estimate a 500-year SWEL for communities that only have information for the 10-, 50-, and 100-year SWEL.

The Huff-Angel and SRCC methods were also used to fit a new equation to the FIS provided 10-, 50-, 100-, and 500-yr SWEL. By estimating the 500-year SWEL from the new regression equation, the applicability of the SRCC and Huff-Angel methodologies was determined by comparing the regressed value with the actual FIS provided 500-year SWEL. Again, SRCC outperformed Huff-Angel on the differences in measurement of feet and the normalized values for eleven of the thirteen communities. Additionally, SRCC was determined a better fit on the basis of the R^2 value, known as the proportion of variability in the data set (Steel et al. 1960), for eleven of the thirteen communities.

Results of the analysis in conjunction with literature and previous research supports using the SRCC method to extrapolate extreme SWEL values from a regression equation fit to existing FIS data. The SRCC regression equation can be used to interpolate values located within the data set or to extrapolate values outside the data set. The SRCC method provides practitioners with a methodology for obtaining SWEL values associated with a variety of return period magnitudes. This conclusion allows for values of the 700- and 1700-year SWEL to be extrapolated on a statistical basis that can be used as risk-consistent with wind design codes. Results of the analysis indicate that 700- and 1700-year SWELs are on the average 25% and 39% higher than the 100-year SWEL, respectively. More importantly, this method can be used for communities where FIS report the 10-, 50-, 100-, and/or 500-year SWEL. This methodology can be used to develop estimates of longer return period SWELs, and integrated with the methodology presented in Chapter 2 to develop DFEs.

CHAPTER 4: CONCLUSIONS AND RECOMMENDATIONS

4.1 Introduction

4.1.1 Summary

The primary goal of this thesis was to improve the understanding of coastal flooding in the changing environment and to provide guidance to practitioners to consider increased mitigation and adaptation for buildings designed in the coastal environment. Three primary objectives were identified as a step toward accomplishing this goal: evaluate current code and code-plus guidance regarding building elevation in coastal high hazard areas; identify existing gaps in code and code-plus practices and to recommend practices to bridge identified gaps, and evaluate existing FIS and develop practitioner-oriented guidance for estimating longer return period flood elevations (e.g. 700 and 1700 years) from existing FIS. This chapter summarizes the way each objective improves the procedure used to arrive at a DFE and provides suggestions for future work on this topic.

4.1.1.1 Literature Review of Design Flood Elevation in Coastal High Hazard Areas

Code requirements and current best available practices were presented and explored in Chapter 2 with the goal of providing a comprehensive methodology summarizing current requirements and practices in a single location. Chapter 3 provided guidance for determining flood elevations for the 700 and 1700 year flood using existing FIS SWELs.

A literature review of current best practices provided by FEMA's Coastal Construction Manual (CCM) revealed gaps in their methodology that can be used to improve the procedure for DFE estimation. The CCM introduces concepts such as erosion, seal-level rise and subsidence that are not included in the elevations on flood insurance rate maps (FIRM) (FEMA 2011; FEMA 2011). An investigation of the best practices exposes gaps that should be improved to provide a comprehensive 'best' methodology. CCM neglects to account for the effects of high

tide effects, vertical datum transformations, and sea level rise that may occur prior to building a structure. In fact, the CCM provides an example of their suggested methods performed using only an elevation referenced to the outdated National Geodetic Vertical Datum of 1929 (NGVD 29) and fails to mention any sort of method or need to convert to the current North American Vertical Datum of 1988 (NAVD 88). The CCM recommends valid practices not included in the FIRM making process, yet gaps within the proposed methods exist.

The literature review revealed a misunderstanding of the '100-year flood', and that the base flood is only meant to have a one-percent chance of being exceeded annually (FEMA 2009). It should also be understood that the 100-year flood is the bare minimum for code requirements, yet it is the only code requirement with the exception of freeboard requirements. Flood codes are not consistent with code requirements for wind design and thus are subject to a higher level of risk. Because flood elevations are tied to insurance policy, the current policy affects code requirements. Thus, there is no reliable guidance available for a practitioner to design above the 100-yer flood elevation. The only current guidance available beyond the 100-year elevation is to approximate the 500-year flood elevation as 1.25 times the 100-year SWEL (FEMA and NAHB Research Center 2010). Recently, FIS have started to report the 500-year SWEL, but these remain below the ASCE 700 and 1700-year design wind speed requirements.

Many journal articles refer to determining extreme flood values, but historical flood elevation data is used which is not tied to current policy or code. These methods consist of plotting historical flood elevations and using an equation of best-fit to demonstrate which extreme value function best matches the historical data. The same method of fitting a regression equation to a dataset was used for the FIS SWEL data to then be extrapolated. The Huff-Angel and SRCC regression methods have been frequently used in hydrologic research when trying to determine extreme values.

Based on this literature review, it was determined that flood design code requirements are not risk consistent with wind code requirements; there are gaps to be improved upon in current best practice recommendations; and there is currently no recommended methodology to determine reliable flood elevations beyond the 100-year return period.

4.1.1.2 Analysis of Current Methodologies

A review of current methodologies for the flood design process reveals a lack of guidance for practitioners seeking risk-based elevation estimation. The analysis was performed from two separate viewpoints: 1) understanding the coastal environment's interaction with the built environment and their possible impacts, and 2) the ability to obtain scientifically based elevations beyond the 100-year BFE.

