Probabilistic Performance-Based Hurricane Engineering (PBHE) Framework

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PROBABILISTIC PERFORMANCE-BASED HURRICANE ENGINEERING (PBHE) FRAMEWORK

A Dissertation

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

in

The Department of Civil and Environmental Engineering

by

Vipin Unnithan Unnikrishnan
B. Tech., University of Kerala, 2005
M.S., Indian Institute of Technology Madras, 2010
August 2015
TO MY PARENTS
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ABSTRACT

In modern times, hurricanes have caused enormous losses to the communities worldwide both in terms of property damage and loss of life. In light of these losses, a comprehensive methodology is required to improve the quantification of risk and the design of structures subject to hurricane hazard.

This research develops a probabilistic Performance-Based Hurricane Engineering (PBHE) framework for hurricane risk assessment. The proposed PBHE is based on the total probability theorem, similar to the Performance-Based Earthquake Engineering (PBEE) framework developed by the Pacific Earthquake Engineering Research (PEER) Center, and to the Performance-Based Wind Engineering (PBWE) framework. The methodology presented in this research disaggregates the risk assessment analysis into independent elementary components, namely hazard analysis, structural characterization, interaction analysis, structural analysis, damage analysis, and loss analysis. It also accounts for the multi-hazard nature of hurricane events by including the separate effects of, as well as the interaction among, hurricane wind, flood, windborne debris, and rainfall hazards.

This research uses the Performance-Based Hurricane Engineering (PBHE) framework with multi-layer Monte Carlo Simulation (MCS) for the loss analysis of structures subject to hurricane hazard. The interaction of different hazard sources is integrated into the framework and their effect on the risk assessment of non-engineered structures, such as low-rise residential buildings, is investigated. The performance of popular storm mitigation techniques and design alternatives for residential buildings are also compared from a cost-benefit perspective. Finally, the PBHE framework is used for risk assessment of engineered structures, such as tall buildings. The PBHE
approach introduced in this study represents a first step toward a rational methodology for risk assessment and design of structures subjected to multi-hazard scenarios.
CHAPTER 1
INTRODUCTION

Hurricanes are among the most costly natural hazards affecting communities worldwide, in terms of both property damage and loss of life. In recent years, severe hurricanes have caused enormous economic losses for the society and have placed tremendous burden on the insurance industry. In the U.S., the average annual economic loss (normalized to 2005 USD) due to hurricanes in the period 1900-2005 was about $10 billion (Pielke et al. 2008). Therefore, new methods for accurate risk assessment, effective risk mitigation, and efficient decision making are needed to improve the resilience of the nation and, in particular, of coastal communities to hurricane events. A fundamental ingredient in reducing the ecological and socioeconomic risks of hurricane hazard is the availability of a widely-accepted, general, and rigorous structural design methodology that is able to account for all pertinent sources of uncertainty and provides direct information on the performance of the structures of interest. A promising approach to develop such structural design methodology for hurricane engineering is offered by the general design philosophy of Performance-Based Engineering (PBE).

PBE is a general methodology that (1) defines the performance objectives for structural systems during their design life, (2) provides criteria and methods for verifying the achievement of the performance objectives, and (3) offers appropriate methodologies to improve the design of structural systems. PBE approaches are widely accepted as means of achieving earthquake resilient designs and, thus, have been vigorously adopted in the field of earthquake engineering and other sub-fields of civil engineering. However, similar approaches are relatively new in field of wind engineering (Petrini 2009, Smith and Caracoglia 2011, Barbato et al. 2013, Spence and Kareem 2014).
1.1 Objectives and Motivation

The advantages demonstrated by a PBE approach in structural design of civil structures provide a strong motivation to develop a comprehensive PBE methodology for structures subject to hurricane hazard. The need for assessing and improving the resilience of the built environment subjected to hurricane hazard is widely recognized. Some initial interest in PBE has been expressed in hurricane engineering, but a complete and rigorous framework is still needed (Augusti and Ciampoli 2008).

The main goal of the research presented in this dissertation is to present a comprehensive Performance-Based Hurricane Engineering (PBHE) framework for the probabilistic hurricane risk assessment and design of structural systems, which rationally accounts for all pertinent sources of uncertainty and explicitly considers the multiple hazard sources that characterize a hurricane event, thereby leading to a reduction and/or control of economic and societal losses from hurricanes. Additional specific objectives are:

1. identify the main hazard sources involved in hurricane events and investigate their interaction;
2. specialize the framework for the risk assessment of pre-engineered buildings and develop a faster re-analysis method to improve the computational efficiency when numerous performance assessment analyses are required for the same building;
3. compare the performance of different storm mitigation techniques and design alternatives for residential buildings subjected to wind and windborne debris hazard;
4. specialize the framework for the risk assessment of tall buildings (engineered structures) subjected to wind hazard.
1.2 Scope of the Study

This dissertation focuses on the development of the Performance-Based Hurricane Engineering (PBHE) framework and its application to the risk assessment of civil structures. The formation of tropical storms that are accompanied by extreme winds and flood, and their related modeling (such as boundary layer modeling or hydrodynamic modeling) are not considered here.

1.3 Organization of the Dissertation

This dissertation is prepared in a multiple-paper format and, as such, presents some repetitions that are inevitable. It is comprised of five chapters and two appendices. Chapters 2 through 4 report research results that have been published, submitted and currently under review, or are being prepared for submission to a peer-reviewed technical journal. The author of this dissertation is the main author of the papers containing the work presented in Chapters 2 through 4, and is a co-author of the published paper containing the material presented in Appendix A.

Chapter 1 contains an introduction of this thesis. Chapter 2 focuses on the hurricane loss analysis of residential buildings and the effects of mitigation techniques for wind and windborne debris hazards on the structural performance. The PBHE framework is specialized to the hurricane risk assessment of low-rise residential buildings and a highly efficient modification of the multi-layer Monte-Carlo simulation (MCS) technique based on copula is also proposed for faster re-evaluation of hurricane risk of the same building when comparing different storm mitigation techniques and/or design alternatives. Chapter 3 presents the performance-based risk assessment of tall buildings subject to wind hazard considering the losses due to damage to structural elements, damage to non-structural elements, and building occupants’ discomfort. In Chapter 4, the PBHE framework is applied to problems in which all different hazard sources occurring during a hurricane event (i.e., wind, windborne debris, rainfall, and flood) are active. The interaction among
these multiple hazard sources and its treatment within the PBHE framework are discussed in detail. A hypothetical case study is presented to illustrate the proposed methodology and the specialized multi-layer MCS approach for loss analysis of residential buildings subject to hurricane hazard including all pertinent hazard sources. Chapter 5 summarizes the findings of this research, draws some conclusions, and outlines possible areas of future work.

Appendix A provides a detailed description of PBHE framework, which has been published as a result of the collaborative research among the author of this dissertation, his major advisor (Dr. Michele Barbato), and Drs. Francesco Petrini and Marcello Ciampoli (University of Rome, Italy). The original, individual contributions of the author of this dissertation to the above paper are:

1. identification of the key parameters for the probabilistic characterization of windborne debris hazard,
2. development of the multi-layer Monte Carlo simulation (MCS) method for PBHE,
3. implementation of the PBHE framework and multi-layer MCS method for the application example presented in the paper, and
4. investigation of the interaction between hazard sources for the specific application example.

Appendix B contains the permission to reproduce published and under review material.

1.4 References


CHAPTER 2
PERFORMANCE-BASED COMPARISON OF DIFFERENT STORM MITIGATION TECHNIQUES FOR RESIDENTIAL BUILDINGS

2.1 Introduction

Hurricanes are among the most costly natural hazards affecting communities worldwide, in terms of both property damage and loss of life. In the U.S., the average annual economic loss due to hurricanes in the period 1900-2005 was about $10 billion (normalized to 2005 USD), and placed a tremendous burden on the society and the insurance industry (Pielke et al. 2008). As the population tends to concentrate on coastal regions and the number of residential buildings in hurricane-prone areas continues to rise, the societal vulnerability to hurricanes is increasing, with the prospect of even higher damages and losses in the future (Li and Ellingwood 2006). Hence, hurricane hazard mitigation is of paramount importance for residential buildings located in hurricane-prone regions. Many mitigation measures are available to reduce the social and economic losses that are associated with hurricane damage, and appropriate engineering criteria must be used to select the most cost-effective solutions for different conditions. In the case of residential buildings, hurricane risk mitigation is limited by the high upfront cost of common hurricane risk mitigation practices. In order to reduce the societal risk posed by hurricane events in a cost-effective manner, appropriate decision-making tools must be developed based on a rigorous performance-based cost-benefit evaluation of different mitigation techniques for residential buildings.

In the last few decades, significant research was devoted to developing vulnerability models (also called fragility curves) for residential buildings subject to hurricane hazard. Leicester et al. (1980) developed global vulnerability curves (i.e., for the entire building) for various housing types based on cyclone damage surveys in different regions of Australia after Cyclone Tracy in 1974. Stubbs
and Perry (1996) defined vulnerability models for different building components based on reliability analysis techniques and investigated the relative contribution from the damage of individual components to the total damage for buildings subject to extreme wind events. Huang et al. (2001) developed a hurricane damage model for single family housing units using event-based simulation and Southeastern U.S. insurance data from Hurricanes Hugo and Andrew to predict the expected losses at a regional level. Pinelli et al. (2004) proposed a probabilistic model for hurricane vulnerability evaluation of residential structures using basic damage modes for individual structural and non-structural components and combining them in possible damage states for specific building types.

systems subject to hurricane hazard. This framework considers the multi-hazard nature of hurricane events, the interaction of different hazard sources (e.g., wind, windborne debris, flood, and rain), and possible sequential effects of these distinct hazards.

In parallel with the development of performance-based design approaches, the last two decades have seen the advancement of risk-based cost-benefit analysis approaches in several subfields of structural engineering (e.g., see Frangopol et al. (1997) for bridge engineering, Porter et al. (2001) for earthquake engineering). Stewart et al. (2003) performed a hurricane damage risk-cost-benefit analysis proposing two scenario-based models to investigate the structural vulnerability change for the existing building stock due to improvements in the building envelope performance, as well as the effects over time of this change on expected insurance losses. Pinelli et al. (2009) analyzed the cost-effectiveness of various mitigation measures for different residential building typologies of different age and quality of construction. Li (2010) proposed a risk-cost-benefit framework for assessing the damage risk and cost-effectiveness of hurricane and earthquake mitigation strategies for residential buildings using life-cycle and scenario-case analysis. Li and van de Lindt (2012) proposed a loss-based formulation for residential buildings subject to multiple hazards, in which cost-benefit analysis was used to compare different design and retrofit options for multi-hazard mitigation.

In this paper, the PBHE framework (Barbato et al. 2013) is adopted for the risk assessment of structural systems located in hurricane-prone regions. Multi-layer Monte Carlo Simulation (MCS) is employed to perform a loss analysis for residential buildings subject to hurricane hazard. A highly efficient modified version of multi-layer MCS is proposed for faster re-evaluation of hurricane risk when different design alternatives and mitigation strategies are considered for the same building. These design alternatives and mitigation strategies are compared using a risk-based
cost-benefit analysis. A realistic case study is presented to illustrate the adopted methodology by comparing the cost-effectiveness of different hurricane hazard mitigation techniques applied to a typical house of an actual residential development located in Pinellas County, FL.

2.2 Summary of PBHE Framework

The PBHE framework proposed in Barbato et al. (2013) disaggregates the performance assessment procedure for structures subject to hurricane hazard into elementary phases that are carried out in sequence. The structural risk within the PBHE framework is expressed by the probabilistic description of a decision variable, $DV$, which is defined as a measurable quantity that describes the cost and/or benefit for the owner, the users, and/or the society resulting from the structure under consideration. The fundamental relation for the PBHE framework is given by:

$$G(DV) = \int \int \int G(DV|DM) \cdot f(DM|EDP) \cdot f(EDP|IM,IP,SP) \cdot f(IP|IM,SP) \cdot f(IM) \cdot f(SP) \cdot dDM \cdot dEDP \cdot dIP \cdot dIM \cdot dSP$$

(2.1)

where $G(\cdot) = \text{complementary cumulative distribution function}$, and $G(\cdot|\cdot) = \text{conditional complementary cumulative distribution function}$; $f(\cdot) = \text{probability density function}$, and $f(\cdot|\cdot) = \text{conditional probability density function}$; $IM = \text{vector of intensity measures}$ (i.e., parameters characterizing the environmental hazard); $SP = \text{vector of structural parameters}$ (i.e., parameters describing the relevant properties of the structural system and non-environmental actions); $IP = \text{vector of interaction parameters}$ (i.e., parameters describing the interaction phenomena between the environment and the structure); $EDP = \text{vector of engineering demand parameters}$ (i.e., parameters describing the structural response for the performance evaluation); and $DM = \text{vector of damage measures}$ (i.e., parameters describing the physical damage to the structure). By means of Eq. (2.1) the risk assessment is disaggregated into the following tasks: (1) hazard analysis, (2)
structural characterization, (3) interaction analysis, (4) structural analysis, (5) damage analysis, and (6) loss analysis.

2.3 Multi-layer Monte Carlo Simulation

Similar to the Pacific Earthquake Engineering Research Center performance-based earthquake engineering framework equation (Cornell and Krawinkler (2000), Porter (2003)), Eq. (2.1) can be solved using different techniques, e.g., closed-form analytical solutions (Shome and Cornell (1999), Jalayer and Cornell (2003), Mackie et al. (2007), Zareian and Krawinkler (2007)), direct integration techniques (Bradley et al. 2009), and stochastic simulation techniques (Porter and Kiremidjian (2001), Au and Beck (2003), Lee and Kiremidjian (2007)). In PBHE, analytical solutions and direct integration techniques require the knowledge of the joint probability density function of the component losses, which is very difficult to obtain for real-world applications. Thus, in this study, multi-layer MCS (Conte and Zhang 2007) is adopted and specialized to efficiently perform loss analysis for residential buildings subject to hurricane hazard. The result of the PBHE equation (Eq.(2.1)) is the annual loss curve, $G(DV)$, i.e., the complementary cumulative distribution function of the annual losses for the residential building under consideration due to hurricane events.

Figure 2.1 shows the flowchart of the general multi-layer MCS technique applied to PBHE considering a one-year time interval. Multi-layer MCS takes into account the uncertainties from all phases of the PBHE framework (namely, hazard analysis, structural characterization, interaction analysis, structural analysis, damage analysis, and loss analysis). Each of these analysis phases is performed in two steps: (1) a sample generation step of random parameters with known probability distributions, which are needed to describe the uncertainties in environmental actions, structural properties, interaction phenomena, analysis techniques, and cost estimates; and (2) an
analysis step based on a deterministic model, which is used to propagate the uncertainties from input to output parameters of each analysis phase.

![Diagram of multilayer MCS approach for PBHE framework](image)

- **IM** parameters
- **SA** parameters
- **DA** parameters
- **LA** parameters

**For each hurricane**

- **SP**
- **IP**
- **EDP**
- **DM**
- **DV**

**DV per year**

**Chain effects**

- Sample generation (stochastic simulation)
- Analysis step (e.g., FE analysis, windborne debris trajectory analysis)

**IA**: interaction analysis; **SA**: structural analysis; **DA**: damage analysis; **LA**: loss analysis

**Figure 2.1** Multi-layer MCS approach for PBHE framework.

It is noted here that the analysis steps are usually more computationally intensive than the corresponding sample generation steps. Thus, it is useful to identify specific conditions under which one or more of the analysis steps can be avoided in order to reduce the computational cost of the multi-layer MCS approach. In particular, this study focuses on hurricane loss analysis for low-rise residential buildings such as single-family houses. For these specific building typology, component strength statistics are commonly available as functions of the environmental action intensity. In fact, most of these structures are constructed based on design models, and their components consist of products that are certified based on building code requirements (NAHB 2000).

Under these conditions, the damage analysis phase can be performed without requiring the statistical description of the structural response of the building. In fact, the probabilistic description
of the strength for the building components subject to damage (i.e., windows, doors, walls, and roof) can be obtained from empirical relations available in the literature as a function of opportune chosen IP.

Figure 2.2 Multi-layer MCS approach for probabilistic hurricane loss estimation of non-engineered residential buildings.

Thus, it is computationally convenient to eliminate the structural analysis phase from the multi-layer MCS procedure. Figure 2.2 shows the flowchart for the multi-layer MCS technique specialized for probabilistic hurricane loss estimation of residential buildings, and provides the list of analysis parameters involved in each analysis phase. As noted above, the structural analysis phase is not performed explicitly to derive the probabilistic description of the EDPs that are related to structural damage. This simplification considerably reduces the computational cost of the multi-layer MCS approach for probabilistic hurricane loss analysis of residential buildings. The following sections of this chapter describe in detail the PBHE phases for the proposed specialized multi-layer MCS technique. It is noted here that, for simple structures of risk category I and II, (ASCE 2010) such as single-family residential buildings, simplified and computationally
inexpensive models are often appropriate to perform the analysis steps required by the PBHE methodology.

2.3.1 Hazard analysis phase

The focus of this chapter is on the effects of mitigation techniques for wind and windborne debris hazards. Thus, the results presented in this chapter are valid for residential buildings that are sufficiently far from water bodies and for which flood hazard mitigation is not required. It is noted here that the general multi-layer MCS methodology presented in this study can be generalized to include also flood and rainfall hazard. However, this generalization is outside the scope of this chapter.