FEMA's FIS provide a 100-year SWEL and 100-year maximum wave crest elevation (BFE) at a minimum for coastal high hazard areas. Some FIS report a 10- (10% annual chance), 50- (2% annual chance), and 100-year (1% annual chance) SWEL, with some of the newer FIS providing a 500-year (0.2% annual chance) SWEL. Unfortunately, the 500-year SWEL is currently very rare and is not risk-consistent with other forms of code (e.g. wind code uses 300-, 700- and 1700-year events) and its importance and use is not emphasized. There are no code requirements or recommendations for using the 500-year SWEL, therefore its existence may be undervalued. If no 500-year SWEL exists, FEMA recommends a 500 year SWEL as 1.25 times the 100-year SWEL (FEMA and NAHB Research Center 2010). Beyond the 500-year SWEL, there is no guidance to obtain elevation values associated with alternative return periods.

NFIP and code requirements may incorporate the use of freeboard, which is the extra elevation between the lowest horizontal structural member (LHSM) and the BFE (FEMA 2011). IRC and IBC code reference ASCE code for DFE procedures, which require freeboard varying from 0 to 2 feet, depending on the structure classification and LHSM orientation to wave

approach (ASCE 2005; ICC 2009; ICC 2011). The IRC specifies the DFE shall be equal to the BFE + 1 foot of freeboard (ICC 2009). The NFIP minimum elevation requirements call for the LHSM to be elevated to the BFE specified on a community's FIRM (FEMA 2011). A community may elect to adopt freeboard to improve their community rating for FEMA's voluntary community rating system (CRS) which in turn lowers insurance premiums (FEMA 2012). If a community chooses to adopt minimum NFIP regulations then the DFE defaults to the BFE, but if a community chooses to exceed NFIP requirements by adopting local freeboard regulations, the DFE then exceeds the BFE and becomes the BFE plus freeboard (FEMA 2011). A freeboard requirement through code is an arbitrary number applied to the entire country which provides an unknown level of protection. SWELs are not the same everywhere, yet freeboard is.

FEMA's CCM best practices integrate subsidence, erosion, and sea level rise, however, they do not specifically address sea level rise that occurs between the SWEL modeling date and the start of construction. Additionally, the CCM does not address the inclusion of tides and their possible effects on the SWEL or the importance of a vertical datum conversion. Chapter 2 provides recommendations for gaps in best practices and code-plus methods for building elevations which incorporate vertical datum transformations, total GSLR amounts, and high tides in SWEL computations, summarized in Table 2.2. The recommended guidance replaces freeboard, as does the CCM, but its use is intended as comprehensive guidance for practitioners.

4.1.1.3 Estimation of Longer Return Period SWELs

SWEL data from thirteen community FIS were evaluated to provide recommendations for estimating SWELs of longer return periods. The Huff-Angel and SRCC regression models were used to fit an equation to the SWEL data. The equation of best fit was extrapolated to determine the associated 700- and 1700-year SWEL for each community. The resulting projections were compared among differences in projected 500-year SWEL values and actual

FIS 500-year SWEL values, differences in the SWEL data normalized about the 100-year SWEL, and R^2 values. After the analysis, it was determined that the SRCC regression method provided the most reliable results for estimation of extreme flood values. A mean transect of the normalized SWEL data for the thirteen communities is developed and bounded by a 95% confidence interval. For the 95% confidence interval, the 700-year SWEL mean was found to be approximately 25% greater than the 100-year SWEL and the 1700-year SWEL mean was approximately 39% above the 100-year SWEL value.

4.2 Framework for New Recommendations

The goal of this research was to analyze gaps in current practices and to determine if SWELs of longer return periods could be developed using existing FIS SWEL data. The result provides an outline of code requirements, a summary of best practices, recommendations for gaps within existing practices, a methodology combining best practices with new recommended practices to reduce the gaps, and a method for determining SWEL values associated with longer return periods through extrapolation.

Chapters 2 and 3 are intended to be used together to estimate a SWEL of desired return period from FEMA FIS (Chapter 3), and to use the resulting SWEL to estimate the effects coastal conditions may impose on the SWEL (Chapter 2) over a structure's expected useful life. The new SWEL may then be used in accordance with Chapter 2 recommendations to arrive at the proposed DFE.

4.3 Conclusions and Applications

The primary research findings during the course of achieving the objectives of this thesis are summarized in the bulleted list below:

- Information on the BFE and DFE is scattered and difficult to find and/or interpret
- Information on the SWEL modeling process including resulting data is difficult to obtain

- The validity of the 100-year BFE as a minimum requirement for all classifications of structures should be challenged and questioned. The DFE should be related to the true risk of flooding rather than an arbitrary nationwide probability of exceedance.
- Freeboard guidance serves as an arbitrary number for designing above the BFE. SWELs and BFEs are not the same across the country, therefore freeboard amounts should not be the same for the entire country. An increase of two-feet above the BFE in Harrison County, Mississippi, will likely be far less effective than two-foot of freeboard in Virginia Beach, Virginia.
- Current best practices, although effective, include gaps:
    - Total high tide effects are ignored in SWEL calculations. Peak storm surge levels can occur simultaneously with normal high tide levels.
    - Sea level rise occurring between the modeling date of the SWEL and the start of construction is neglected. FIS SWEL modeling dates may be dated back multiple decades, thus leaving the effective SWEL unchanged for the current time period and susceptible to increases of the sea-level during this same time frame.
    - The importance of vertical datum differences are disregarded and guidance for converting among different datums is minimal. Vertical datum transformations can positively or negatively affect a DFE and should not be ignored. The variations can be the difference between flooding and not flooding.
- FIS for a minimal amount of communities provide the 500-year SWEL and if not provided, the only guidance is 1.25 times the 100-year SWEL (FEMA and NAHB Research Center 2010).
    - There is no guidance beyond the 500-year SWEL.

- The 500-year SWEL is not consistent with wind design, which uses 300-, 700-, and 1700-year events; and it is currently not used or required for elevation of any classification of structures.
- SRCC regression methods can be reliably used to determine SWEL values for events of longer return periods

The results of this research may be applied as a recommendation to any coastal community. Specific local characteristics of a community are necessary to achieve accurate results in accordance with Chapter 2. Chapter 2 methodologies are not limited to coastal high hazard areas; A-zones may be in locations experiencing subsidence, erosion, effects of sea level rise, and tidal changes. In addition, the vertical datum recommendation is applicable to any community, coastal or inland, referencing NGVD 29. Chapter, 2 in-parts or in-whole, may be applied to any community after analyzing the effects the community may be subject to in accordance with those presented in Chapter 2.

Chapter 3 presents a method for estimating SWELs that may be applied to any community that reports a SWEL for each of the 10, 50, 100, and/or 500-year events. A community with an FIS that calculates SWELs from peak-discharges, such as those subject to riverine and rainfall run-off flooding, can use this method by using the resulting SWELs. The flood zone associated with the community is primarily important for determining whether or not the effect of waves are included, which is described for coastal high hazard zones in Chapter 2. Therefore SWELs for any community can be used to fit a regression equation that can then be extrapolated to find the desired SWEL.

4.4 Final Remarks

The primary motivations behind this thesis were to improve the understanding of the effects changing coastal conditions may have on a SWEL and BFE, to investigate the reasoning

for designing above the 100-year BFE, to determine the amount of guidance that was available to accomplish this research, and to provide a recommendation for obtaining SWEL elevations beyond the 100-year frequency. The preceding sections describe the progress and findings of this research.

This research provides recommendations to close gaps among existing practices and describes how these gaps may affect risk associated with the coastal environment. However, more work is needed to improve the details and justify any potential change in building code. The presented case study illustrates the effects of the proposed recommendations, but more case studies and research can be used to solidify the need for the improvements and validate the findings of this methodology.

Estimation of SWELs associated with longer return periods serves to provide practitioners with a scientifically based method to determine new SWELs, but alternative regression-extrapolation methods may be used for different regions and locales that can provide more accurate results. More research in defining models that fit different regions will aid in providing more verifiable results. In addition, as newer FIS are published, the data used for SWEL determination will change and more communities may be provided with data for the four return periods, thus making them capable of implementing the methodology proposed in this research. Only 16 communities were evaluated in this research, and 13 of which were found to have the data needed to estimate the 700- and 1700-year SWEL. Including a number of intermediate communities between the communities in this thesis will yield more accurate groupings of SWELs, leading to more accurate regional projections across the coasts. Ideally, the findings of this thesis might serve as a basis for each community determining their own 700- and 1700-year flood elevations. With enough research and analysis, each individual community

should be able to determine SWELs of desired return periods using methods similar to these recommendations.

Incorporating the potential effects of conditions of the coastal environment with SWEL determination procedures, flood losses to structures elevated to the 100-year BFE can be effectively reduced. Including these coastal conditions with design flood levels above the 100-year can further mitigate flood losses. Improving the understanding of DFE and providing proposed methods to improve DFEs provides an opportunity to mitigate flood damages to structures, personal property, and loss of life.

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APPENDIX A: SUMMARY OF TRANSECT SELECTION

This appendix contains a description of how the transects for each specific community were obtained. Some transects are repeated and therefore are not used; some are not directly on the coast and are eliminated; and some provide ranges of elevations with the lower elevation representing areas that are not designated as a Coastal high Hazard Zone. The descriptions include where the SWEL values come from within each FIS.

1. South Padre, Cameron County, TX. The stillwater elevations (SWELs) are taken from *Table 3 Summary of Stillwater Elevation.* The SWELs are the same for each of the 22 transects shown in Figure 1 *Transect Location Map* and Table 4 T*ransect Descriptions*.
2. Jamaica Beach, Galveston County, TX. One transect represents the areas subject to flooding from the Gulf of Mexico. This information is provided in Table 1 *Summary of Stillwater Elevations*, Table 2 *Transect Description*, and Table 3 *Transect Data.*
3. Harrison County, MS. Transects 8-68 were used for the analysis whose primary source of flooding for these transects is the Gulf of Mexico. SWEL information is provided in Table 6 *Coastal Data Table.*
4. Bay County, FL. The FIS summarizes the transects in Table 10 *Transect Descriptions* and Figure 1 *Transect Location Map*. Transects 1-21 are used in the analysis; their primary source of flooding is the Gulf of Mexico. The 10, 50, and 500-year SWELs are summarized in Table 11 Transect Data. The elevation of 10.1 for the 100-year SWEL is used rather than 7.6 because this study is for structures in coastal high hazard areas subject to wave effects, and the 10.1 SWEL includes wave setup of 2.5.
5. Pinellas County, FL. Unique values for SWEL data used in the analysis are provided in Table 7 *Summary of Coastal Stillwater Elevations*. Only data whose primary source of flooding was the Gulf of Mexico was used in the analysis.