2.3.1.1 Wind hazard characterization

The first step in the proposed multi-layer MCS approach is the simulation of the number of hurricanes affecting the considered structure in a given year, e.g., according to a Poisson occurrence model (Russel (1971), Chouinard and Liu (1997), Elsner and Kara (1999)). For each of these hurricanes, a corresponding wind field needs to be simulated in order to characterize the wind hazard. Three methodologies of increasing accuracy and computational cost can be adopted to define the hurricane wind field (FEMA 2007): (1) deriving the statistical description of the 3-second gust wind velocity, \( V \), at the building site from existing peak wind speed data (Batts et al. (1980), Peterka and Shahid (1998), Li and Ellingwood (2006)); (2) using site specific statistics of fundamental hurricane parameters to obtain a mathematical representation of a hurricane at the building location, including the statistics of the wind speed (Batts et al. (1980), Vickery and Twisdale (1995)); and (3) modeling the full track of a hurricane from its initiation over the ocean until final dissipation and using appropriate wind field models to obtain the wind speed statistics corresponding to the specified track at the building site (Vickery et al. 2000).
In this paper, the first methodology (i.e., using existing peak wind speed data at the buildings site to derive the statistical description of the 3-second gust wind velocity) is adopted in order to reduce the computational cost of the proposed procedure. However, for important structures, one of the more accurate procedures would be more appropriate and should be selected. It is also noteworthy that, when the number of hurricanes per year is equal to zero, the proposed PBHE framework reduces to the performance-based wind engineering framework proposed in Petrini (2009), and can be used to assess the performance of structure subject to non-hurricane wind actions. When the number of hurricanes per year is larger than zero, the procedure shown in Figure 2.1 or Figure 2.2 is always performed first for non-hurricane wind actions, and then repeated for hurricane actions a number of times equal to the simulated number of hurricanes.

2.3.1.2 Windborne debris hazard characterization

The windborne debris hazard is described by the wind field intensity (which is also needed to describe the wind hazard) and the characteristics of the windborne debris that can affect the structure under study. The parameters needed to describe the windborne debris are: (1) the relative distribution of different debris types, e.g., compact-type, rod-type, and sheet-type debris (Wills et al. 2002); (2) the physical properties of the debris, e.g., for sheet-type debris, $M_d = \text{mass per unit area of the debris}$, and $A_d = \text{area of the single debris}$; (3) the density of debris sources, e.g., the buildings’ density (applicable for expanding residential developments), $n_{\text{buildings}}$, and the vegetation density, $n_{\text{vegetation}}$, at the building’s site; (4) the resistance model for the debris sources (which contributes to determine the number of windborne debris generated by a given source under a specified wind speed); and (5) the trajectory model for the debris (which describes the debris flight path).
The relative distribution of the debris types and the statistical description of the variables defining the physical properties of the debris can be obtained either from the literature or through damage surveys at the site from previous hurricane events. In residential developments, the windborne debris are predominantly sheet-type, e.g., roof shingles and sheathing (Holmes 2010), hence this chapter focuses on sheet-type debris. The debris source’s density can be obtained from direct observation of the building site, as well as from development and/or urban planning documents. Several debris generation models are available in the literature, e.g., component-based pressure-induced model (Gurley et al. 2005), empirical models based on damage surveys (FEMA-325 2007). In this study, the debris generation model employed by the Florida Public Hurricane Loss Model (FPHLM) is adopted. This model is a component-based pressure-induced damage model, which provides the number of debris generated from each source house as a function of (1) the percentage of roof cover damage for a given 3-second gust wind speed, and (2) the geometry of the house.

Two different types of debris trajectory models are available in the literature to estimate the debris flight path: (1) models that investigate the two dimensional motion of debris in uniform wind flow using simplified dimensionless equations of motion (Holmes (2004), Lin et al. (2007), Baker (2007)), and (2) models that consider the debris trajectory in a three dimensional space through the numerical integration of the three- or six-degree-of-freedom debris equations of motion (Twisdale et al. (1996), Richards et al. (2008), Grayson et al. (2012)). To reduce the computational cost of windborne debris hazard analysis, a two dimensional model using simplified dimensionless equations of motion proposed by Lin et al. (2007) is adopted in this study to estimate the debris flight trajectory. This model provides the landing position of the debris in terms of two Gaussian random variables, i.e., $X = $ along-wind flight distance and $Y = $ across-wind flight distance, which
described by the following parameters: \( \mu_x \) = mean along-wind flight distance; \( \mu_y \) = 0 m = mean across-wind flight distance; \( \sigma_x = \sigma_y = 0.35 \mu_x \) = standard deviation of the along-wind and across-wind flight distance, respectively (Lin and Vanmarcke 2008). The parameter \( \mu_x \) is computed as:

\[
\mu_x = \frac{2M_d}{\rho_a} \left[ \frac{1}{2} C \left( K \cdot T \right)^2 + c_1 \left( K \cdot T \right)^3 + c_2 \left( K \cdot T \right)^4 + c_3 \left( K \cdot T \right)^5 \right]
\]

(2.2)

in which \( \rho_a \) = air density; \( K = \frac{\rho_a \cdot V^2}{2M_d \cdot g} \) = Tachikawa number; \( \tilde{T} = \frac{g \cdot T}{V} \) = normalized time; \( g \) = gravity constant; \( T \) = flight time in seconds; and \( C, c_1, c_2, \) and \( c_3 \) = non-dimensional coefficients that depend on the shape of the debris and were calibrated using wind tunnel tests (Lin et al. 2007).

2.3.1.3 Interaction among hazards in the hazard analysis phase

The interaction among different hazard sources can take place in the form of: (1) interacting hazards, and (2) hazard chains. The PBHE framework accounts for the former type of interaction within the hazard analysis phase by considering two modes of interaction: (1) different hazards described using shared \( IM \) (e.g., wind and windborne debris hazards require the description of the wind field, which is common to both hazards for a given hurricane event); and (2) one or more hazards described by statistically dependent \( IM \), which can be modeled using joint probability density functions (see, e.g., Myers (1975), Myers and Ho (1975), Toro et al. (2010)). In this study, the \( IM \) used to describe the wind field and the debris properties are assumed as independent random variables.

2.3.2 Structural characterization phase

The structural characterization phase provides the probabilistic description of the \( SP \) vector, which includes the random structural properties that can influence the loading applied to the structure and/or its components through the \( IP \) vector. These properties can include, e.g., geometrical
properties, such as position and dimensions of windows and doors as well as the dimensions of the building (length, width, and height); mechanical properties, such as natural period and damping; and other parameters that determine the intensity of the wind effects on the structure and its components. Geometrical properties can usually be treated as deterministic quantities, since they can be directly measured for existing structures or are characterized by a small variability. In general, the variability of the mechanical properties of a low-rise residential building has a negligible effect on the performance of the building itself and can also be neglected. The statistical characterization of the other parameters affecting the intensity of the wind effects can be obtained through wind tunnel tests or from appropriate statistical distributions available in the literature. The latter approach is followed in this study. It is noteworthy that the statistical distributions of these parameters usually change any time the building envelope is breached. Thus, it is important to account for these changes in order to properly evaluate the effects of hazard chains (Barbato et al. 2013). In this study, the following random structural parameters are considered: wind pressure exposure factor (evaluated at \( h = \) height of the target building), \( K_h \); external pressure coefficient for the \( j \)-th building component, \( GC_{p,j} \); and internal pressure coefficient for the \( j \)-th building component, \( GC_{pi,j} \) \((j = 1, \ldots, n_c\), where \( n_c \) = number of building components\). The variability of the wind gust factor \( G \) is incorporated in that of external and internal pressure coefficients because it is usually small for the building typology considered in this study (Li and Ellingwood 2006).

### 2.3.3 Interaction analysis phase

The choice of the \( IP \) vector is crucially dependent on the hazard sources, limit states, and performance levels of interest for both structural and non-structural elements. In this study, the \( IP \) vector is selected to represent the effects of wind and windborne debris hazard on the different limit states of interest for low-rise residential buildings.
The interaction analysis for the wind hazard provides the statistical characterization of the wind pressure acting on the different components of the buildings, $p_{w,j}$. In this study, the wind pressure acting on the $j$-th component of the building is computed as (ASCE 2010)

$$p_{w,j} = q_h \cdot \left( GC_{p,j} - GC_{p,\emptyset} \right)$$

(2.3)

in which the velocity pressure evaluated at $h$, $q_h$, is given by

$$q_h = 0.613 \cdot K_h \cdot K_{at} \cdot V^2 \quad \text{(units: N/m}^2)$$

(2.4)

The relevant IP components controlling the effects of windborne debris impact are: (1) number of impacting debris, $n_d$; (2) impact linear momentum, $L_d$; and (3) impact kinetic energy, $K_d$. The impact linear momentum is well correlated with the damage to envelope components with a brittle behavior (e.g., glazing portions of doors and windows (Masters et al. 2010)), whereas the impact kinetic energy is better correlated with the damage to envelope components with a ductile behavior (e.g., aluminum storm panels, see Herbin and Barbato (2012), Alphonso and Barbato (2014)).

The analysis step of the interaction analysis phase requires an impact model to estimate $n_d$, $L_d$, and $K_d$ (Barbato et al. 2013). The debris impact model uses the debris flight path obtained from the trajectory model to check for any impact with the target building. In the event of an impact, it uses the horizontal component of the missile velocity and data relative to the missile size and mass (obtained from the debris generation model) to compute the impact linear momentum and kinetic energy of the missile, which are given by:

$$L_d = M_d \cdot A_d \cdot u_d$$

$$K_d = \frac{1}{2} M_d \cdot A_d \cdot u_d^2$$

(2.5)

18
in which $u_d$ denotes the debris horizontal velocity at impact and is given by (Lin and Vanmarcke 2008)

$$u_d = V \cdot \left[1 - \exp \left(-\sqrt{2 \cdot C \cdot K \cdot x}\right)\right]$$

(2.6)

in which $x = \frac{g \cdot X}{V^2}$ = dimensionless horizontal flight distance of the debris.

### 2.3.4 Damage analysis phase

In the methodology proposed here for low-rise residential buildings, the structural analysis phase is not performed explicitly and the strength of vulnerable components is directly compared to the corresponding $IP$. Following a procedure commonly used in performance-based earthquake engineering, the physical damage conditions are represented using a limit state function $LSF$ for each damage limit state, i.e.,

$$LSF_j = DM_j - IP_j$$

(2.7)

where $DM_j$ correspond to the limit state capacity of the component $j$, for the given damage limit state. The limit states generally considered for residential buildings are (1) breaking of annealed glass windows/doors, (2) uplift of the roof sheathings, (3) uplift of the roof covers, (4) roof truss failure, and (5) wall failure. The $IP$s are compared with the limit state capacity of different components of the building, and if the $IP$s assume values larger than the corresponding limit state capacity of the building component, the component is assumed to fail. In case of any breach in the building envelope, the interaction and damage analysis phases are repeated with updated $SP$s until there is no further additional breach (Figure 2.2).

### 2.3.5 Loss analysis phase

The loss analysis phase gives the estimate of the annual probability of exceedance of the $DV$. The $DV$ can be chosen as the repair cost related to the hurricane induced damage, or the total cost of
the structural system during its design lifetime (including construction and maintenance costs, repair costs, economic losses due to structural and content damage, and loss of functionality) (Bjarnadottir et al. 2011). The statistical description of the repair cost for each of the building components can be obtained from the literature and/or market, and the loss can be calculated as a function of the percentage of component damage. Repair costs depend on local labor cost, availability of materials and local construction practices. Loss data from insurance companies can also be used to derive an appropriate probabilistic description of losses.

2.4 Faster Re-analysis Multi-layer MCS Method

The ordinary multi-layer MCS method proposed in the previous sections for risk assessment of residential buildings can be modified to achieve an improved computational efficiency when numerous performance assessment analyses are required for the same building (e.g., when comparing different design alternatives and hazard mitigation strategies). For this type of problems, the hazard and interaction analysis phases remain the same as long as the location and geometry of the building do not change. Under these conditions, the computational effort of the multi-layer MCS procedure can be significantly reduced by randomly generating the IPs based on their statistical description obtained from a first application of the multi-layer MCS technique (e.g., on an unmitigated structure), thus avoiding the repetition of the hazard and interaction analysis phases.

The statistical description of the IPs consists of the marginal probability distributions and the correlations between pairs of IPs. Thus, the random generation of the IPs requires the joint probability distribution of the random variables that describe the IPs. Different techniques are available in the literature to generate the joint probability distribution of random variables given their marginal distributions and correlations, e.g., the Chow-Liu tree (Chow and Liu 1968), the
Nataf transformation, and the copula approach (Nelsen 2007). In this study, the copula approach is adopted to model the joint probability distribution of the IPs in conjunction with the faster re-analysis multi-layer MCS method.

A copula is a multivariate joint distribution defined on the $n$-dimensional unit cube $[0, 1]^n$ such that every marginal distribution is uniform on the interval $[0, 1]$ (Sklar (1959), Nelsen (2007)). According to Sklar’s theorem (Sklar 1959), the multivariate joint cumulative distribution function (CDF) of $n$ random variables, $X_1,\ldots,X_n$, can be expressed as

$$F(X_1,\ldots,X_n) = C[F_1(X_1),\ldots,F_n(X_n)] = C(U_1,\ldots,U_n) \quad (2.8)$$

where $F(X_1,\ldots,X_n)$ = joint CDF of variables $X_1,\ldots,X_n$; $U_i = F_i(X_i)$ = marginal CDF of $X_i \ (i = 1,\ldots,n)$; and $C(U_1,\ldots,U_n)$ = copula function.

From Eq. 3.7, the joint probability distribution function (PDF) $f(x_1,\ldots,x_n)$ can be obtained as (Nelsen (2007), (Goda 2010))

$$f(x_1,\ldots,x_n) = f_1(x_1) \cdot \ldots \cdot f_n(x_n) \cdot c(U_1,\ldots,U_n)$$

$$c(U_1,\ldots,U_n) = \frac{\partial^n C(U_1,\ldots,U_n)}{\partial U_1 \ldots \partial U_n} \quad (2.9)$$

where $f_i(X_i)$ = marginal PDFs of $X_i \ (i = 1,\ldots,n)$; and $c(U_1,\ldots,U_n)$ = copula density function.

The joint CDF and PDF of $X_i \ (i = 1,\ldots,n)$ can be determined by Eq. (2.8) and Eq. (2.9) if their marginal distributions and the copula function are known. Different types of copulas can be used to describe the dependence between the random variables (Tang et al. 2013). In this study, a Gaussian copula is adopted to model the dependence between the variables. The investigation of the efficiency of different copulas in modeling the dependence structure of the variables, albeit important, is out of the scope of this study.
The IPs obtained from the interaction analysis are \( p_{w,j} \) for each building component, \( n_d \), and \( L_d \) and \( K_d \) for each impact. The wind pressure values depend on the velocity pressure, \( q_h \), and on the SPs through Eq. (2.3). Based on the results obtained from numerous applications of the multi-layer MCS method, it is assumed that, for a given wind velocity, both \( L_d \) and \( K_d \) follow a lognormal distribution, which is completely characterized by its mean and standard deviation (i.e., \( \mu_{L_d} \) and \( \sigma_{L_d} \) for \( L_d \), and \( \mu_{K_d} \) and \( \sigma_{K_d} \) for \( K_d \)).

![Modified multi-layer MCS approach for probabilistic hurricane loss estimation of non-engineered residential buildings requiring multiple re-analyses.](image)

These means and standard deviations are modeled as random variables, each described by an empirical CDF. It is further observed that the correlation coefficients between \( \mu_{L_d} \) and \( \mu_{K_d} \), and between \( \sigma_{L_d} \) and \( \sigma_{K_d} \) are very close to 1. Thus, a Gaussian copula function is generated for variables \( q_h \), \( n_d \), \( \mu_{L_d} \), and \( \sigma_{L_d} \), based on the marginal distributions and correlation coefficients obtained in the first application of the multi-layer MCS method. In the subsequent re-analyses, the hazard analysis and interaction analysis phases are substituted in the modified multi-layer MCS approach.
method by a sample generation step (see Figure 2.3), in which (1) variables $q_h, n_d, \mu_{t_q}$, and $\sigma_{t_q}$ are sampled from the joint probability distribution constructed using the previously obtained copula function; (2) for each of the $n_d$ impacts, variables $L_d$ and $K_d$ are sampled from the corresponding lognormal distributions with means and standard deviations $\mu_{t_q}$ and $\sigma_{L_q}$, and $\mu_{K_q}$ and $\sigma_{K_q}$, respectively; and (3) variables $GC_{p_j}$ and $GC_{p_{w,j}}$ are sampled for each building component and variables $p_{w,j}$ are obtained from Eq. (2.3).

2.5 Cost-Benefit Analysis

Cost-benefit analysis can be used to compare the cost of different storm mitigation techniques and the benefits achieved from improved performance of the building over its entire design life. Cumulative monetary damages or losses over a specific period of time are of interest to decision-makers and can be estimated based on the expected annual loss. The relationship between the cost of mitigation tactics and its benefits are explicitly quantified and thereby facilitate effective decision making for investment in the safety of buildings (Liel and Deierlein 2013). The expected present value of economic benefit of a hurricane mitigation technique (B) can be expressed as

$$B = \sum_{n=0}^{t} \frac{EAL_u - EAL_r}{(1 + \rho)^n} - C_r$$

(2.10)

after retrofit, $\rho$ = discount rate, $t$ = planning period and $C_r$ = cost of the retrofit. The expected annual loss (EAL) is defined as the average economic loss that occurs every year in the building (Raul and Vitelmo 2004) and is equal to the area under the corresponding annual probability of exceedance curve. The retrofit or redesign is financially viable if the corresponding expected value of economic benefit is greater than zero.
2.6 Case Study

A realistic case study of a single-family house subject to wind and windborne debris hazards is presented here to illustrate the proposed PBHE framework and to compare the costs and benefits of different storm mitigation techniques and/or design alternatives when applied to a base structure.

![Figure 2.4 Plan view of the residential development (source: Google Maps).](image)

The house is located in a residential development in Pinellas County, FL, which contains 201 similar gable roof wooden residential buildings (see Figure 2.4). The roof covers were considered as debris sources, whereas the walls, windows and doors were considered as debris impact vulnerable components. The value of the target structure was taken as $200,000 and the content value was assumed equal to $100,000.
2.6.1 Hazard analysis

The number of hurricanes per year was simulated using a Poisson occurrence model, with an annual hurricane occurrence rate $\nu_{\text{hurricane}} = 0.52$ obtained from the National Institute of Standards and Technology (NIST) database (NIST 2005). The 3-second wind speed ($V$) recorded at 10 m above the ground was adopted as $IM$ for wind hazard. The hurricane wind speed variability was described by using a two-parameter Weibull distribution with the following cumulative distribution function:

$$F(V) = 1 - \exp \left[ - \left( \frac{V}{a} \right)^b \right]$$  \hspace{1cm} (2.11)

The two shape parameters $a$ and $b$ are site and direction specific and were determined for sixteen different wind directions through maximum likelihood estimation of the hurricane wind speed records provided by NIST for milepost 1400 (see Table 2.1).