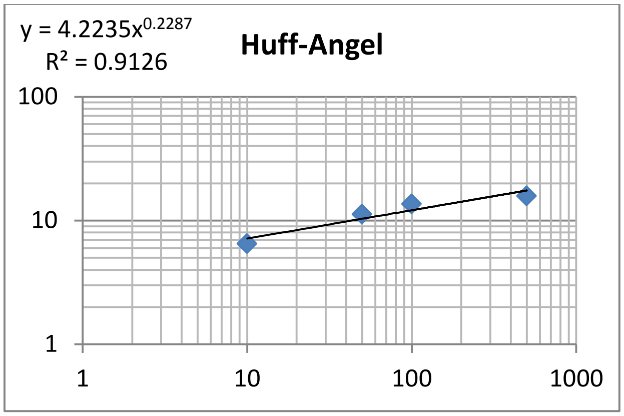
6. Martin County, FL. Unique values for SWEL data is provided in the FIS Table 5 *Summary of Stillwater Elevations*. Only transects whose flooding source is the Atlantic Ocean located along the open coast are used. Transects 1-15 are summarized in Table 8 *Transect Descriptions* and Figure 1 *Transect Location Map.*
7. St. Johns County, FL. Unique values for SWEL data is provided in Table 8 *Summary of Stillwater Elevations* for areas with the Atlantic Ocean as their primary flooding source. Transect locations and descriptions are summarized in Table 9 *Transect Descriptions* and Figure 2 *Transect Location Map.*
8. Charleston County, SC. Transect locations are provided in Figure 1 *Transect Location Map* and their descriptions are provided in Table 6 *Transect Descriptions*. The Unique values are provided in Table 7 *Transect Data.* Transects 1-5, and those located along rivers and channels are not included, only those transects that begin on the Atlantic shoreline are included.
9. Carteret County, NC. Transect locations are provided in Figure 2 *Transect Location Map* and their descriptions are provided in Table 13 *Summary of Coastal Analyses.* SWEL data is summarized in Table 13 *Summary of Coastal Analyses.* Only transects 1-46 are used in the analysis; transects 48+ are considered anomalies as they are located along undevelopable barrier islands.
10. City of Virginia Beach, VA. Table 2 *Summary of Stillwater Elevations* provides the SWEL data. Table 2 specifies flooding from the Atlantic Ocean is approximately the same throughout the shoreline of the entire community. The associated values are used as the unique value for the analysis.
11. Ocean County, NJ. Four sets of unique values used for the analysis are summarized in Table 5 *Summary of Stillwater Elevations*. Only the data with a value for each of the 10%, 2%, 1&,

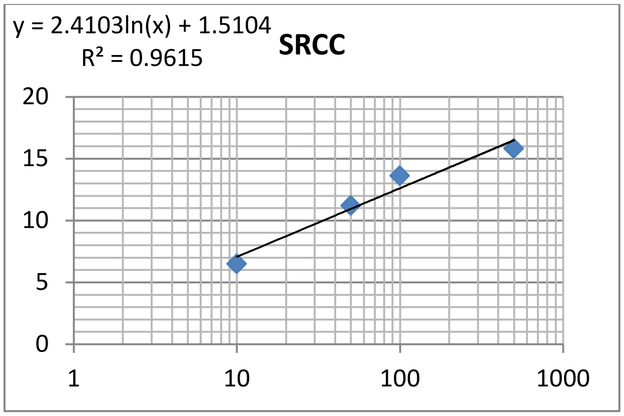
and 0.2% chance SWEL are used. Transect information is provided in Table 6 *Transect Descriptions* and Figure 1a & 1b *Transect Location Map.*

12. Dukes County, MA. SWEL values used in the analysis are provided in Table 10 *Summary of Stillwater Elevations* and are represented as those whose flooding source is the Atlantic Ocean. Transect information is provided in Table 11 *Transect Descriptions* and Table 12 *Transect Data*; no map for transect locations is provided.
13. Rockingham County, NH. SWEL values used in the analysis are provided in Table 5 *Summary of Stillwater Elevations* and are represented as those whose flooding source is the Atlantic Ocean. The SWELs for 'Entire Shoreline within Portsmouth' are not included in the analysis; Portsmouth is located slightly inland rather than directly on the coast thus making the values an anomaly. Transect information is provided in Figure 1 *Transect Location Map* and Table 7 *Transect Data.*