The NIST wind speed records contain data sets of simulated 1-minute hurricane wind speeds at 10 m above the ground in an open terrain near the coastline (NIST 2005). Before fitting, the wind speed data were multiplied by a factor equal to 0.89, to obtain the corresponding 3-second wind speeds for exposure category B (Lungu and Rackwitz 2001). For each generated hurricane event, the maximum 3-second wind speed was generated according to this fitted Weibull distribution.

Non-hurricane wind hazard was also considered in addition to hurricane wind hazard.

The daily maximum 3-second wind speeds at the building location were obtained from the Iowa Environmental Mesonet (IEM) database for the 1971-2013 period (IEM 2001). The historical hurricane tracks that passed within a 250 miles radius from the site during the same 1971-2013 period were obtained from the National Oceanic and Atmospheric Administration (NOAA) database and were used to separate the non-hurricane wind speeds from the hurricane wind speeds.
Table 2.1 Weibull shape parameters for different directions.

<table>
<thead>
<tr>
<th>Direction</th>
<th>$a$</th>
<th>$b$</th>
</tr>
</thead>
<tbody>
<tr>
<td>North</td>
<td>22.96</td>
<td>2.79</td>
</tr>
<tr>
<td>North–Northeast</td>
<td>22.11</td>
<td>2.69</td>
</tr>
<tr>
<td>Northeast</td>
<td>21.61</td>
<td>2.86</td>
</tr>
<tr>
<td>East–Northeast</td>
<td>21.24</td>
<td>2.9</td>
</tr>
<tr>
<td>East</td>
<td>20.79</td>
<td>2.68</td>
</tr>
<tr>
<td>East–Southeast</td>
<td>22.08</td>
<td>2.09</td>
</tr>
<tr>
<td>Southeast</td>
<td>23.34</td>
<td>2.38</td>
</tr>
<tr>
<td>South–Southeast</td>
<td>26.22</td>
<td>2.74</td>
</tr>
<tr>
<td>South</td>
<td>20.9</td>
<td>1.89</td>
</tr>
<tr>
<td>South–Southwest</td>
<td>19.68</td>
<td>2.12</td>
</tr>
<tr>
<td>Southwest</td>
<td>19.44</td>
<td>2.21</td>
</tr>
<tr>
<td>West–Southwest</td>
<td>19.37</td>
<td>2.13</td>
</tr>
<tr>
<td>West</td>
<td>18.01</td>
<td>1.67</td>
</tr>
<tr>
<td>West–Northwest</td>
<td>20.55</td>
<td>2.25</td>
</tr>
<tr>
<td>Northwest</td>
<td>24.59</td>
<td>2.83</td>
</tr>
<tr>
<td>North–Northwest</td>
<td>23.14</td>
<td>2.87</td>
</tr>
</tbody>
</table>

The yearly maximum non-hurricane 3-second wind speeds were then obtained and fitted to a lognormal distribution, with a mean of 18.34 m/s and standard deviation of 1.08 m/s.

The IMs considered for windborne debris hazard were area of debris, $A_d$, and mass per unit area of debris, $M_d$. They were assumed to follow uniform distributions defined in the range [0.108, 0.184] m² and [10.97, 14.97] kg/m², respectively (Gurley et al. 2005). The FPHLM debris generation model was used to simulate the number of debris originating from the source houses.
2.6.2 Description of the base structure and hazard mitigation techniques

The wind pressure exposure factor $K_h$ was assumed as normally distributed with a mean value of 0.71 and a coefficient of variation (COV) of 0.19. The topographic factor was modeled as a deterministic quantity with value $K_{zt} = 1$.

Table 2.2 shows the (deterministic) geometric parameters describing the target residential building (Gurley et al. 2005).

<table>
<thead>
<tr>
<th>Structural Parameter</th>
<th>Dimension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>60ft</td>
</tr>
<tr>
<td>Width</td>
<td>40ft</td>
</tr>
<tr>
<td>Height of wall</td>
<td>10ft</td>
</tr>
<tr>
<td>Roof Pitch</td>
<td>5/12</td>
</tr>
<tr>
<td>Eave overhang</td>
<td>2ft</td>
</tr>
<tr>
<td>Space between roof trusses</td>
<td>2ft</td>
</tr>
<tr>
<td>Roof sheathing panel dimension</td>
<td>8ft X 4ft</td>
</tr>
</tbody>
</table>

The position and dimension of the windows and doors of the target building are shown in Figure 2.5.

![Unfolded view of target building](image)

**Figure 2.5 Unfolded view of target building.**

The statistical characterization of the external and internal pressure coefficients is given in Table 2.3 (Li and Ellingwood 2006).
Table 2.3 Statistical characterization of external and internal pressure coefficients.

<table>
<thead>
<tr>
<th>Location/Condition</th>
<th>$G_{Cp}$</th>
<th>$G_{Cpi}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof (zone 1)</td>
<td>-0.855</td>
<td>0.150</td>
</tr>
<tr>
<td>Roof (zone 2)</td>
<td>-1.615</td>
<td>0.460</td>
</tr>
<tr>
<td>Roof (zone 3)</td>
<td>-2.47</td>
<td></td>
</tr>
<tr>
<td>Windward wall</td>
<td>0.950</td>
<td></td>
</tr>
<tr>
<td>Leeward wall</td>
<td>-0.76</td>
<td></td>
</tr>
<tr>
<td>Side wall</td>
<td>-1.045</td>
<td></td>
</tr>
<tr>
<td>Enclosed</td>
<td>0.150</td>
<td></td>
</tr>
<tr>
<td>Breached</td>
<td>0.460</td>
<td></td>
</tr>
</tbody>
</table>

The base structure is characterized by (1) roof cover made of shingles, (2) nailing pattern 8d C6/12 (i.e., 8 mm diameter smooth shank nails, with a spacing of 6 inches at the center and 12 inches at the edge) for the roof sheathing, (3) unprotected windows and doors, and (4) wooden walls. The statistics of the limit state capacity for the different components of the base building and their corresponding limit states are shown in Table 2.4 (Gurley et al. 2005, Datin et al. 2010, Masters et al. 2010).

The following storm mitigation techniques and design alternatives are considered: (1) using clay tiles as roof cover instead of asphalt shingles; (2) using an improved roof nailing pattern of 8d C6/6 (i.e., 8 mm diameter smooth shank nails, with a spacing of 6 inches) or 8d R6/6 (i.e., 8 mm diameter ring shank nails, with a spacing of 6 inches) instead of the traditional 8d C6/12 pattern;
(3) using aluminum hurricane protection panels for windows; and (4) using masonry walls instead of wooden walls.

Table 2.4 Statistics of the limit state capacity for different components.

<table>
<thead>
<tr>
<th>Component</th>
<th>Limit state</th>
<th>Mean</th>
<th>COV</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof cover (Shingles)</td>
<td>Separation or pull off ($R_{cover1}$)</td>
<td>3.35 kN/m²</td>
<td>0.19</td>
<td>Normal</td>
</tr>
<tr>
<td>Roof sheathing (Nailing pattern 8d C6/12)</td>
<td>Separation or pull off ($R_{sh1}$)</td>
<td>6.20 kN/m²</td>
<td>0.12</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Doors</td>
<td>Pressure failure ($R_{door}$)</td>
<td>4.79 kN/m²</td>
<td>0.20</td>
<td>Normal</td>
</tr>
<tr>
<td>Windows</td>
<td>Pressure failure ($R_{w, pressure}$)</td>
<td>3.33 kN/m²</td>
<td>0.20</td>
<td>Normal</td>
</tr>
<tr>
<td></td>
<td>Impact failure ($R_{w, impact}$)</td>
<td>4.72 kg/m/s</td>
<td>0.23</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Wall sheathing</td>
<td>Pressure failure ($R_{wsh, pressure}$)</td>
<td>6.13 kN/m²</td>
<td>0.40</td>
<td>Normal</td>
</tr>
<tr>
<td></td>
<td>Impact failure ($R_{wsh, impact}$)</td>
<td>642.00 kg m²/s²</td>
<td>0.07</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Roof to wall connections (Wood)</td>
<td>Tensile failure ($R_{wcon, wood}$)</td>
<td>16.28 kN</td>
<td>0.20</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Wall (Wood)</td>
<td>Lateral Failure ($R_{wall, wt}$)</td>
<td>5.40 kN/²</td>
<td>0.25</td>
<td>Normal</td>
</tr>
<tr>
<td></td>
<td>Uplift Failure ($R_{wall, wu}$)</td>
<td>9.00 kN/m²</td>
<td>0.25</td>
<td>Normal</td>
</tr>
</tbody>
</table>

* Toe nail connection  ** Sheathing nail connection

The statistics of the limit state capacity for the different storm mitigation techniques and design alternatives, as well as their corresponding limit states are shown in Table 2.5 (Gurley et al. 2005, Datin et al. 2010, Alphonso and Barbato 2014). The combination of different storm mitigation techniques and design alternatives were considered, giving a total of 24 configurations (i.e., Case #1 through Case #24) including the base structure (corresponding to Case #1).
Table 2.5 Statistics of the limit state capacity for different storm mitigation techniques and design alternatives.

<table>
<thead>
<tr>
<th>Component</th>
<th>Limit state</th>
<th>Mean</th>
<th>COV</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof cover (Tiles)</td>
<td>Separation or pull off (R\text{cover}_2)</td>
<td>5.25 kN/m²</td>
<td>0.20</td>
<td>Normal</td>
</tr>
<tr>
<td>Roof sheathing (Nailing pattern 8d C6/6)</td>
<td>Separation or pull off (R\text{sh}_2)</td>
<td>9.83 kN/m²</td>
<td>0.10</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Roof sheathing (Nailing pattern 8d R6/6)</td>
<td>Separation or pull off (R\text{sh}_3)</td>
<td>12.08 kN/m²</td>
<td>0.07</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Windows with hurricane panels</td>
<td>Impact failure (R\text{panel, impact})</td>
<td>12.70 cm</td>
<td>0.15</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Roof to wall connections (Masonry)</td>
<td>Tensile failure (R\text{wall, masonry})</td>
<td>18.68 kN</td>
<td>0.20</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Wall (Masonry)</td>
<td>Combined uplift and bending failure (R\text{wall, masonry})</td>
<td>18.00 kN 1.31 kN m</td>
<td>0.20</td>
<td>Normal</td>
</tr>
</tbody>
</table>

The total loss during a 30-year design lifetime for the building (given by the sum of the repair cost and the content loss) was assumed as $DV$. The repair costs of each damaged component were generated based on a lognormal distribution, with mean given by the percentage of damage of the given component multiplied by its total cost expressed as a percentage of the building cost according to the values shown in Table 2.6 for each sub-assembly (i.e., set of components of the same type within a building), and COV equal 0.1 (Gurley et al. 2005).

The content loss was estimated using the approach followed in HAZUS-MH (FEMA 2012), i.e., by using empirical functions that express the content loss associated with the damage of each individual component as a percentage of the total value of the content. The content loss was sampled from a lognormal distribution with mean equal to the highest loss estimate obtained from the HAZUS-MH content loss functions and COV equal to 0.1 (FEMA 2012). The total loss was
calculated by adding up all the losses due to the damage of various components and the content damage.

Table 2.6 Sub-assembly cost ratios.

<table>
<thead>
<tr>
<th>Sub-assembly</th>
<th>Average cost (% of total building cost)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site work</td>
<td>1%</td>
</tr>
<tr>
<td>Foundation</td>
<td>13%</td>
</tr>
<tr>
<td>Exterior wall</td>
<td>22%</td>
</tr>
<tr>
<td>Framing</td>
<td>8%</td>
</tr>
<tr>
<td>Roof sheathing</td>
<td>5%</td>
</tr>
<tr>
<td>Roof covers</td>
<td>7%</td>
</tr>
<tr>
<td>Interiors</td>
<td>40%</td>
</tr>
<tr>
<td>Windows and doors</td>
<td>4%</td>
</tr>
<tr>
<td>Contents</td>
<td>50%</td>
</tr>
</tbody>
</table>

In order to accurately estimate the annual probability of exceedance of the total loss (which coincides with the complementary cumulative distribution function of the $DV$), 100,000 samples were used for all results presented in this study. Three sets of results are presented here: (1) the hurricane loss analysis for the base structure; (2) the validation of the proposed faster re-analysis method; and (3) the cost/benefit comparison of different storm mitigation strategies and design alternatives.

2.6.3 Loss analysis results for the base structure

Figure 2.6 plots, in a semi-logarithmic scale, the annual probabilities of exceedance of the loss for the target building for different hazard scenarios. It also provides the EAL and standard deviation of loss (SDL) for each of the hazard scenarios considered.
Figure 2.6 Annual probabilities of loss exceedance for base building under different hazard scenarios.

From the results presented in Figure 2.6, it is observed that for hurricane induced losses, the loss due to windborne debris hazard is predominant for losses lower than about $15,000, whereas the loss due to wind hazard is predominant for losses higher than about $15,000. This result is due to the fact that, at lower wind speeds, the probability of damage to the windows due to windborne debris is lower than that due to wind pressure. For non-hurricane winds, the loss due to wind hazard is predominant, while the loss due to windborne debris is negligible (i.e., zero loss over the 100,000 samples), because for non-hurricane winds the number of generated windborne debris and, thus, the number of debris impact is generally very small. It is also observed that the EAL due to the interaction of all hazards is about 15% higher than the sum of the EALs due to each individual hazard. This result suggests a significant level of interaction among the different hazards for the case study considered here. In addition, it is observed that for all the hazard scenarios, the SDL is
significantly higher than the EAL, which indicates that the annual loss is characterized by a high dispersion. Therefore, the EAL is not sufficient alone to describe the loss analysis results.

### 2.6.4 Validation of the faster re-analysis multi-layer MCS procedure

In order to validate the newly proposed faster re-analysis multi-layer MCS procedure, the hurricane loss analysis for the base structure (Case #1) was repeated ten times using both the original and faster re-analysis multi-layer MCS procedures. The results from the different runs were compared in terms of annual probabilities of loss exceedance (which are plotted in Figure 3.7), as well as of EAL and SDL (which are reported for each run in Table 2.7, together with their sample statistics).

<table>
<thead>
<tr>
<th>Run</th>
<th>Original EAL</th>
<th>Original SDL</th>
<th>Faster re-analysis EAL</th>
<th>Faster re-analysis SDL</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$1,291</td>
<td>$13,571</td>
<td>$1,283</td>
<td>$13,372</td>
</tr>
<tr>
<td>2</td>
<td>$1,287</td>
<td>$13,330</td>
<td>$1,290</td>
<td>$13,498</td>
</tr>
<tr>
<td>3</td>
<td>$1,275</td>
<td>$13,265</td>
<td>$1,285</td>
<td>$13,362</td>
</tr>
<tr>
<td>4</td>
<td>$1,287</td>
<td>$13,470</td>
<td>$1,266</td>
<td>$13,129</td>
</tr>
<tr>
<td>5</td>
<td>$1,292</td>
<td>$13,557</td>
<td>$1,275</td>
<td>$13,315</td>
</tr>
<tr>
<td>6</td>
<td>$1,280</td>
<td>$13,219</td>
<td>$1,276</td>
<td>$13,241</td>
</tr>
<tr>
<td>7</td>
<td>$1,276</td>
<td>$13,284</td>
<td>$1,274</td>
<td>$13,300</td>
</tr>
<tr>
<td>8</td>
<td>$1,269</td>
<td>$13,182</td>
<td>$1,288</td>
<td>$13,471</td>
</tr>
<tr>
<td>9</td>
<td>$1,277</td>
<td>$13,296</td>
<td>$1,271</td>
<td>$13,210</td>
</tr>
<tr>
<td>10</td>
<td>$1,273</td>
<td>$13,334</td>
<td>$1,282</td>
<td>$13,299</td>
</tr>
<tr>
<td>Mean</td>
<td>$1,281</td>
<td>$13,351</td>
<td>$1,279</td>
<td>$13,320</td>
</tr>
<tr>
<td>St. Dev.</td>
<td>$8</td>
<td>$136</td>
<td>$8</td>
<td>$113</td>
</tr>
<tr>
<td>Confidence Interval (95%)</td>
<td>$1,286</td>
<td>$13,448</td>
<td>$1,285</td>
<td>$13,400</td>
</tr>
<tr>
<td></td>
<td>$1,274</td>
<td>$13,253</td>
<td>$1,273</td>
<td>$13,239</td>
</tr>
</tbody>
</table>
Figure 2.7 Comparison of original and faster re-analysis multi-layer MCS approaches.

From the results presented in Figure 2.7 it is observed that the annual probability of exceedance curves obtained using the proposed re-analysis approach based on copula are similar to those obtained using the original multi-layer MCS method, with a variability between the different repetitions of the two methods that is very close to the variability observed among different repetitions obtained from the same method. Additionally, a 95% confidence interval was calculated for the sample mean and standard deviation for the original and faster re-analysis multi-layer MCS method.

It was found that the sample mean and standard deviation for the faster re-analysis multi-layer MCS method are within the confidence interval for the sample mean and standard deviation of the original method, and vice-versa. Thus, it was concluded that the difference between the mean and standard deviations of the two sets of samples is not statistically significant. Hence, the proposed faster re-analysis approach can be used for problems that require risk re-assessment.
2.6.5 Cost/benefit comparison of different hazard mitigation techniques

The annual probabilities of loss exceedance for the base structure and each of the 23 mitigation scenarios considered in this study were calculated using the faster re-analysis multi-layer MCS method. Some of these curves are shown in Figure 2.8 using a semi-logarithmic scale, together with the corresponding EAL and SDL. A cost-benefit analysis was carried out to compare the cost effectiveness of different retrofit techniques and design alternatives. In this study, discount rate and planning period were assumed as 3% and 30 years, respectively. The cost of retrofit includes the cost of the materials and the cost for the installation of the retrofits and was obtained as the mean values of the quotes obtained by directly contacting several local suppliers and contractors.

![Figure 2.8 Annual probability of loss exceedance for different hazard mitigation scenarios.](image)

Figure 2.8 Annual probability of loss exceedance for different hazard mitigation scenarios.
Figure 2.9 Savings for each considered mitigation scenario.