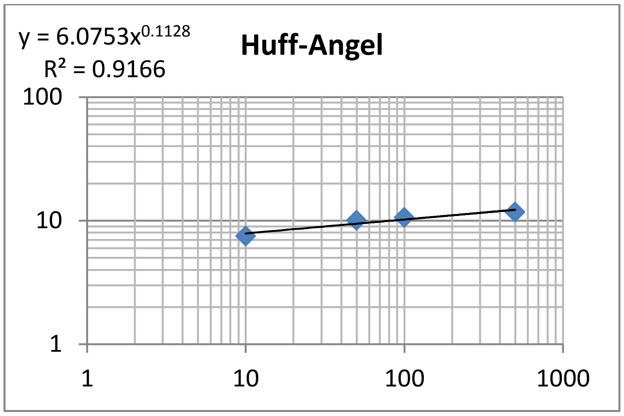
APPENDIX B: REGRESSION PLOTS FOR EACH COMMUNITY

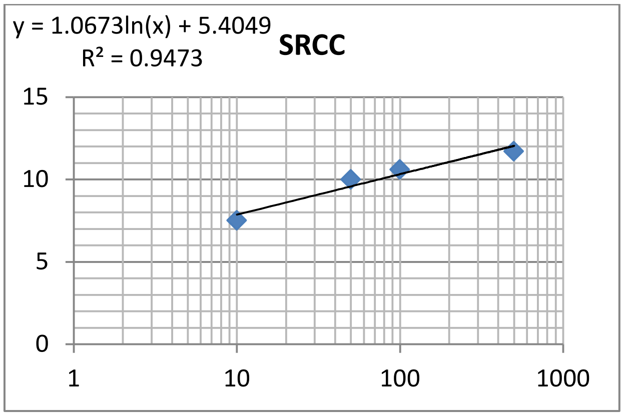
This appendix contains the Huff-Angel and SRCC regression plots for each of the thirteen communities. The equation and R^2-Value of each method and each individual community are provided. The four scatter points on each graph are representative of the 10, 50, 100, and 500-year SWEL given from the community's FIS, and the straight line is the regression equation fit to the four scatter points. The Huff-Angel graph is displayed with log-log axes and a power formatted trendline; the SRCC graph is displayed with a logarithmic x-axis and a linear y-axis and a logarithmic formatted trendline.