Table 2.8 provides the EAL and SDL, cost of retrofit, discounted mean loss in 30 years, and discounted expected savings in 30 years for each mitigation scenario when compared to the base structure. Figure 2.9 summarizes the results of the cost/benefit analysis in terms of discounted expected savings in 30 years for all mitigation scenarios.

From the results presented in Table 2.8 and Figure 2.9, it is observed that roof re-nailing using 8d R6/6 can result in an overall savings of $12,472 and is the most effective solution to reduce hurricane risk among the mitigation techniques considered in this study. Similarly, the use of aluminum panels for window protection can provide savings of about $5,000. The design alternative of using masonry or the use of clay roof tiles is not a financially viable approach to reduce hurricane risk. In addition, the combination of aluminum storm panels and improved roof nailing pattern can reduce considerably the expected total loss due to hurricanes, resulting in savings of about $15,000.
Table 2.8 Risk assessment of different retrofit scenarios.

<table>
<thead>
<tr>
<th>Mat.</th>
<th>Window protection</th>
<th>Roof cover</th>
<th>Roof nailing pattern</th>
<th>Case #</th>
<th>Loss analysis</th>
<th>Cost/benefit analysis</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>EAL</td>
<td>SDL</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wood</td>
<td>No</td>
<td>Shingles</td>
<td>8d C6/12</td>
<td>1</td>
<td>$1,287</td>
<td>$13,330</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8d C6/6</td>
<td>2</td>
<td>$394</td>
<td>$6,656</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8d R6/6</td>
<td>3</td>
<td>$372</td>
<td>$6,550</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Shingles</td>
<td>8d C6/12</td>
<td>4</td>
<td>$1,184</td>
<td>$13,286</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8d C6/6</td>
<td>5</td>
<td>$379</td>
<td>$6,559</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8d R6/6</td>
<td>6</td>
<td>$363</td>
<td>$6,507</td>
</tr>
<tr>
<td></td>
<td>Yes</td>
<td>Shingles</td>
<td>8d C6/12</td>
<td>7</td>
<td>$957</td>
<td>$12,639</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8d C6/6</td>
<td>8</td>
<td>$170</td>
<td>$5,011</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8d R6/6</td>
<td>9</td>
<td>$130</td>
<td>$4,451</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Shingles</td>
<td>8d C6/12</td>
<td>10</td>
<td>$901</td>
<td>$12,201</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8d C6/6</td>
<td>11</td>
<td>$151</td>
<td>$4864</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8d R6/6</td>
<td>12</td>
<td>126</td>
<td>$4395</td>
</tr>
<tr>
<td></td>
<td>No</td>
<td>Tiles</td>
<td>8d C6/12</td>
<td>13</td>
<td>$1,093</td>
<td>$13,069</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8d C6/6</td>
<td>14</td>
<td>$291</td>
<td>$5,627</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8d R6/6</td>
<td>15</td>
<td>$278</td>
<td>$5,499</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Tiles</td>
<td>8d C6/12</td>
<td>16</td>
<td>$1003</td>
<td>$13,010</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8d C6/6</td>
<td>17</td>
<td>$281</td>
<td>$5,528</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8d R6/6</td>
<td>18</td>
<td>$263</td>
<td>$5,392</td>
</tr>
<tr>
<td></td>
<td>Masonry</td>
<td>Shingles</td>
<td>8d C6/12</td>
<td>19</td>
<td>$888</td>
<td>$12,115</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8d C6/6</td>
<td>20</td>
<td>$100</td>
<td>$4,399</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8d R6/6</td>
<td>21</td>
<td>$90</td>
<td>$4,112</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Shingles</td>
<td>8d C6/12</td>
<td>22</td>
<td>$871</td>
<td>$12,064</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8d C6/6</td>
<td>23</td>
<td>$81</td>
<td>$3,870</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8d R6/6</td>
<td>24</td>
<td>$76</td>
<td>$3,747</td>
</tr>
</tbody>
</table>
2.7 Conclusions

In this chapter, the Performance-Based Hurricane Engineering (PBHE) framework is specialized for hurricane risk assessment of low-rise residential buildings. The focus of this chapter is on the hurricane loss analysis of residential buildings and the effects of mitigation techniques for wind and windborne debris hazards on the structural performance. The problem of risk assessment is disaggregated into the following basic probabilistic components: (1) hazard analysis, (2) structural characterization, (3) interaction analysis, (4) structural analysis, (5) damage analysis, and (6) loss analysis. A highly efficient modification of the multi-layer Monte Carlo simulation (MCS) technique based on copula is proposed for faster re-evaluation of hurricane risk. The proposed faster re-analysis multi-layer MCS method is used in conjunction with cost/benefit analysis to compare different hazard mitigation technique and design alternative.

A realistic case study consisting of an actual residential development located in Pinellas County, FL, is presented to illustrate the framework. The annual probabilities of exceedance of the loss for the target building for different hazard scenarios are calculated. It is found that for hurricane induced loss, the loss due to windborne debris hazard is predominant for lower loss levels, whereas the loss due to wind hazard is predominant for higher loss levels; and for non-hurricane induced loss, windborne debris hazard is negligible. The proposed faster re-analysis approach is validated based on the corresponding results obtained using the original multi-layer MCS. The cost-effectiveness of different hurricane hazard mitigation techniques and design alternatives typically used for low-rise residential buildings are compared. For the specific application example considered here, it is observed that, among the different types of retrofits compared in this study, the most economically viable form of retrofit is the use of roof re-nailing with an 8d R6/6 pattern and the least is the use of masonry walls.
It is concluded that the PBHE methodology, in conjunction with the faster re-analysis multi-layer MCS method proposed here and cost/benefit analysis, can be effectively used to improve the design or select appropriate hurricane hazard mitigation techniques for a specific low-rise residential building. It is noteworthy that the presented probabilistic methodology differs from the HAZUS-MH approach because it is concerned with the design and/or retrofit of specific buildings and structures, whereas HAZUS-MH focuses on loss analysis at a regional level.

2.8 References


CHAPTER 3
PERFORMANCE-BASED HURRICANE RISK ASSESSMENT OF TALL BUILDINGS

3.1 Introduction

In recent years, performance-based engineering (PBE) approaches have been receiving significant attention by researchers in wind and hurricane engineering. In particular, the performance of high-rise buildings under wind actions is crucial in driving the design, and a probabilistic risk assessment analysis becomes necessary to ensure appropriate serviceability and safety in combination with an economic design. A coherent evaluation of performance in monetary terms can be used to design optimal structural systems that maintain an acceptable performance during their whole life cycle. Thus, a PBE approach can be very beneficial in the analysis and design of this building typology.

Although PBE approaches have been vigorously adopted in the field of earthquake engineering and other sub-fields, they have been only recently introduced in wind and hurricane engineering (Petrini 2009, Smith and Caracoglia 2011, Barbato et al. 2013, Spence and Kareem 2014). PBE approaches were used to investigate the performance of low-rise buildings, in which damage and collapse were related to localized loss of capacity in key members or connections (Ellingwood and Tekie 1999, Ellingwood and Rosowsky 2004). Earlier studies on PBE of high-rise buildings developed a framework for the analysis of uncertainty (Bashor and Kareem 2007) and a methodology for the design of buildings (Jain et al. 2001, Norton et al. 2008). Bashor and Kareem (2007) developed a probabilistic framework to evaluate the performance of tall buildings in terms of occupants’ comfort. The random variables considered were the wind speed and structural damping. Reliability analyses based on the First-Order Reliability Method (FORM) and Monte-Carlo simulation (MCS) were used to assess the probability of failure, i.e., the probability of
occupants’ discomfort. Augusti and Ciampoli (2008) and Petrini (2009) developed a performance-based wind engineering (PBWE) framework by extending the performance-based earthquake engineering (PBEE) approach proposed by Pacific Earthquake Engineering Research Center (PEER). Smith and Caracoglia (2011) proposed a numerical algorithm for the simulation of the along-wind dynamic response of tall buildings under turbulent winds. The proposed algorithm was further used to find the statistical characterization of comfort criteria for a hypothetical tall office building. Barbato et al. (2013) developed a probabilistic Performance-Based Hurricane Engineering (PBHE) framework based on the total probability theorem, which can be used for the risk assessment and loss analysis of structural systems subject to hurricane hazard. This framework considered the multi-hazard nature of hurricane events, the interaction of different hazard sources, and the possible sequential effects of these distinct hazards.

As demonstrated by the existing technical literature, in addition to strength-based safety design considerations, several serviceability design performance objectives need to be considered in the design of tall buildings, e.g., satisfying serviceability design requirements in terms of wind-induced lateral deflection and acceleration (Huang et al. 2012); limiting the probability of discomfort of the occupants due to wind-induced vibrations (ISO 2003, Bernardini et al. 2014); ensuring the integrity of cladding under extreme wind (Kareem 1986, Baker 2007, Bashor et al. 2012); and minimizing non-structural damage such as damage to partitions, building’s content, plumbing system, electrical system, heating, ventilation, and air conditioning (Griffis 2003).

In this paper, a rigorous procedure based on the general PBHE framework was developed to perform the loss analysis for high rise buildings by considering both hurricane and regular wind hazards. Well-established models were employed to perform hazard, structural, and interaction analyses; whereas models used in HAZUS® were adopted for damage and loss evaluations. An
application example consisting of the performance assessment of a tall building subjected to both hurricane and non-hurricane wind hazard is presented to illustrate the proposed procedure.

3.2 Summary of PBHE framework

The PBHE framework proposed in Barbato et al. (2013) disaggregates the performance assessment procedure for structures subject to hurricane hazard into elementary phases that are carried out in sequence. The structural risk within the PBHE framework is expressed by the probabilistic description of a decision variable, \( DV \), which is defined as a measurable quantity that describes the cost and/or benefit for the owner, the users, and/or the society resulting from the structure under consideration. The fundamental relation for the PBHE framework is given by:

\[
G(DV) = \int \int \int G(DV|DM) \cdot f(DM|EDP) \cdot f(EDP|IM,IP,SP) \cdot f(IP|IM,SP) \cdot f(IM) \cdot f(SP) \cdot dDM \cdot dEDP \cdot dIP \cdot dIM \cdot dSP
\]

where \( G(\cdot) = \) complementary cumulative distribution function, and \( G(\cdot|\cdot) = \) conditional complementary cumulative distribution function; \( f(\cdot) = \) probability density function, and \( f(\cdot|\cdot) = \) conditional probability density function; \( IM = \) vector of intensity measures (i.e., parameters characterizing the environmental hazard); \( SP = \) vector of structural parameters (i.e., parameters describing the relevant properties of the structural system and non-environmental actions); \( IP = \) vector of interaction parameters (i.e., parameters describing the interaction phenomena between the environment and the structure); \( EDP = \) vector of engineering demand parameters (i.e., parameters describing the structural response for the performance evaluation); and \( DM = \) vector of damage measures (i.e., parameters describing the physical damage to the structure). By means of Eq.(3.1), the risk assessment analysis is disaggregated into the following tasks: (1) hazard analysis, (2) structural characterization, (3) interaction analysis, (4) structural analysis, (5) damage analysis, and (6) loss analysis.
3.3 General PBHE procedure for engineered buildings

Eq. (3.1) can be solved using different techniques, e.g., closed-form analytical solutions (Shome and Cornell 1999, Jalayer and Cornell 2003, Mackie et al. 2007, Zareian and Krawinkler 2007), direct integration techniques (Bradley et al. 2009), and stochastic simulation techniques (Porter and Kiremidjian 2001, Au and Beck 2003, Lee and Kiremidjian 2007). In PBHE, analytical solutions and direct integration techniques require the knowledge of the joint probability density function of the component losses, which is usually very difficult to obtain for real-world applications. Thus, in this study, a multi-layer MCS technique (Conte and Zhang 2007) was adopted and specialized to efficiently perform loss analysis for tall buildings subject to hurricane and wind hazard. The result of the PBHE equation (Eq. (3.1)) is the annual loss curve, $G(DV)$, i.e., the complementary cumulative distribution function of the annual losses for the tall building under consideration due to wind hazards.

Figure 3.1 shows the flowchart of the general multi-layer MCS technique applied to PBHE considering a one-year time interval. The multi-layer MCS technique allows to take into account the uncertainties from all phases of the PBHE framework (namely, hazard analysis, structural characterization, interaction analysis, structural analysis, damage analysis, and loss analysis). Each of these analysis phases is performed in two step: (1) a sample generation step of random parameters with known probability distributions, which are needed to describe the uncertainties in environmental actions, structural properties, interaction phenomena, analysis techniques, and cost estimates; and (2) an analysis step based on a deterministic model, which is used to propagate the uncertainties from input to output parameters of each analysis phase.
In particular, this study focuses on hurricane loss analysis for tall buildings. Figure 3.2 shows the flowchart for the multi-layer MCS technique specialized for probabilistic hurricane loss estimation of tall buildings, and provides the list of analysis parameters involved in each analysis phase.

Figure 3.1 Multi-layer MCS approach for PBHE framework

Figure 3.2 Multi-layer MCS approach for probabilistic hurricane loss estimation of tall buildings
3.3.1 Hazard analysis

The focus of this paper is on the risk assessment of tall buildings subjected to wind hazards. It is noted here that the general multi-layer MCS methodology presented in this study can include the effects on the structural performance also of windborne debris, flood, and rainfall hazards. However, this inclusion for loss analysis of tall buildings is outside the scope of this chapter.

The first step in the proposed multi-layer MCS approach is the simulation of the number of hurricanes affecting the considered structure in a given year, e.g., according to a Poisson occurrence model (Russel 1971, Chouinard and Liu 1997, Elsner and Kara 1999). For each of these hurricanes, a corresponding wind field needs to be simulated in order to characterize the wind hazard. Three methodologies of increasing accuracy and computational cost can be adopted to define the hurricane wind field (FEMA 2007): (1) deriving the statistical description of the 3-second gust wind velocity, \( V \), at the building site from existing peak wind speed data (Batts et al. 1980, Peterka and Shahid 1998, Li and Ellingwood 2006); (2) using site specific statistics of fundamental hurricane parameters to obtain a mathematical representation of a hurricane at the building location, including the statistics of the wind speed (Batts et al. 1980, Vickery and Twisdale 1995); and (3) modeling the full track of a hurricane from its initiation over the ocean until final dissipation and using appropriate wind field models to obtain the wind speed statistics corresponding to the specified track at the building site (Vickery et al. 2000).

In this paper, the first methodology (i.e., using existing peak wind speed data at the buildings site to derive the statistical description of the 3-second gust wind velocity) is adopted in order to reduce the computational cost of the proposed procedure. However, for important structures, one of the more accurate procedures may be more appropriate. It is also noteworthy that, when the number of hurricanes per year is equal to zero and only wind hazard is considered, the proposed PBHE
framework reduces to the PBWE framework proposed in Petrini (2009), and can be used to assess the performance of structure subject to non-hurricane wind actions. When the number of hurricanes per year is larger than zero, the procedure shown in Figure 3.2 is always performed first for non-hurricane wind actions, and then repeated for hurricane actions a number of times equal to the simulated number of hurricanes.

In this study, the horizontal dimensions of the building are considered to be sufficiently small that the horizontal variability of the wind speed can be neglected. The three components of the wind velocity field at a given floor \( j \) are denoted as \( V_u(z_j) \), \( V_v(z_j) \), and \( V_w(z_j) \), respectively, where the subscripts \( u, v, w \) represent the along wind, across wind, and vertical directions, respectively; and \( z_j \) denotes the vertical quote of floor \( j \) measured from the ground, with \( j = 1, 2, \ldots, N_f \), where \( N_f \) denotes the total number of floors of the building. These three components can be expressed as the sum of a mean (time-invariant) value \( V_m(z_j) \) and a turbulent component \( v_u(z_j), v_v(z_j), v_w(z_j) \), having mean value equal to zero. Assuming that the mean value of the velocity is different than zero only in the \( x \)-direction, the three components of the velocity are given by:

\[
V_u(z_j) = V_m(z_j) + v_u(z_j); \quad V_v(z_j) = v_v(z_j); \quad V_w(z_j) = v_w(z_j)
\]  

(3.2)

The variation of the mean velocity \( V_m \) with the height \( z \) over a horizontal surface of homogeneous roughness can be described by a power law as (Simiu and Scanlan 1978):

\[
V_m(z_j) = V_{10} \cdot \left( \frac{z_j}{10} \right)^\alpha
\]  

(3.3)
where $V_{10}$ is the mean velocity of wind averaged over a time interval of 10 minutes and measured at an elevation of 10 m above ground, and $\alpha$ is a site-dependent parameter.

The turbulent components of the wind velocity are modelled as zero-mean Gaussian ergodic independent processes (Ciampoli and Petrini 2012). Only the random spatial variation with the height $z$ is taken into account by considering the wind acting on $N$ vertically aligned points. The vertical component of the turbulence, $v_w$, can be neglected and the turbulent components $u_v$ and $v_v$ are completely characterized by their power spectral density (PSD) matrices $[S_{uv}], (l = u,v)$ (Carassale and Solari 2006). The diagonal terms (auto-spectra) $S_{vv}^{(j,j)}(n)$ of $[S_{vv}]$ ($j = 1,2,...,N_j$) are expressed by the following normalized one-sided PSD functions (Solari and Picardo 2001):

\[
\frac{n \cdot S_{vv}^{(j,j)}(n)}{\sigma_{vv}^2} = \frac{6.868n_v(z_j)}{[1+10.302n_u^2(z_j)]^{5/3}}
\]

\[
\frac{n \cdot S_{uv}^{(j,j)}(n)}{\sigma_{uv}^2} = \frac{9.434n_v(z_j)}{[1+14.15n_v^2(z_j)]^{5/3}}
\]

where $n$ is the current wind frequency (in Hz), $z_j$ is measured in meters, $\sigma_{vv}^2$ and $\sigma_{uv}^2$ are the variances of the velocity fluctuations, which can be assumed independent on $z_j$ and are given by (Solari and Picardo 2001):

\[
\sigma_{uv}^2 = 6 - 1.1 \arctan \left[ \ln(z_o) + 1.75 \right] \cdot u_v^2
\]

\[
\frac{\sigma_{uv}}{\sigma_{vv}} = 0.75
\]

where $u_v$ is the friction or shear velocity (in m/s), given by $\sqrt{(K)_{10} \cdot V_{10}}$, where $K$ is a coefficient.
depending on the roughness length \( z_0 \); \( n_l(z_j) \) \((l = u,v)\), is a non-dimensional height-dependent frequency given by \( n_l(z_j) = \frac{n \cdot L_l(z_j)}{V_m(z_j)} \), and the integral length scales \( L_l(z_j) \) of the turbulent components can be derived for \( l = u, v \) as (Carassale and Solari 2006):

\[
L_l(z_j) = L_1 \left( \frac{z_j}{z_i} \right)^{0.67 + 0.05\ln(z_i)}
\]  

(3.6)

where \( L_1 \) is the reference integral length scale and \( z_i \) is the reference height.