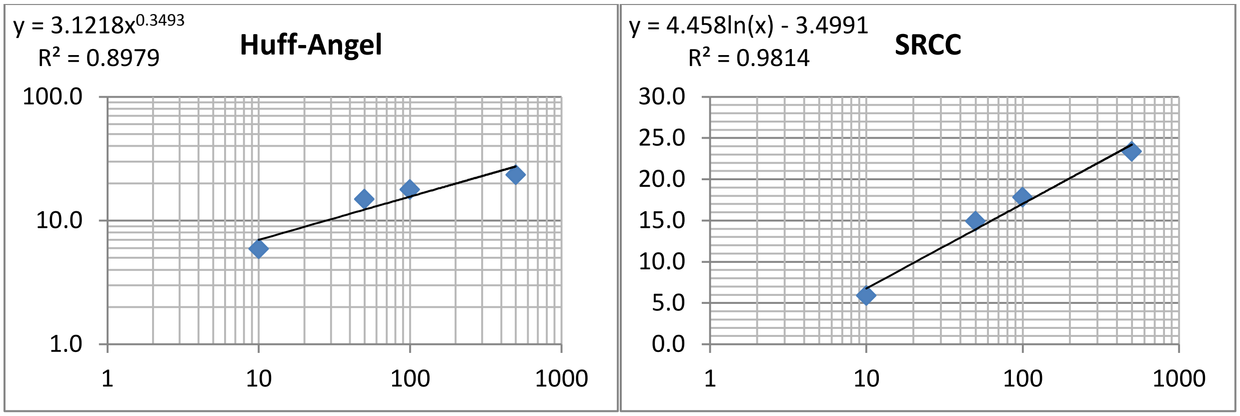


Cameron County, TX

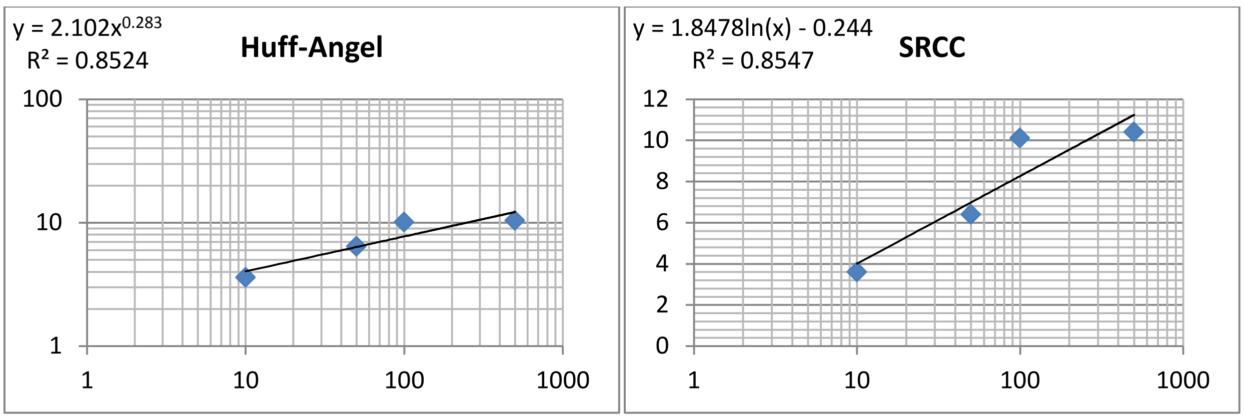




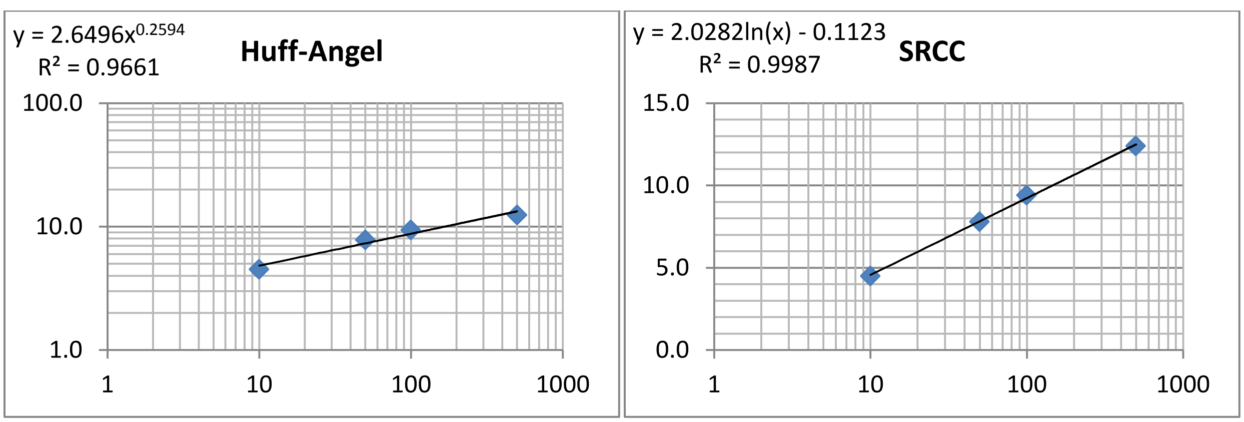
Galveston County, TX



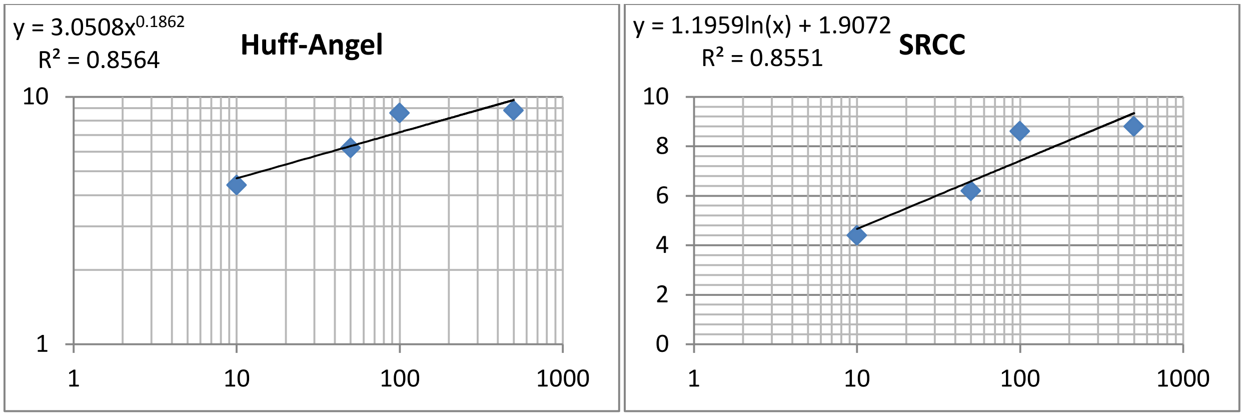
Harrison County, MS



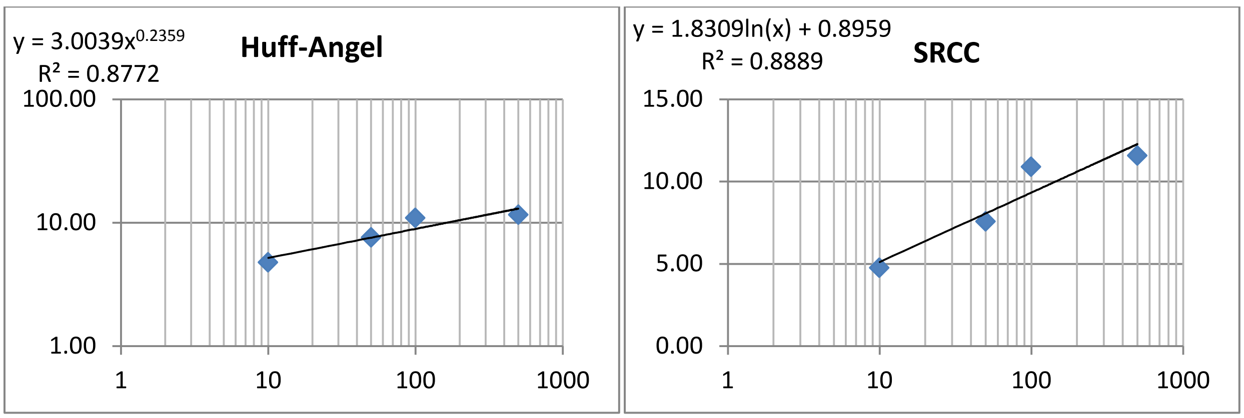
Bay County, FL



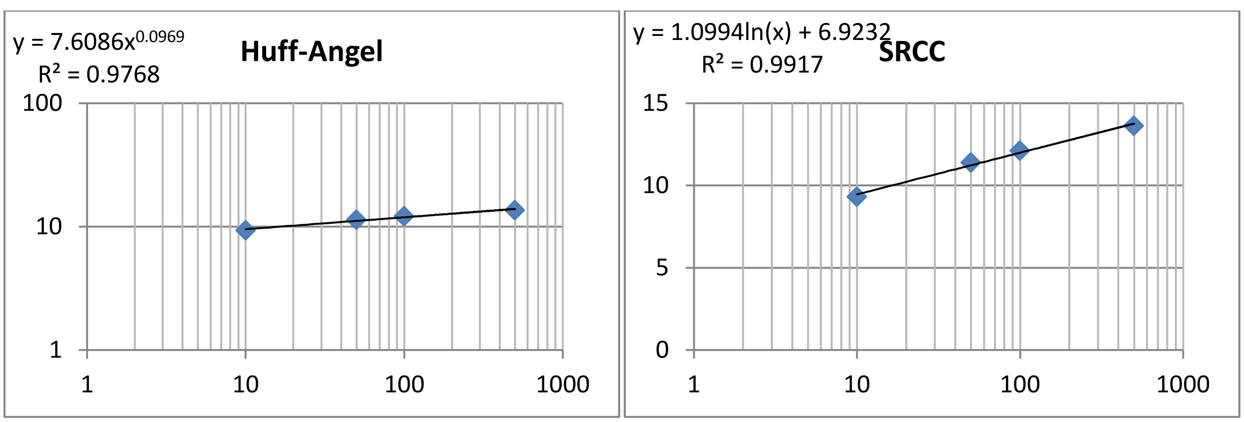
Pinellas County, FL



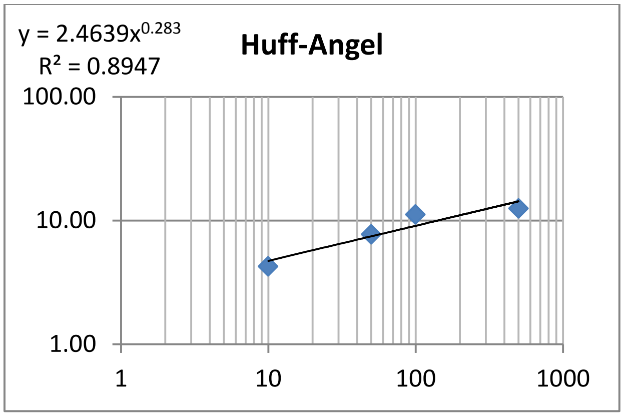
Martin County, FL

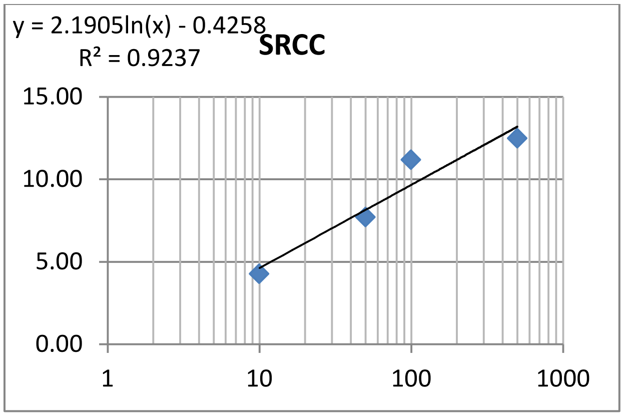


St. Johns County, FL



Charleston County, SC





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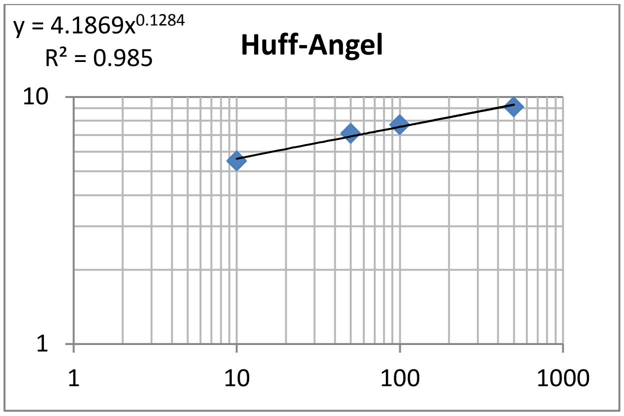
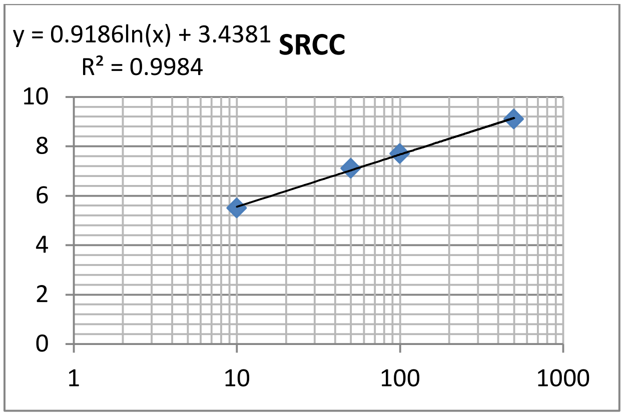
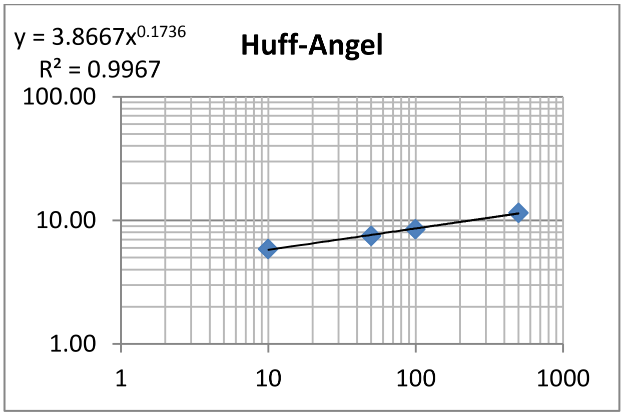
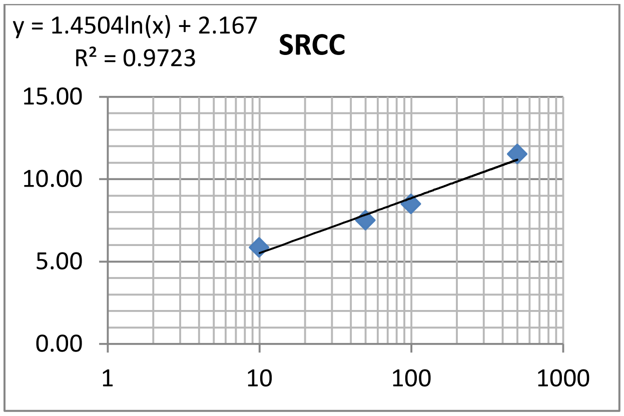