The non-diagonal terms (cross-spectra) \( S_{lj}^{jk}(n) \) \((j,k = 1,2,\ldots,N_f)\) of \( [S_{lj}] \) \((l = u,v)\) are given by

\[
S_{lj}^{jk}(n) = \sqrt{S_{jji}^{lj}(n) \cdot S_{jki}^{lj}(n)} \cdot \exp[-f^{(j,k)}(n)]
\]  

(3.7)

where, for vertically aligned points, \( f^{(j,k)}(n) = \frac{|n| \cdot C_z \cdot |z_j - z_k|}{V_m(z_j) + V_m(z_k)} \) (Di Paola 1998), and \( C_z \) is a decay coefficient that is inversely proportional to the spatial correlation of the process.

### 3.3.2 Structural characterization

The structural characterization phase provides the probabilistic description of the \( SP \) vector, which includes the random structural properties that can influence the loading applied to the structure and/or its components through the \( IP \) vector. These properties can include, e.g., geometrical properties, such as position and dimensions of openings as well as the dimensions of the building; mechanical properties, such as natural period and damping; and other parameters that determine the intensity of the wind effects on the structure and its components, such as pressure coefficients and gust effect factor. Geometrical properties can usually be treated as deterministic quantities, since they can be directly measured for existing structures or are characterized by a small
variability. It is noteworthy that the statistical characterization of the SP vector usually changes any time the building envelope is breached or there is damage to the structural and non-structural components. Thus, it is important to account for these changes in order to properly evaluate the effects of hazard chains (Barbato et al. 2013). The statistical characterization of the other parameters affecting the intensity of the wind effects can be obtained through wind tunnel tests or from appropriate statistical distributions available in the literature. In this study, the following random structural parameters are considered: circular frequency, \( \omega_q = 2\pi n_q \) (where \( n_q \) denotes the natural frequency in Hz), and viscous damping ratio, \( \xi_q \), corresponding to the \( q \)-th vibration mode; exposed wind tributary area for the \( j \)-th floor, \( A_t^{(j)} \); gust effect factor, \( G \); external pressure coefficient, \( C_p \); and internal pressure coefficient, \( C_{pi} \).

### 3.3.3 Interaction analysis

The choice of the IP vector is crucially dependent on the hazard sources, limit states, and performance levels of interest for both structural and non-structural elements. In this study, the IP vector is selected to represent the effects of wind hazard on the different limit states of interest for tall buildings.

The interaction parameter considered are the aerodynamic coefficients of drag, \( C_d \), and lift, \( C_L \).

The statistical characterization of these coefficients can be obtained through wind tunnel tests or from appropriate statistical distributions available in the literature. The interaction analysis for the wind hazard provides the statistical characterization of the wind force acting on each floor of the tall building in both the along and across wind characterized by their respective PSD matrices (i.e., \( \left[ S_{F_lF_l} \right], l = u, v \)). The cross-PSD matrix of the along wind force is given as

\[
S^{(j,k)}_{F_lF_l}(n) = A^{(j)} \cdot S^{(j,k)}_{F_lF_l}(n) \cdot A^{(k)} \quad (j, k = 1, 2, \ldots, N_f)
\]  

(3.8)
where
\[ A^{(i)} = \rho \cdot C_\alpha \cdot Ar^{(j)} \cdot V_m(z_j) \]  
(3.9)
and \( \rho \) is the density of air.

The across wind force consists of two components, the first one due to the turbulence effect, and the second one due to vortex shedding. The diagonal terms of the across wind force due to vortex shedding is given as (Liang et al. 2002):
\[
S_{F,F}^{(j,j)}(n) = \frac{\sigma^2_j}{n} \left( A \frac{H(C_1)\bar{n}^2}{(1-\bar{n}^2)^2 + C_1\bar{n}^2} + (1-A) \frac{C_2^{0.50}\bar{n}^3}{1.56(1-\bar{n}^2)^2 + C_2\bar{n}^2} \right)
\]
(3.10)
where \( \sigma_j \) is the root mean square of the across wind force at floor \( j \), \( \sigma_j = \frac{1}{2} \rho \cdot V_m^2(z_j) \cdot \bar{C}_L \cdot B \), \( B \) is the width of the building, \( \bar{C}_L \) is mean of the lift coefficient, \( A \) is the power-assigmentation coefficient, which is given by:
\[
A = \frac{H}{\sqrt{S}} \left[ -0.118 \left( \frac{D}{B} \right)^2 + 0.358 \left( \frac{D}{B} \right) - 0.214 \right] + \left[ 0.066 \left( \frac{D}{B} \right)^2 - 0.26 \left( \frac{D}{B} \right) + 0.894 \right]
\]
(3.11)
\( D \) is the length of the building, \( S \) is the area of cross section, \( H \) is the height of the building, \( \bar{n} = \frac{n}{n_s} \),
\[
n_s = \frac{S \cdot V_m(z_j)}{B}
\]
is the frequency of vortex shedding, \( H(C_1) = 0.179C_1 + 0.65\sqrt{C_1} \), and \( C_1 \) is a parameter correlated to bandwidth (Liang et al. 2002). The non-diagonal terms (cross-spectra) \( S_{F,F'}^{(j,k)}(n) \) of the across wind force due to vortex shedding are given by
\[
S_{F,F'}^{(j,k)}(n) = \sqrt{S_{F,F}^{(j,j)}(n) \cdot S_{F,F'}^{(k,k)}(n)} \cdot \exp\left[ -\left( \frac{\Delta}{\delta} \right)^2 \right]
\]
(3.12)
where $\Delta = \frac{z_j - z_k}{B}$, and $\delta$ is a constant that depends on the aspect ratio of the horizontal dimensions of the building (Liang et al. 2002).

It is assumed that the across wind turbulent forces and vortex shedding forces are mutually independent and the PSD function of the total across wind force can be obtained as the sum of the two PSD functions of across wind force due to turbulence and due to vortex shedding as:

$$
S_{F_{x,F_y}}^{(j,k)}(n) = \left[ A^{(j)} \cdot S_{v,v_c}^{(j,k)}(n) \cdot A^{(k)} \right] + S_{F_{x,F_y}}^{(j,k)}(n)
$$

(3.13)

The interaction analysis also provides the statistical characterization of the wind pressure acting on the cladding of the building at various height, $p_{w}^{(j)}$. In this study, the wind pressure acting on the cladding at the $j$-th floor of the building is computed as (ASCE 2010)

$$
p_{w}^{(j)} = q^{(j)} \cdot (G_{p} - G_{ps})
$$

(3.14)

in which the velocity pressure evaluated at $j$-th floor, $q^{(j)}$, is given by

$$
q^{(j)} = 0.613 \cdot K^{(j)} \cdot K_{z} \cdot V_{3sec}^2 \text{ (units: N/m}^2) \text{ )}
$$

(3.15)

where $K_{z}$ is assumed to be deterministically equal to 1, $K^{(j)}$ is the velocity pressure coefficient at height $z_j$, $K^{(j)} = 2.01 \left( \frac{z_j}{z_0} \right)^{2/3}$.

### 3.3.4 Structural analysis

The structural analysis phase provides the statistical description of the chosen EDPs, which concisely represent the essential aspects of the structural response for damage and performance evaluation. The choice of the EDPs depend on the choice of the limit states and DMs considered. For tall buildings, the following EDPs are commonly selected: (1) interstory drifts in the along wind and across wind directions at the $j$-th story ($I_{u}^{(j)} = D_{u}^{(j)} - D_{u}^{(j-1)}$ and $I_{v}^{(j)} = D_{v}^{(j)} - D_{v}^{(j-1)}$, 56
respectively, where $D^{(j)}_u$ and $D^{(j)}_v$ denote the displacement in the along wind and across wind directions at the $j$-th story, respectively; and (2) floor accelerations in the along wind and across wind directions at the $j$-th story ($A^{(j)}_u$ and $A^{(j)}_v$, respectively). A frequency domain approach can be adopted to calculate the response of the structure. The PSD functions of the displacement and acceleration are computed as:

$$
[S_{D,D} (n)] = \sum_{p=1}^{N_p} \sum_{q=1}^{N_q} \left\{ H_q (n) \cdot H^*_p (n) \cdot \left( \Phi_q \cdot \Phi_p^T \right) \cdot [S_{F,F} (n)] \cdot \left( \Phi_p \cdot \Phi_p^T \right) \right\}, \quad (l = u,v)
$$

$$
[S_{A,A} (n)] = n^4 \sum_{p=1}^{N_p} \sum_{q=1}^{N_q} \left\{ H_q (n) \cdot H^*_p (n) \cdot \left( \Phi_q \cdot \Phi_q^T \right) \cdot [S_{F,F} (n)] \cdot \left( \Phi_p \cdot \Phi_p^T \right) \right\}, \quad (l = u,v)
$$

where $H_q (n) = \frac{1}{4\pi} \left( \frac{1}{n_q^2 - n^2 + 2i \cdot \xi_q \cdot n \cdot n_q} \right)$ is the frequency response function for the $q$-th mode of vibration of the structure, $\Phi_q$ is the structural mode shape for the $q$-th mode of vibration, $i = \sqrt{-1}$, and the superscript $T$ represents the transpose operation. The variance and the covariance of the response can be obtained by integrating the corresponding PSD function over an appropriate range of frequencies. The peak value $r_p$ of a response quantity $r$ is given as

$$
r_p = r_m + g_r \cdot \sigma_r
$$

where $r_m$ is the mean value of the response (i.e., response of the building to mean wind velocity), $\sigma_r$ is the standard deviation of the response; and $g_r$ is the peak factor for response quantity $r$ and is assumed to be constant for all the floors for a given response quantity. The peak interstory drift between the $j$-th floor and the $(j-1)$-th floor in the along wind direction, $I^{(j)}_{u,p}$, is given by:

$$
I^{(j)}_{u,p} = \left( D^{(j)}_m - D^{(j-1)}_m \right) + g_{I_u} \cdot \sqrt{\sigma_{D^{(j)}_u}^2 + \sigma_{D^{(j-1)}_u}^2 - 2\text{COV} \left( D^{(j)}_u, D^{(j-1)}_u \right)}}
$$
where $D_{m}^{(j)}$ is the mean displacement at floor $j$, $\sigma_{D_{m}^{(j)}}^{2}$ is the variance of the displacement response in the along wind direction at floor $j$, $\text{COV}(D_{u}^{(j)},D_{u}^{(j-1)})$ is the covariance of the displacement response in the along wind direction between floor $j$ and $j-1$, and $g_u$ is the peak factor for the interstory drift in the along wind direction. Similarly, the peak interstory drift between the $j$-th floor and the $(j-1)$-th floor in the across wind direction can be calculated as

$$I_{v,p}^{(j)} = g_u \cdot \sqrt{\sigma_{D_{v}^{(j)}}^{2} + \sigma_{D_{v}^{(j-1)}}^{2} - 2\text{COV}(D_{v}^{(j)},D_{v}^{(j-1)})}$$

(3.19)

where $\sigma_{D_{v}^{(j)}}^{2}$ is the variance of the displacement response in the across wind direction at floor $j$, $\text{COV}(D_{v}^{(j)},D_{v}^{(j-1)})$ is the covariance of the displacement response in the across wind direction between floor $j$ and $j-1$, and $g_u$ is the peak factor for the interstory drift in the across wind direction.

The peak floor accelerations in the along wind and across wind directions are given by

$$A_{l,p}^{(j)} = g_A \cdot \sqrt{\sigma_{A_{l}^{(j)}}^{2} + \sigma_{A_{l}^{(j-1)}}^{2} - 2\text{COV}(A_{l}^{(j)},A_{l}^{(j-1)})}$$

(3.20)

where $\sigma_{A_{l}^{(j)}}^{2}$ is the variance of the acceleration response in the $l$-th direction at floor $j$, $\text{COV}(A_{l}^{(j)},A_{l}^{(j-1)})$ is the covariance of the acceleration response in the $l$-th direction between floor $j$ and $j-1$, and $g_A$ is the peak factor for the acceleration response in the $l$-th direction.

### 3.3.5 Damage analysis

The damage analysis provides the probabilistic description of $DM$ conditional to the values of $EDP$. In this analysis phase, the building components are categorized into different damage states based on the response of the building to the loads acting on it. Two types of approaches for building-specific loss estimation are available in the literature: (1) component-based loss
(Aslani and Miranda 2005), and (2) story-based loss estimation (FEMA 2007, Ramirez and Miranda 2009).

In the component-based loss estimation, building-specific damage and loss estimation procedures are developed at the component level. Each building component is assigned a fragility function to estimate the damage based on the level of structural response (Ramirez and Miranda 2009). It is assumed that the total loss in a building is equal to the sum of repair and replacement costs of the individual components damaged during the damaging event. Unlike single story residential buildings, obtaining a complete inventory of components for a complex building, such as a tall building, can be time consuming and expensive. Moreover, the amount of data to keep track of (e.g., the number of response parameters and their locations, the number of building components, the number of damage states) can become overwhelming, making the loss estimation process computationally too expensive (Ramirez and Miranda 2009). In the story-based loss estimation, individual component losses are grouped per each story. In this study, a story-based damage-loss estimation is used. The components of each floor of the building are categorized into three broad categories: (1) structural drift-sensitive components, (2) non-structural drift-sensitive components, and (3) non-structural acceleration-sensitive components (FEMA 2007, Ramirez and Miranda 2009). The damage to drift-sensitive components is primarily a function of the interstory drift, whereas the damage for acceleration-sensitive components is a function of the floor absolute acceleration.

The damage model describes the probable damage state of a component, described in terms of damage measures (DMs), given a level of engineering demand parameters (EDPs) using a mathematical relation between EDPs and DMs (Mackie et al. 2008). In this study, the damage models used in HAZUS are used for the damage analysis. Consistently with the approach by
HAZUS, four damage states (i.e., Slight, Moderate, Extensive, and Complete) are defined for structural drift-sensitive components, non-structural drift-sensitive components, and non-structural acceleration-sensitive components. The detailed description of the damage states for each category can be found in FEMA 2007.

The probability of exceeding a given damage state, $ds$, is modeled using a lognormal cumulative distribution function:

$$
P[DS \geq ds | EDP] = \Phi \left( \frac{1}{\beta_{ds}} \ln \left( \frac{EDP}{EDP_{th, ds}} \right) \right)
$$

(3.21)

where $EDP_{th, ds}$ is the median value of $EDP$ at which the component group reaches the threshold of the damage state, $ds$; $\beta_{ds}$ is the standard deviation of the natural logarithm of $EDP$ for damage state $ds$, and $\Phi$ is the standard normal cumulative distribution function.

In addition to the damage limit states described above, the serviceability limit state was also considered. The performance assessment of serviceability of tall buildings subject to wind hazard was carried out by evaluating the peak values of displacements and floor accelerations at different heights (Ciampoli and Petrini 2012), while considering uncertainties in design wind speed, as well as in the dynamic and aerodynamic properties of the building (Kwok et al. 2009). Tamura (2009) showed that the occupant motion perception is related to body sensation and/or visual cues. In general, the perception related to body sensation is dominant in case of low-frequency vibrations (less than 2 Hz), while the perception related to visual cues is dominant in case of relatively high-frequency vibrations (greater than 2 Hz). Many experimental tests were carried out to assess the motion perception thresholds for people performing different activities; the tests considered sinusoidal, random and elliptic motions simulating wind-induced building vibrations (Burton et al. 2006, Kwok et al. 2009, Tamura 2009). On the basis of these experimental results, it was concluded
that randomness does not affect the perception thresholds, as the perception for random motion was almost the same as that for sinusoidal motions. Therefore, the motion perception for wind-induced vibrations of a building can be simply based on the acceleration amplitude and the predominant natural frequency of the building (Tamura 2009). In this study, the human perception threshold for horizontal building vibrations was defined according to the comfort criteria reported in CNR 2008.

3.3.6 Loss analysis

The annual probability of exceedance of the $DV$ is estimated in this phase of the framework. The $DV$ is commonly chosen as the repair cost or the total cost of the structural system during its design lifetime (including construction and maintenance costs, repair costs, economic losses due to structural and content damage, and loss of functionality) (Bjarnadottir et al. 2011).

Consistent with other PBE approaches, in PBHE the losses are broadly classified into “direct” or “indirect” losses. The direct losses include losses due to damaged components (both structural and non-structural), losses related to serviceability limit state, and losses related to work disruption or to the discomfort of building occupants due to wind-induced vibrations. It is very difficult to quantify direct losses related to serviceability limit states if they cannot be associated with physical damages (e.g., damage of non-structural components). Direct serviceability limit state losses related to non-structural damages can occur in high-rise buildings during hurricanes due to both excessive drift displacements and debris impact on facades. The indirect losses are mainly due to the negative publicity and perception of lack of safety for the building which has shown excessive vibrations (Petrini et al. 2014). This study focuses only on the direct losses associated with wind hazard.
Using a story-based loss estimation method, the cost of replacement of each story can be calculated from the building inventory and from the construction cost data. The total cost of the each story is then divided into each component group category (i.e., structural drift-sensitive, non-structural drift-sensitive, and non-structural acceleration-sensitive). However, the data to derive a detailed cost allocation of the different components for individual buildings are rarely available and, thus, a more common approach is to adopt from the literature a general replacement cost allocation among different component groups for a given building typology (Ramirez and Miranda 2009).

### 3.4 Application Example

The presented PBHE framework is illustrated through an application example consisting of the risk assessment for a 74-story building, subjected to both hurricane and non-hurricane associated winds.