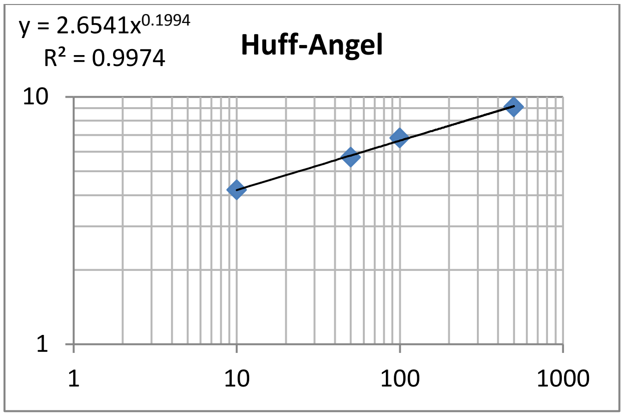


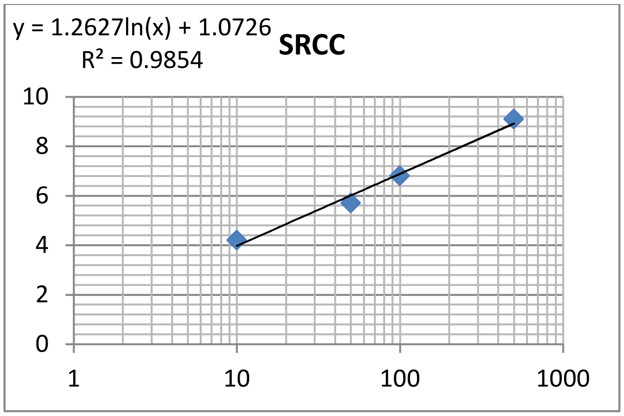
Virginia Beach City County, VA



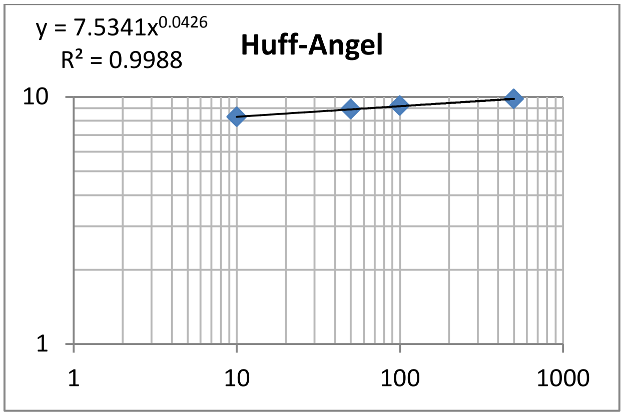


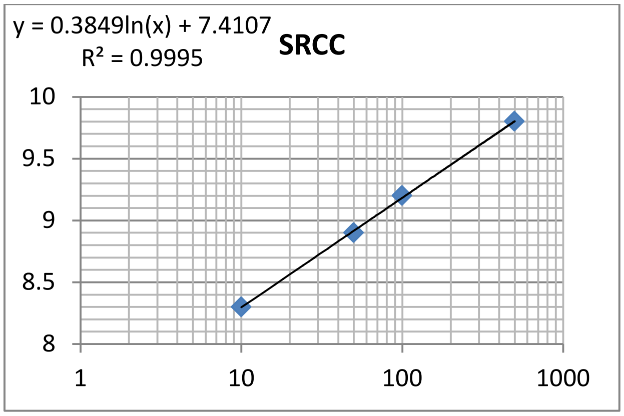
Ocean County, NJ





Dukes County, MA





Rockingham County, NH

VITA

Frank H. Bohn was born in Galveston, Texas to Linda J. Bohn and Paul M. Bohn. Frank has two older siblings, Kathy and David, and one younger sister, Allison. Frank graduated in the top 3% of his senior class from Galveston Ball High School and subsequently attended Louisiana State University. Frank graduated with a bachelor's degree in Construction Management with Magna Cum Laude honors. Frank began the Accelerated Master of Science program in Engineering Science during the senior year of his undergraduate program. Frank has also completed thirty extra hours of surveying coursework to enable him to sit for the Fundamentals of Surveying exam. Frank has accepted a job with Vaughn Construction in Galveston, Texas, following graduation.