![Figure 3.3 Finite element model of the target building](image)

(a) full FE model; (b) 3D frame on the external perimeter; (c) bracing system; and (d) central core.
Figure 3.3 shows the finite element model of the target building. The building has a square plan with a side length $B = 51$ m and a total height $H = 305$ m. The main structural system is composed of a central core (a 3D frame with 16 columns), and a 3D frame composed by 28 columns on the external perimeter. The two substructures are connected at three levels (at 100, 200, and 300 m) by stiffening systems extended for 3 floors. The columns have a hollow square section, with dimensions and thickness varying with the height (1.20 m and 0.06 m floors 1-23, 0.9 m and 0.045 m for floors 24-48, and 0.5 m and 0.025 m for floors 49-74). The beams are double-T steel beams and the beam–column joints are considered as being rigid. The bracing system is composed by double-T or hollow square struts (Figure 3.3). The building is assumed to be located in Miami, Florida. The total value of the structure is $329$ Million.

3.4.1 Details of the different analysis steps

The number of hurricanes per year was simulated using a Poisson occurrence model, with an annual hurricane occurrence rate $\nu_{\text{hurricane}} = 0.54$ taken from the Iowa Environmental Mesonet (IEM) database for the 1962-2013 period (IEM 2001). The 10-minute wind speed ($V_{10}$) and 3-second wind speed ($V_{3\text{sec}}$) recorded at 10 m above the ground were adopted as IM for wind hazard for structural responses and local responses, respectively. The hurricane wind speed variability was described by using a three parameter type II generalized extreme value distribution with the following cumulative distribution function:

\[
F(V_{10}) = \begin{cases} 
0; & V_{10} \leq c \\
 e^{-[(\nu_{10}-c)/b]^{-a}}; & V_{10} > c
\end{cases} \tag{3.22}
\]

The three parameters $a$, $b$ and $c$ are site specific and were determined through maximum likelihood estimation of the hurricane wind speed records provided by the IEM database. The IEM wind
speed records contain data sets of recorded 3-second wind speeds at 10 m above the ground. Before fitting, the wind speed data were multiplied by a factor equal to 0.7, to obtain the corresponding 10-minute wind speeds for exposure category B (Lungu and Rackwitz 2001). For each generated hurricane event, the maximum 10-minute wind speed was generated according to this fitted type II distribution.

Non-hurricane wind hazard was also considered in addition to hurricane wind hazard. The daily maximum 3-second wind speeds at the building location were obtained from the IEM database for the 1962-2013 period (IEM 2001). The historical hurricane tracks that passed within a 250 miles radius from the site during the same 1962-2013 period were obtained from the National Oceanic and Atmospheric Administration (NOAA) database and were used to separate the non-hurricane wind speeds from the hurricane wind speeds. The yearly maximum non-hurricane 10-minute wind speeds were then obtained and fitted to a truncated log-normal distribution, with a mean of 19.3 m/s and standard deviation of 0.5 m/s. The roughness length \( z_0 \) was assumed to follow a lognormal distribution with mean value of 0.1 m and standard deviation of 0.03 m (Zhang et al. 2008).

In this case study, only the first 6 modes in the lateral direction were considered, which corresponds to 95% model mass participation ratio, and torsional effects were not considered. The structural damping ratios for each of the considered modes of vibration, \( \xi_q \) \( (q = 1, 2, \ldots, 6) \), were assumed to be statistically independent and to follow a lognormal distribution with mean value of 0.02 and coefficient of variation (CV) of 0.4 (Petrini and Ciampoli 2011). The structural mode shape and the corresponding frequencies (see Table 3.1) were computed using a finite element model developed in STAAD.Pro (STAAD.Pro v8i 2012). It was also assumed that the structural parameters do not change during the hazard event (i.e., after the damage analysis the structural
parameters were not updated). This assumption is reasonable, since the number of occurrences of the building entering into the plastic range is very small, and the potential excursions in the plastic range are sufficiently small that they have a negligible effect on the vibrational characteristics of the building.

Table 3.1 Structural mode and corresponding frequency.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>First</td>
<td>0.185</td>
</tr>
<tr>
<td>Second</td>
<td>0.587</td>
</tr>
<tr>
<td>Third</td>
<td>1.082</td>
</tr>
<tr>
<td>Fourth</td>
<td>2.057</td>
</tr>
<tr>
<td>Fifth</td>
<td>2.652</td>
</tr>
<tr>
<td>Sixth</td>
<td>3.293</td>
</tr>
</tbody>
</table>

The drag coefficient, $C_d$, was assumed to follow a Gaussian distribution with mean value of 1.05 (which was obtained from wind tunnel experimental tests reported in Spence et al. 2008) and CV of 0.05, and the lift coefficient, $C_L$, was assumed to be deterministically equal to zero (Ciampoli and Petrini 2012). The PSD functions for the wind forces corresponding to each vertically aligned node in the along wind and across wind direction were computed using Eq. (3.8) and Eq. (3.12), respectively. Figure 3.4 shows the PSD function for the wind forces acting in the along and across wind directions for $V_{10} = 20$ m/s at floors 30, 50 and 74. The structural analysis was performed in the frequency domain, and the displacement and acceleration PSD functions were calculated using Eq. (3.16). Figure 3.5 and Figure 3.6 shows the displacement and acceleration PSD functions in the along wind direction and the across wind directions for selected floors.
The PSD function for the responses at each floor was integrated over the entire frequency range to obtain the variances of the responses. The peak response at each floor was obtained using Eq. (3.17) through (3.20). The peak response factor, \( g_r \), was assumed to follow a Gaussian distribution with mean value \( \mu_{g_r} \), and standard deviation \( \sigma_{g_r} \), which are given by (Davenport 1983):

\[
\mu_{g_r} = \frac{0.577}{\sqrt{2 \ln(\eta \cdot T_{\text{wind}})}}
\]

\[
\sigma_{g_r} = \frac{\pi}{\sqrt{12 \ln(\eta \cdot T_{\text{wind}})}}
\]

respectively, where \( \eta \) is the mean zero-crossing rate of the response, that can be approximated by the first natural frequency \( f_1 \) of the structure, and \( T_{\text{wind}} \) is the duration of the time interval over which the peak response is evaluated.
Figure 3.5 Power spectral density function for displacements: (a) along wind direction, and (b) across wind direction.

Figure 3.6 Power spectral density function for floor accelerations: (a) along wind direction, and (b) across wind direction.
The fragility curves for the different component groups were obtained using the HAZUS damage function (Eq. 3.21) and the parameter values shown in Table 3.2 (FEMA 2007).

Table 3.2 Fragility curve parameters for different component groups.

<table>
<thead>
<tr>
<th>Components</th>
<th>Slight</th>
<th>Moderate</th>
<th>Extensive</th>
<th>Complete</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$E_D P_{th}$</td>
<td>$\beta_{ds}$</td>
<td>$E_D P_{th}$</td>
<td>$\beta_{ds}$</td>
</tr>
<tr>
<td>Structural drift sensitive</td>
<td>0.25%</td>
<td>0.4</td>
<td>0.5%</td>
<td>0.4</td>
</tr>
<tr>
<td>(Interstory drift ratio)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Non-structural drift sensitive</td>
<td>0.4%</td>
<td>0.5</td>
<td>0.8%</td>
<td>0.5</td>
</tr>
<tr>
<td>(Interstory drift ratio)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Non-structural acceleration sensitive</td>
<td>0.3</td>
<td>0.6</td>
<td>0.66</td>
<td>0.6</td>
</tr>
<tr>
<td>(Floor acceleration, (m/s²))</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 3.7 Fragility curves for different component groups: (a) structural drift-sensitive, (b) non-structural drift-sensitive, and (c) non-structural acceleration-sensitive.
Figure 3.7 shows the fragility curve for the different component groups. The probability of the component group exceeding each damage state was calculated using Eq. (3.21) and, based on that probability, a damage state was generated for each component group.

Table 3.3 Repair costs (in % of floor cost) for component groups for each damage state.

<table>
<thead>
<tr>
<th>Component group</th>
<th>Slight damage</th>
<th>Moderate damage</th>
<th>Extensive damage</th>
<th>Complete damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drift-sensitive, structural components</td>
<td>0.4</td>
<td>1.9</td>
<td>9.6</td>
<td>19.2</td>
</tr>
<tr>
<td>Drift-sensitive, non-structural components</td>
<td>0.7</td>
<td>3.4</td>
<td>16.4</td>
<td>32.9</td>
</tr>
<tr>
<td>Acceleration-sensitive, non-structural components</td>
<td>0.9</td>
<td>4.8</td>
<td>14.4</td>
<td>47.9</td>
</tr>
</tbody>
</table>

The repair cost of each component group at each floor was generated based on a lognormal distribution, with a mean given as a percentage of the total floor cost for each damage state as shown in Table 3.3. For the serviceability limit state, the floor acceleration was compared with the human perception threshold value, which was assumed to be 0.15 m/s² (Ciampoli and Petrini 2012). The business interruption loss due to exceedance of the human perception threshold was generated based on a lognormal distribution with mean value of $0.92 per square foot of any given floor (FEMA 2007) and CV of 0.1 for each day during which the business was interrupted.
In the event of a hurricane, if the human perception threshold limit was exceeded at any floor, then it was assumed that the whole building was closed for the entire duration of the hurricane. The duration of the hurricane is assumed to follow a uniform distribution with range $[1 – 3]$ days.

The loss due to business interruption during non-hurricane winds was calculated by first examining whether the yearly maximum wind caused any exceedance of the human perception threshold. If the threshold was exceeded during a specific one-year simulation, then the minimum threshold velocity that could cause the exceedance of the human perception threshold was calculated by scaling down the yearly maximum wind velocity by assuming a linear relation between wind velocity and the maximum floor acceleration. Daily maximum wind velocities (in a number of 364 minus the number of days during which a hurricane event took place) were then randomly generated (using rejection sampling) for this specific one-year simulation using a lognormal distribution truncated at the upper tail in correspondence to the yearly maximum wind velocity. The mean value of this lognormal distribution was generated from the joint probability distribution of yearly maximum wind velocity and mean daily maximum wind velocity. This choice was based on the observation that the mean daily maximum wind velocity presented significant differences from year to year, with a strong correlation between the mean daily maximum wind velocity computed over the year and the corresponding yearly maximum wind velocity. The dependence between the yearly maximum wind velocity and the mean daily maximum wind velocity was modeled using a Frank’s copula (Nelsen 2007). The standard deviation was calculated from the daily maximum wind speed records of the entire IEM database, based on the observation that the standard deviation of the daily maximum wind speeds was almost constant for different years. Based on this inner loop of stochastic simulations, the number of days during which the daily maximum wind velocity was higher than the minimum threshold velocity was estimated and used
to calculate the annual loss due to business interruption. It was assumed that the business on a particular floor was interrupted for a day if the daily acceleration response was greater than the human perception threshold value.

The total loss was calculated by adding up all the floor losses due to the damage of the different component groups and the loss due to business interruption. In order to accurately estimate the annual probability of exceedance of the total loss (which coincides with the complementary cumulative distribution function of the $DV$), 10,000 samples were used for all results presented in this study.

### 3.4.2 Loss analysis results for the target structure

Figure 3.8 plots in semi-logarithmic scale the annual probability of exceedance of the maximum acceleration and the maximum displacement in the along wind and across wind direction for the 74th floor of the building. Similar results were obtained also for all other building’s floors.

![Annual probability of exceedance for different responses in the along wind and across wind directions at floor 74.](image)

**Figure 3.8** Annual probability of exceedance for different responses in the along wind and across wind directions at floor 74.
From the results presented in Figure 3.8, it is observed that the annual probability of exceedance is significantly higher for the displacement in the along wind direction than for the displacement in the across wind direction, whereas the annual probability of exceedance is significantly higher for the acceleration in the across wind direction than for the acceleration in the along wind direction. The first result is mainly due to the deflection produced by the mean wind velocity (i.e., the time-invariant component of the wind field) in the along wind direction, whereas the second result is mainly due to the vortex shedding effect in the across wind direction.

Figure 3.9 plots, in a semi-logarithmic scale, the annual probabilities of exceedance of the loss for the target building for different limit states (i.e., drift-sensitive structural components, drift-sensitive non-structural components, acceleration-sensitive non-structural components, and serviceability) and their combination (i.e., loss due to damage and total loss).
The corresponding expected annual losses (EALs) were also computed and are shown in Figure 3.9 for each of the different limit states. The EAL for each case considered here is defined as the average economic loss that occurs every year in the building (Raul and Vitelmo 2004) and is equal to the area under the corresponding annual probability of exceedance curve.

From the results presented in Figure 3.9, it is observed that the loss due to business interruption caused by floor accelerations exceeding the human perception threshold is predominant when compared to the loss due to component damage. Among the component losses, the loss due to non-structural acceleration-sensitive components is predominant when compared to the losses due to drift-sensitive structural and non-structural components. This result suggests the need to increase the performance of the building with respect to the acceleration response, e.g., by using an efficient Tuned Mass Damper (TMD). As highlighted by Ciampoli and Petrini (2012), the use of a TMD may improve the serviceability performance of the building by increasing the human perception thresholds, and decreasing the peak value of the across-wind acceleration.

3.5 Conclusions

In this paper, the Performance-Based Hurricane Engineering (PBHE) framework was used for the risk assessment of tall buildings subjected to both hurricane and non-hurricane wind hazards. The general multi-layer Monte-Carlo simulation (MCS) approach was specialized for the risk assessment of engineered buildings such as high-rise buildings. The problem of risk assessment was disaggregated into the following basic probabilistic components: (1) hazard analysis, (2) structural characterization, (3) interaction analysis, (4) structural analysis, (5) damage analysis, and (6) loss analysis. The different random parameters involved in these analysis phases were identified and their statistical characteristics were obtained from the literature. A story-based loss
estimation method was used for the loss analysis in conjunction with damage and loss functions taken from HAZUS®.

An application example consisting of a 74-story building located in Miami County, Florida, was presented to illustrate the framework. The annual probabilities of exceedance of the response in the along and across wind directions were calculated. For this application example, it was observed that: (1) the annual probabilities of exceedance of the displacements in the along wind direction are significantly larger than the corresponding probabilities for the displacements in the across wind direction, due to the effects of the mean wind speed on the response; and (2) the annual probabilities of exceedance of the accelerations in the across wind direction are significantly larger than the corresponding probabilities for the accelerations in the along wind direction, due to the effects of the vortex shedding on the structural response.

The expected losses for the target building for different limit states were also calculated. It was found that the loss due to business interruption is predominant when compared to the loss due to structural and non-structural damage. Among the different component losses, it was found that the loss due to damage of non-structural acceleration-sensitive components is predominant.

Based on the results presented in this chapter, it is concluded that the PBHE framework can be used for performance-based design, risk assessment, and/or loss assessment of tall buildings. It can also assist owners, insurers, designers, and policy makers in making informed decisions on design and retrofit of buildings subject to hurricane and non-hurricane wind hazards.

3.6 References


http://bbaa6.mecc.polimi.it/papers.html#papers.


CHAPTER 4
PERFORMANCE-BASED HURRICANE ENGINEERING: A MULTI-HAZARD APPROACH

4.1 Introduction

Structures located in coastal regions at tropical and subtropical latitudes are at high risk of suffering severe damages and losses from wind and surge hazards due to tropical storms. As the population tends to concentrate on coastal regions and the number of residential buildings in hurricane-prone areas continues to rise, the societal vulnerability to hurricanes is increasing, with the prospect of even higher damages and losses in the future (Li and Ellingwood 2006). Most of the U.S.’s densely populated Atlantic and Gulf Coast coastlines lie less than 10 ft above mean sea level (NOAA 2011), and are vulnerable to hurricane-induced surge. During Hurricane Katrina in 2005, hurricane-induced surge caused catastrophic damage to residential buildings and tragic loss of life (Eamon et al. 2007, van de Lindt et al. 2009).

Early studies on hurricane hazard assessment and mitigation focused on the damage/loss from individual hazards like wind (including water intrusion due to rainfall) or surge. Powell and Houston (1995) proposed a real-time damage assessment model based on a damage function relating various meteorological variables to the percentage of damage to the buildings. Thomalla et al. (2002) developed a storm surge and inundation model for the risk assessment of residential buildings. Discrete damage states were identified and assigned on the basis of inundation and component damage of the building. Li and Ellingwood (2006) developed a probabilistic risk assessment methodology to assess the performance and reliability of low-rise light-frame wood residential construction subjected to hurricane wind hazard. More recently, the widespread losses observed in the recent hurricanes motivated researchers to consider the combined effects of hurricane wind and surge hazards. Phan et al. (2007) proposed a methodology for creating site-
specific joint distributions of combined hurricane wind and surge for Tampa, Florida using full track hurricanes to compute the wind speed and the Sea, Lake, and Overland Surge from Hurricanes (SLOSH) model (Jelesnianski et al. 1992) to estimate surge heights. Lin and Vanmarcke (2010) developed an integrated vulnerability model to explicitly accounts for the correlation between wind-borne debris damage and wind pressure damage. This integrated vulnerability model was obtained by coupling a pressure-damage model derived from the component-based model of the Florida Public Hurricane Loss Model (FPHLM, Gurley et al. 2005) with the wind-borne debris risk model developed by Lin and Vanmarcke (2008). Friedland and Levitan (2011) developed a joint hurricane wind–surge damage scale based on a loss-consistent approach using HAZUS-MH (Hazards United States Multi-Hazards) hurricane model damage and loss functions (FEMA 2012) and the USACE (US Army Corps of Engineers) flood depth-loss functions (USACE 2000) for the assessment of damage from combined wind and flood events. Li et al. (2011) conducted a risk assessment for residential buildings by estimating the combined losses from hurricane wind, storm surge, and rainwater intrusion. The correlation between wind and surge was considered in their study by implementing a hurricane-induced surge model through regression analysis of historical data. Dao and van de Lindt (2011) presented a methodology based on the combination of existing wind tunnel data and rainwater intrusion model, for estimating the probability of rainwater intrusion into each room of typical wood-frame structures subjected to hurricanes. Barbato et al. (2013) developed a Performance-Based Hurricane Engineering (PBHE) framework and applied it for the risk assessment of residential buildings subjected to wind and windborne debris impact. They also observed that the interaction between different hazard sources can significantly affect the risk assessment and emphasized the need to consider the multi-hazard nature of hurricane events for accurate probabilistic loss analysis. Pei et al. (2014) developed joint
hazard maps of combined hurricane wind and surge for Charleston, South Carolina. The surface wind speeds and surge heights from individual hurricanes were computed using the Georgiou’s wind field model (Georgiou 1985) and the SLOSH model (Jelesnianski et al. 1992), respectively.

In this chapter, the PBHE framework (Barbato et al. 2013) is adopted for the risk assessment of structural systems located in hurricane-prone regions. A hypothetical case study is presented to illustrate the adopted methodology and the specialized multi-layer MCS approach for loss analysis of residential buildings subject to hurricane hazard including all pertinent hazard sources (i.e., wind, windborne debris, surge, and rainfall).

4.2 Summary of PBHE Framework

The PBHE framework proposed in Barbato et al. (2013) disaggregates the performance assessment procedure for structures subject to hurricane hazard into elementary phases that are carried out in sequence. The structural risk within the PBHE framework is expressed by the probabilistic description of a decision variable, $DV$, which is defined as a measurable quantity that describes the cost and/or benefit for the owner, the users, and/or the society resulting from the structure under consideration. The fundamental relation for the PBHE framework is given by:

$$G(DV) = \int \int \int G(DV|DM) \cdot f(DM|EDP) \cdot f(EDP|IM, IP, SP) \cdot f(IP|IM, SP) \cdot f(IM) \cdot f(SP) \cdot dDM \cdot dEDP \cdot dIP \cdot dIM \cdot dSP$$

where $G(\bullet) = \text{complementary cumulative distribution function}$, and $G(\bullet|\bullet) = \text{conditional complementary cumulative distribution function}$; $f(\bullet) = \text{probability density function}$, and $f(\bullet|\bullet) = \text{conditional probability density function}$; $IM = \text{vector of intensity measures}$ (i.e., the parameters characterizing the environmental hazard); $SP = \text{vector of structural parameters}$ (i.e., the parameters describing the relevant properties of the structural system and non-environmental actions); $IP = \text{vector of interaction parameters}$ (i.e., the parameters describing the interaction phenomena
between the environment and the structure); \( EDP \) = engineering demand parameter (i.e., a parameter describing the structural response for the performance evaluation); and \( DM \) = damage measure (i.e., a parameter describing the physical damage to the structure). In Eq.(4.1), \( IM \) and \( SP \) are assumed as uncorrelated and independent of \( IP \), while \( IP \) is dependent on both \( IM \) and \( SP \). By means of Eq.(4.1), the risk assessment is disaggregated into the following tasks: (1) hazard analysis, (2) structural characterization, (3) interaction analysis, (4) structural analysis, (5) damage analysis, and (6) loss analysis.

4.3 Multi-layer Monte Carlo Simulation

Eq. (4.1) can be solved using different techniques, e.g., closed-form analytical solutions (Shome and Cornell 1999, Jalayer and Cornell 2003, Zareian and Krawinkler 2007, Mackie et al. 2007), direct integration techniques (Bradley et al. 2009), and stochastic simulation techniques (Porter et al. 2001, Au and Beck 2003, Lee and Kiremidjian 2007). In PBHE, analytical solutions and direct integration techniques require the knowledge of the joint probability density function of the component losses, which is very difficult to obtain for real-world applications. Thus, in this study, the general multi-layer MCS approach (Conte and Zhang 2007) is adopted and specialized to efficiently perform loss analysis for residential buildings subject to hurricane hazard. The result of the PBHE equation (Eq.(4.1)) is the annual loss curve, \( G(DV) \), i.e., the complementary cumulative distribution function of the annual losses for the residential building under consideration due to hurricane events.

Figure 4.1 shows the flowchart of the general multi-layer MCS technique applied to PBHE. Multi-layer MCS takes into account all phases of the PBHE framework (namely, hazard analysis, structural characterization, interaction analysis, structural analysis, damage analysis, and loss analysis). Each of these analysis phases is performed in two step: (1) a sample generation step of
random parameters with known probability distributions, which are needed to describe the uncertainties in environmental actions, structural properties, interaction phenomena, analysis techniques, and cost estimates; and (2) an analysis step based on a deterministic model, which is used to model the propagation of uncertainties from input to output parameters of each analysis phase. It is noted here that the analysis steps are usually more computationally intensive than the corresponding sample generation steps. Thus, it is useful to identify specific conditions under which one or more of the analysis steps can be avoided in order to reduce the computational cost of the multi-layer MCS approach.

![Diagram](image)

This is a general multi-layer MCS approach for PBHE framework.

4.3.1 Specialized Multi-layer MCS Approach for Pre-engineered and Non-engineered Buildings

Pre-engineered and non-engineered buildings, e.g., single-family residential buildings, are structures that are constructed based on design models with components consisting of products that are certified based on building code requirements (NAHB 2000). For these specific building typologies, component strength statistics are commonly available as functions of the...
environmental action intensity. Under these conditions, the damage analysis phase can be performed without requiring the statistical description of the structural response of the building. In fact, the probabilistic description of the strength for the building components subject to damage (i.e., windows, doors, walls, and roof) can be obtained from empirical relations available in the literature as a function of opportently chosen \( IP \). Thus, it is computationally convenient to eliminate the structural analysis phase from the multi-layer MCS procedure. This simplification considerably reduces the computational cost of the multi-layer MCS approach for probabilistic hurricane loss analysis of residential buildings and other pre-engineered buildings. It is noted here that, for simple structures of risk category I and II (ASCE 2010), such as single-family residential buildings, simplified and computationally inexpensive models are often appropriate to perform the analysis steps required by the PBHE methodology.

### 4.3.2 Multi-hazard Characterization of Hurricane Events

The multi-hazard nature of the phenomena related to hurricanes and their effects on the built environment can manifest itself in the following three different modalities (Barbato et al. 2013):

1. Independent hazards, when different hazards affect the structure independently. For example, windborne debris and flood hazard can be considered as independent of each other because no mutual interaction between the two hazards has the effect of modifying the intensity of the corresponding actions. These hazards can occur individually or simultaneously.

2. Interacting hazards, when the actions produced on a structure by different hazards are interdependent (e.g., wind and windborne debris hazards).

3. Hazard chains, when the effects of some hazards modify sequentially the effects of other hazards. For example, the actions on a structure due to windborne debris can damage the structural envelope and increases the vulnerability of the subject structure to strong winds.
In the PBHE framework, the first two cases (i.e., independent and interacting hazards) are treated within the hazard analysis, by assuming proper interaction models between the hazards, e.g., by using a proper joint probability distribution function to describe the variability of the IM for different hazards as in Phan et al. (2007). The study of hazard chains requires modeling the structural system configuration and properties as a function of the level of structural damage caused by the different hazards. In particular, the presence of a hazard chain implies that the SP can change as a consequence of DM exceeding specified thresholds. Thus, structural characterization, interaction analysis, and structural analysis cannot be carried out without any information or assumption on the values of DM.

4.4 Case Study

The PBHE framework is illustrated here by considering a case study in which wind, windborne debris, flood, and rainfall hazards interact. This case study consists of a hypothetical residential development, located near the coast in Panama City, Florida and composed by 25 identical concrete block gable roof structures (Figure 4.2).

![Figure 4.2 Plan view of the residential development.](image)
The roof covers are considered as debris sources, whereas the windows and doors are considered as debris impact vulnerable components. The value of the target building is assumed to be $200,000 and the value of the content is assumed equal to $100,000. In this study, the cost associated with loss of usage is not considered.

4.4.1 Hazard analysis

In this study, the 3-second wind speed ($V$) recorded at 10 m above the ground is considered as the $IM$ for wind hazard. Among the different wind field models available in the literature (Batts et al. 1980, Peterka and Shahid 1998, Li and Ellingwood 2006), the Weibull distribution is adopted here to describe the hurricane wind speed variability. The two-parameter Weibull cumulative distribution function is given by:

$$F(V) = 1 - \exp \left[ -\left(\frac{V}{a}\right)^b \right]$$  \hspace{1cm} (4.2)

The two shape parameters $a$ and $b$ are site specific and are determined for sixteen different wind directions by fitting to a Weibull distribution the hurricane wind speed records for the corresponding directions provided by the National Institute of Standards and Technology (NIST). The NIST wind speed records contain data sets of simulated 1-minute hurricane wind speeds at 10 m above the ground in an open terrain near the coastline, for locations ranging from milepost 150 (near Port Isabel, Texas) to milepost 2850 (near Portland, Maine), spaced at 50 nautical mile intervals (92,600 m). Considering Panama City, Florida as the location for the case study, the dataset corresponding to milepost 1000 is used for fitting the distribution.

The parameters needed to describe the windborne debris are: (1) the relative distribution of different debris types, e.g., compact-type, rod-type, and sheet-type debris (Wills et al. 2002); (2) the physical properties of the debris, e.g., for sheet-type debris, $M_d = \text{mass per unit area of the}$
debris, and \( A_d = \) area of the single debris; (3) the resistance model for the debris sources (which contributes to determine the number of windborne debris generated by a given source under a specified wind speed); and (4) the trajectory model for the debris (which describes the debris flight path).

In residential developments, the windborne debris are predominantly sheet-type, e.g., roof shingles and sheathing (Holmes 2010), hence this study focuses on sheet-type debris. The area and mass per unit area of debris are assumed to follow a uniform distribution defined in the range \([0.108, 0.184]\) m\(^2\) and \([10.97, 14.97]\) kg/m\(^2\), respectively.

The debris generation model employed by the FPHLM (Gurley et al. 2005) is adopted in this study. This model is a component-based pressure-induced damage model, which provides the number of debris generated from each source house as a function of (1) the percentage of roof cover damage for a given 3-second gust wind speed, and (2) the geometry of the house.

The debris trajectory model provides the landing position of the debris as identified by the random variables \( X = \) along-wind flight distance, and \( Y = \) across-wind flight distance. These random variables are modeled using a two-dimensional Gaussian distribution (Lin and Vanmarcke 2008) described by the following parameters: \( \mu_x = \) mean along-wind flight distance; \( \mu_y = 0 \) m = mean across-wind flight distance; \( \sigma_x = \sigma_y = 0.35\mu_x = \) standard deviation of the along-wind and across-wind flight distance, respectively. The parameter \( \mu_x \) is computed as (Lin and Vanmarcke 2008):

\[
\mu_x = \frac{2M_d}{\rho_a} \left[ \frac{1}{2} C \cdot \left( K \cdot T \right)^2 + c_1 \cdot \left( K \cdot T \right)^3 + c_2 \cdot \left( K \cdot T \right)^4 + c_3 \cdot \left( K \cdot T \right)^5 \right] 
\]

(4.3)

in which \( \rho_a = 1.225\) kg/m\(^3\) = air density; \( K = \frac{\rho_a \cdot V^2}{2M_d \cdot g} = \) Tachikawa number; \( \tilde{T} = \frac{g \cdot T}{V} = \) normalized time; \( g = \) gravity constant; \( T = \) flight time in seconds; \( C, c_1, c_2, \) and \( c_3 = \) non-
dimensional coefficients that depend on the shape of the debris. The flight time is assumed to follow a uniform distribution with range \([1, 2.5]\) seconds. For the sheet-type debris considered in this study, \(C = 0.91, c_1 = -0.148, c_2 = 0.024, \) and \(c_3 = -0.0014.\)

In this study, a hurricane-induced surge model proposed by Irish et al. (2008) based on the regression analysis of historical data is used. The surge height \((\zeta)\) is considered as the intensity measure for flood hazard and is computed as (Irish et al. 2008):

\[
\sqrt{\zeta} = \left[ \sqrt{R_{\text{max}}} \begin{bmatrix} 1 \\ \Delta p \\ 1 \end{bmatrix} \right] \cdot \begin{bmatrix} \Delta p^2 \\ \Delta p \\ 1 \end{bmatrix}
\]

(4.4)

where \(\zeta = \frac{\zeta \cdot g}{V^2}, \) \(R_{\text{max}} = \frac{R_{\text{max}} \cdot g}{V^2}, \) \(\Delta p = \frac{\Delta p}{p_{\text{atmos}}}, \) \(R_{\text{max}} = \) radius of maximum wind (in km), \(\Delta p = \) central pressure deficit (in millibars), \(p_{\text{atmos}} = \) atmospheric pressure (in millibars), and

\[
C(S_0) = \begin{bmatrix} -1.078 \times 10^{-1} & 3.996 \times 10^{-2} & 4.444 \times 10^{-4} \\ 3.974 \times 10^0 & -1.093 \times 10^0 & -1.653 \times 10^1 \end{bmatrix} = 2 \times 3 \text{ curve fitting coefficient matrix}
\]

(assuming an ocean slope of 1:5000). The radius of maximum wind \((R_{\text{max}})\) is assumed to follow a normal distribution with a mean of 39.4 km and COV of 0.46, and the central pressure deficit \((\Delta p)\) follows a lognormal distribution with mean of 70.4 mb and COV of 0.22.

In this study, the rainfall hazard model used in the FPHLM (Pita et al. 2012) is adopted to compute the impinging rainfall rate \((IRR)\), which is considered as the intensity measure. This model describes the \(IRR\) as function of 3-second gust speed \((V)\) and is given as:

\[
IRR = 0.84205V - 11.482 \quad \text{(units: } IRR = \text{cm, } V = \text{m/s)}
\]

(4.5)

The hazard curves for the different hazard sources are computed and plotted in a semi-logarithmic scale. (see Figure 4.3).
4.4.2 Structural characterization

The structural characterization phase provides the probabilistic description of the SPs. The SPs represent the geometrical and/or mechanical properties of the structure which influence the loading applied to the structure itself and, thus, the IPs. Geometrical properties can usually be treated as deterministic quantities, since they can be directly measured for existing structures or are characterized by a small variability. The position and dimension of the windows and doors of the target building are shown in Figure 4.4.
Table 4.1 shows the parameters corresponding to the target residential building (Gurley et al. 2006).

Table 4.1 Structural parameters of target building.

<table>
<thead>
<tr>
<th>Structural Parameter</th>
<th>Dimension</th>
</tr>
</thead>
<tbody>
<tr>
<td>Length</td>
<td>60 ft</td>
</tr>
<tr>
<td>Width</td>
<td>4 ft</td>
</tr>
<tr>
<td>Height of wall</td>
<td>10 ft</td>
</tr>
<tr>
<td>Roof Pitch</td>
<td>5/12</td>
</tr>
<tr>
<td>Eave overhang</td>
<td>2 ft</td>
</tr>
<tr>
<td>Space between roof trusses</td>
<td>2 ft</td>
</tr>
<tr>
<td>Roof sheathing panel dimension</td>
<td>8 ft X 4 ft</td>
</tr>
</tbody>
</table>

The SPs considered in this case study also include: (1) the wind pressure exposure factor (evaluated at $h =$ height of the target building), $K_h$; (2) the external pressure coefficients, $GC_p$; and (3) the internal pressure coefficients, $GC_{pi}$.

The pressure coefficients include the effects of the gust factor $G$ and are different for different locations within the building (roof zones and windward/leeward/side walls) and/or different conditions of the envelope (enclosed or breached). The value of the topographic factor, $K_{zt}$, is assumed deterministically equal to one. The $SP K_h$ is assumed as normally distributed with a mean value of 0.71 and a coefficient of variation (COV) of 0.19 (Lee and Rosowsky 2005). The statistical characterization of the external and internal pressure coefficients is given in Table 4.2 (Li and Ellingwood 2006).
Table 4.2 Structural characterization of external and internal pressure coefficients.

<table>
<thead>
<tr>
<th>Location/Condition</th>
<th>Mean</th>
<th>COV</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof (zone 1)</td>
<td>-0.855</td>
<td>0.12</td>
<td>Normal</td>
</tr>
<tr>
<td>Roof (zone 2)</td>
<td>-1.615</td>
<td>0.12</td>
<td>Normal</td>
</tr>
<tr>
<td>Roof (zone 3)</td>
<td>-2.47</td>
<td>0.12</td>
<td>Normal</td>
</tr>
<tr>
<td>Windward wall</td>
<td>0.95</td>
<td>0.12</td>
<td>Normal</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>-0.76</td>
<td>0.12</td>
<td>Normal</td>
</tr>
<tr>
<td>Side wall</td>
<td>-1.045</td>
<td>0.12</td>
<td>Normal</td>
</tr>
<tr>
<td>Enclosed</td>
<td>0.15</td>
<td>0.33</td>
<td>Normal</td>
</tr>
<tr>
<td>Breached</td>
<td>0.46</td>
<td>0.33</td>
<td>Normal</td>
</tr>
</tbody>
</table>

4.4.3 Interaction analysis

The choice of the IP vector is crucially dependent on the hazard sources, limit states, and performance levels of interest for both structural and non-structural elements. In this study, the IP vector is selected to represent the effects of wind and windborne debris hazard on the different limit states of interest for low-rise residential buildings.

The interaction analysis for the wind hazard provides the statistical characterization of the wind pressure acting on the different components of the buildings, $p_w$. In this study, the wind pressure acting on the $j$-th component of the building is computed as (ASCE 2010)

$$p_{w,j} = q_h \cdot (GC_{p,j} - GC_{pi,j})$$  \hspace{1cm} (4.6)$$

where the velocity pressure, $q_h$, evaluated at $h$, is given by

$$q_h = 0.613 \cdot K_h \cdot K_n \cdot V^2$$  \hspace{1cm} (4.7)$$
The relevant IP components controlling the effects of windborne debris impact are: (1) number of impacting debris, $n_d$; (2) impact linear momentum, $L_d$; and (3) impact kinetic energy, $KE_d$. The impact linear momentum is well correlated with the damage to envelope components with a brittle behavior (e.g., glazing portions of doors and windows (Masters et al. 2010), whereas the impact kinetic energy is better correlated with the damage to envelope components with a ductile behavior such as aluminum storm panels (Herbin and Barbato 2012, Alphonso and Barbato 2014). In this study, only envelope components with brittle behavior are considered.

The analysis step of the interaction analysis phase requires an impact model to evaluate $n_d$ and $L_d$ (Barbato et al. 2013). The debris impact model uses the debris flight path obtained from the trajectory model to check for any windborne debris impact with the target building. In the event of an impact, the horizontal component of the missile velocity and data relative to the missile size and mass (obtained from the debris generation model) are used to compute the impact linear momentum of the missile (i.e., the linear momentum corresponding to the windborne debris velocity component orthogonal to the impacted surface, conditional to the event of at least one impact on vulnerable components). The impact linear momentum is given by:

$$L_d = M_d \cdot A_d \cdot u_d$$

The debris horizontal velocity at impact, $u_d$, is a function of the wind velocity and the distance travelled by the debris (determined by its landing position), and is given by (Lin and Vanmarcke 2008):

$$u_d = V \cdot \left[1 - \exp \left(-\sqrt{2 \cdot C \cdot K \cdot x}\right) \right]$$

in which $x = \frac{g \cdot X}{V^2}$ = dimensionless horizontal flight distance of the debris.
The \( IP \) component relevant to the flood hazard is the height of water due to the surge \((h_s)\) which is calculated as the difference between the surge height \((\zeta)\) and the building ground elevation, which is assumed to be equal to 1 m in this study. The major \( IP \) for the rainfall hazard is the rainfall intrusion height \((h_r)\) is computed as (Pita et al. 2012):

\[
h_r = \frac{IRR \cdot RAF}{A_b} \cdot \left[ \sum_j (d_j \cdot a_j) + a_0 \right]
\]

(4.10)

where \( RAF \) = rainfall admittance factor, \( d_j \) = percentage of damaged area for component \( j \), \( a_j \) = area of component \( j \), \( a_0 \) = area of pre-existing openings in the building, and \( A_b \) = base area of the house.

The rainfall admittance factor accounts for the influence that building geometry exerts on the free-flow rain and measures the fraction of the rain that will actually fall on the building windward envelope (i.e., the impinging rain) (Pita et al. 2012). For low-rise buildings, the \( RAF \) ranges from 0.2 to 0.5 (Straube and Burnett 2000) and is assumed here to follow a uniform distribution.

### 4.4.4 Structural analysis/Damage analysis

In this study, the structural analysis phase is not performed explicitly and the strength of vulnerable components is directly compared to the corresponding \( IP \). This approach is computationally convenient and usually appropriate for non-engineered and pre-engineered structures. Following a procedure commonly used in performance-based earthquake engineering, the physical damage conditions are represented using a limit state function \( LSF \) for each damage limit state, i.e.,

\[
LSF_j = DM_j - IP_j
\]

(4.11)

in which \( DM_j \) correspond to the limit state capacity of component \( j \) for the given damage limit state. The limit states generally considered for residential buildings are: (1) breaking of annealed glass windows/doors, (2) uplift of the roof sheathings, (3) uplift of the roof covers, (4) roof truss
failure, and (5) wall failure. The IPs are compared with the limit state capacity of different components of the building, and if the IPs assume values larger than the corresponding limit state capacity of the building component, the component is assumed to fail. In case of any breach in the building envelope, the iteration is repeated with updated SPs until no additional breach is observed (see Figure 4.5). The statistics of the limit state capacity for different components of the building and their corresponding limit states are given in Table 4.3 (Datin et al. 2010, Gurley et al. 2005, Masters et al. 2010).

Table 4.3 Statistics of the limit state capacity for different components.

<table>
<thead>
<tr>
<th>Component</th>
<th>Limit State</th>
<th>Mean</th>
<th>COV</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof cover</td>
<td>Separation or pull off</td>
<td>3.35 kN/m²</td>
<td>0.19</td>
<td>Normal</td>
</tr>
<tr>
<td>Roof sheathing</td>
<td>Separation or pull off</td>
<td>6.20 kN/m²</td>
<td>0.12</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Door</td>
<td>Pressure failure</td>
<td>4.79 kN/m²</td>
<td>0.2</td>
<td>Normal</td>
</tr>
<tr>
<td>Windows</td>
<td>Pressure failure</td>
<td>3.33 kN/m²</td>
<td>0.2</td>
<td>Normal</td>
</tr>
<tr>
<td>Windows</td>
<td>Impact failure</td>
<td>4.72 kg-m/s</td>
<td>0.23</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Roof to wall connections</td>
<td>Tensile failure</td>
<td>18.28 kN</td>
<td>0.2</td>
<td>Lognormal</td>
</tr>
</tbody>
</table>

The damage states of the target building used in this case study are governed by the performance of the building envelope (damage state of the components) and are divided into five states, varying between 0 (no damage) and 4 (destruction) as shown in Table 4.4 (Vickery et al. 2006, Womble et al. 2006, Li et al. 2011). A rainfall intrusion limit state is used in conjunction with the other limit states for determining the damage state for the contents only.
<table>
<thead>
<tr>
<th>Dam. state</th>
<th>Qualitative damage description</th>
<th>Roof cover (a)</th>
<th>Window/door failures (b)</th>
<th>Roof deck (c)</th>
<th>Roof failure (d)</th>
<th>Wall failure (e)</th>
<th>Surge height (f)</th>
<th>Rainfall intrusion (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>Very minor damage</td>
<td>≤2%</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>None</td>
<td>&gt; 0 cm &amp; ≤ 0.02 cm</td>
</tr>
<tr>
<td>1</td>
<td>Minor damage</td>
<td>&gt;2 &amp; ≤15%</td>
<td>One opening failure</td>
<td>No</td>
<td>No</td>
<td>No</td>
<td>None</td>
<td>&gt;0.02 cm &amp; ≤ 0.25 cm</td>
</tr>
<tr>
<td>2</td>
<td>Moderate damage</td>
<td>&gt;15 &amp; ≤50%</td>
<td>&gt;1 &amp; ≤ the larger of 20% &amp; 3 panels</td>
<td>No</td>
<td>No</td>
<td>&gt;0.01 ft. &amp; ≤ 2 ft.</td>
<td>&gt;0.25 cm &amp; ≤ 1.0 cm</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Severe damage</td>
<td>&gt;50%</td>
<td>&gt; the larger of 20% &amp; 3 &amp; ≤ 50%</td>
<td>No</td>
<td>No</td>
<td>&gt;2 ft. &amp; ≤ 8 ft.</td>
<td>&gt;1 cm &amp; ≤ 2.5 cm</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Destruction</td>
<td>-</td>
<td>&gt;50%</td>
<td>&gt;25%</td>
<td>Yes</td>
<td>Yes</td>
<td>&gt; 8 ft.</td>
<td>&gt;2.5 cm</td>
</tr>
</tbody>
</table>

Thus, the damage state of the building is determined as the worst damage state among limit states (a) through (f), whereas the damage state for the content loss is determined as the worst damage state among limit states (a) through (g) (Table 4.4).

### 4.4.5 Loss analysis

The $DV$ in this case study is the repair cost of the building and its annual probability of exceedance is calculated using the multi-layer MCS (see Figure 4.5). The number of hurricanes in each year is simulated according to a Poisson random occurrence model with annual occurrence rate obtained from the NIST database. For each generated hurricane, a peak wind speed, $V$, is generated randomly according to the Weibull distribution given by Eq. (4.2). For this value of $V$, the wind
pressure is calculated using Eq. (4.6), the number of debris impacts is calculated by comparing the flight trajectory with the position of the target house, the surge height is calculated using Eq. (4.4), and the impinging rainfall rate by Eq. (4.5).

Figure 4.5 Multi-layer MCS approach for probabilistic hurricane loss estimation of residential buildings.

For each debris impact, the corresponding linear momentum is calculated using Eq. (4.8). The IPs are then compared with the limit state capacity of different components of the building, and if the IPs assume values larger than the corresponding limit state capacity of the building component, the component is assumed to fail. The building envelope is checked for any breach, in the event of which the internal pressure is modified. The remaining undamaged building components are checked for further damage due to the modified pressure. The amount of rainfall intrusion through the damaged components is calculated using Eq. (4.10). The damage state of the building is calculated based on the extent of the component damages, the surge height, and the rainfall intrusion (see Table 4.4). The repair cost and the content loss are then generated for the corresponding damaged state according to the probability distributions given in Table 4.5. In this study, it is assumed that the building is fully repaired after each hurricane. Research is ongoing to include the effects of downtime and reconstruction time, which depend on the extent of the damage.
and the time interval between consecutive hurricanes and are needed to estimate the cost associated with loss of use.

The single-year simulation described above is repeated a large number of times (e.g., in this example, 100,000 samples are used) to estimate the annual probability of exceedance (which coincides with the complementary cumulative distribution function of the $DV$) of the total loss.

Table 4.5 Repair cost (% of building cost) and content loss (% of total content value) for different damage states

<table>
<thead>
<tr>
<th>Damage state</th>
<th>Mean</th>
<th>COV</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.2%</td>
<td>0.2</td>
<td>Lognormal</td>
</tr>
<tr>
<td>2</td>
<td>2%</td>
<td>0.2</td>
<td>Lognormal</td>
</tr>
<tr>
<td>3</td>
<td>10%</td>
<td>0.2</td>
<td>Lognormal</td>
</tr>
<tr>
<td>4</td>
<td>30%</td>
<td>0.2</td>
<td>Lognormal</td>
</tr>
<tr>
<td>5</td>
<td>70%</td>
<td>0.2</td>
<td>Lognormal</td>
</tr>
</tbody>
</table>

Figure 4.6 plots, in a semi-logarithmic scale, the annual probabilities of exceedance of the loss for the target building for different hazard scenarios. The expected annual loss (EAL), which is defined as the average economic loss that occurs every year in the building (Raul and Vitelmo 2004) and is equal to the area under each annual probability of exceedance curve, is also computed and shown in Figure 4.6 for each of the different hazards and their interaction.

From the results presented in Figure 4.6, it is observed that the loss due to surge hazard is predominant for repair costs lower than about $60,000, while the loss due to wind hazard is predominant for repair costs higher than about $60,000. This behavior can be explained by comparing the hazard curves for surge and wind hazard shown in Figure 4.3 with the damage states corresponding to these two hazards.
In particular, it is observed that storm surge values that can cause even significant damage to the structure have an annual probability of occurrence that is similar to wind speed values for which it is unlikely to have significant structural damage. However, the annual probability of occurrence for storm surge decreases significantly faster than the annual probability of occurrence of wind speed. Similarly, in comparison with the wind hazard, windborne debris hazard has a larger effect on loss for values lower than about $30,000, because the probability of damage to the windows due to windborne debris is lower than that due to wind pressure at lower wind speeds. It is also observed that the annual probability of loss exceeding the sum of the building cost and its content value is small but not negligible. This phenomenon is most likely due to (1) the assumption that the building is fully repaired after each hurricane and that more than one hurricane can take place in a single year, and (2) the assumption of lognormal distribution for the loss corresponding to a given damage state.
In terms of EAL, it is observed that, for the example considered in this study, the losses due to surge hazard are significantly higher than those due to other hazards. In addition, the EAL due to the interaction of all hazards is about 5% higher than the sum of the EALs due to each individual hazard. This result suggests a moderate level of interaction among the different hazards for the case study considered here. The fact that the interaction is only moderate is most likely due to the predominance of the surge hazard effects on expected losses. However, the level of interaction among different hazards can be significant when the hazard effects on losses are of similar magnitude (see Barbato et al. 2013). Thus, in general, the multi-hazard nature of hurricane events must be taken into account for accurate probabilistic loss analyses.

4.5 Conclusions

In this paper, the Performance-Based Hurricane Engineering (PBHE) framework is applied to risk assessment of structures subjected to combined wind, windborne debris, flood, and rainfall hazards. Risk assessment analysis is disaggregated into the following basic probabilistic components: (1) hazard analysis, (2) structural characterization, (3) interaction analysis, (4) structural analysis, (5) damage analysis, and (6) loss analysis. In contrast to other existing performance-based engineering approaches, which considers explicitly only single hazards, the PBHE framework accounts for the multi-hazard nature of hurricane events by considering independent hazards, interacting hazards, and hazard chains. The general multi-layer MCS approach is specialized for the risk assessment of pre-/non-engineered buildings such as single-family residential buildings.

The PBHE framework is illustrated through of the risk assessment analysis for a target building in a hypothetical residential development in Panama City, Florida. The annual probabilities of loss exceedance and the expected annual loss of the target building are computed for different
individual hazards and their interaction. For the specific example considered in this paper, it is observed that the loss due to surge hazard is predominant for lower loss levels, whereas the loss due to wind hazard is predominant for higher loss levels. It is also observed that the interaction among different hazards is only moderate, because of the overall predominance of losses due to surge hazard when compared to the losses due to other hazards. However, in general, the multi-hazard nature of hurricane events needs to be taken into account for accurate probabilistic hurricane risk assessment, particularly when the losses due to different hazards are similar.

It is noteworthy that the presented probabilistic methodology differs from the HAZUS-MH approach because it is concerned with the design and/or retrofit of specific buildings and structures, whereas HAZUS-MH can be used to perform loss analysis for a region or for a hurricane event. Thus, the PBHE framework can be used for design and/or loss assessment of specific buildings and structures. This framework can also be used to compare the cost effectiveness of different storm mitigation strategies and can assist owners, insurers, designers, and policy makers in making informed decisions on design and retrofit of specific structures subject to hurricane hazard.

4.6 References


ASCE. (2010). "Minimum design loads for buildings and other structures." American Society of Civil Engineers, Reston, VA.


CHAPTER 5
CONCLUSIONS

5.1 Summary

Hurricanes are among the most costly natural hazards affecting mankind and cause severe damage to the society both in terms of property damage and loss of life. Hence, rigorous design and risk assessment methodologies are required to reduce the damage and for cost effective risk management of structures subjected to hurricanes. PBE approach is a rational way of assessing and reducing risk for engineering facilities subject to natural and man-made hazards, and has been used successfully in other subfields of structural engineering such as earthquake engineering, wind engineering blast engineering etc. Thus, the development of a Performance-Based Hurricane Engineering (PBHE) methodology could contribute to reduce the societal and economic losses associated with hurricanes and improve the resilience of the nation. However, the development of a general PBHE methodology presents several additional challenges when compared to other existing PBE methodologies. In fact, while the existing PBE methodologies focus on single hazards, the landfall of a hurricane involves different hazard sources (i.e., wind, windborne debris, flood, and rain) that interact to generate the hazard scenario for a given structure and to determine its global risk. This thesis presents a fully probabilistic PBHE methodology based on total probability theorem for the performance-based risk assessment and design of structures subjected to hurricane hazards.

In the following sections, the major conclusions of the research work presented in this thesis are summarized and future research directions are suggested.

5.2 Conclusions

Chapter 2 presented the general PBHE framework and discussed the interaction among the multiple hazards, and proposed a scheme for representing the uncertainties from all pertinent
sources and their propagation through a probabilistic performance assessment analysis. In addition, the feasibility of the proposed framework was demonstrated through an application example consisting in the risk assessment for a target building in a hypothetical residential development under three different hazard scenarios. It was observed that the interaction between wind and windborne debris hazard can affect significantly the value of the annual probability of exceedance of repair cost. This observation suggests the need to consider the multi-hazard nature of hurricane events for accurate probabilistic loss analysis.

In Chapter 3, the PBHE framework was specialized for hurricane risk assessment of low-rise residential buildings. The chapter focused on the hurricane loss analysis of residential buildings and the effects of mitigation techniques for wind and windborne debris hazards on the structural performance. A highly efficient modification of the multi-layer Monte Carlo simulation (MCS) technique based on copula was proposed for faster re-evaluation of hurricane risk. The proposed faster re-analysis multi-layer MCS method was used in conjunction with cost/benefit analysis to compare different hazard mitigation technique and design alternative. A realistic case study consisting of an actual residential development located in Pinellas County, FL, was presented to illustrate the framework. For the specific example considered, it was found that, for hurricane induced loss, the loss due to windborne debris hazard is predominant for lower loss levels, whereas the loss due to wind hazard is predominant for higher loss levels; and for non-hurricane induced loss, windborne debris hazard is negligible. The proposed faster re-analysis approach was validated based on the corresponding results obtained using the original multi-layer MCS. Additionally, the cost-effectiveness of different hurricane hazard mitigation techniques and design alternatives typically used for low-rise residential buildings were compared. For the specific application example considered, it was observed that, among the different types of retrofits
compared in this study, the most economically viable form of retrofit is the use of roof re-nailing with an 8d R6/6 pattern, whereas the use of masonry walls was not economically viable.

In Chapter 4, a specialized PBHE framework for engineered building typology subjected to wind hazard was developed. The different uncertainty parameters involved in the analysis of this building typology were identified and addressed. The framework was illustrated through an application focused on the performance assessment of a tall building subjected to both hurricane and non-hurricane wind hazard. Direct losses due to structural and non-structural damage, and building occupant discomfort were calculated. For the specific application example considered, it was observed that the losses due to occupants’ discomfort were predominant for lower loss levels, whereas the losses due to structural and non-structural damage were predominant for higher loss levels.

In Chapter 5, the PBHE framework specialized for pre-/non-engineered buildings was extended for the risk assessment of structures subjected to combined wind, windborne debris, flood, and rainfall hazards. The framework was illustrated through the risk assessment analysis for a target building in a hypothetical residential development in Panama City, Florida. The annual probabilities of loss exceedance and the expected annual loss of the target building were computed for different individual hazards and their interaction. For the specific example considered, it was observed that the loss due to surge hazard was predominant for lower loss levels, whereas the loss due to wind hazard was predominant for higher loss levels. It was also observed that the interaction among different hazards is only moderate, because of the overall predominance of losses due to surge hazard when compared to the losses due to other hazards. However, in general, the multi-hazard nature of hurricane events needs to be taken into account for accurate probabilistic hurricane risk assessment, particularly when the losses due to different hazards are similar.
The PBHE framework presented in this work can be used to compare different design solutions in terms of structural performance, to assess the performance of existing structures in hurricane prone regions, and to perform life cycle cost-benefit analyses for different retrofit and storm mitigation strategies. The proposed framework offers significant potential benefits for owners, builders, and the insurance industry, due to its capability to accurately estimate the potential losses due to hurricanes. It is noteworthy that the proposed probabilistic approach is consistent with the state-of-the-art in hurricane hazard and loss modeling, and at the same time is flexible, since it is based on the total probability theorem, which allows for independence of the different analysis components. The author believes that the proposed PBHE framework is a major step towards a performance-based design methodology, which allows for the design of structures that satisfy the performance objectives selected by the owners or other relevant stakeholders beyond the minimum requirements of traditional prescriptive design codes.

5.3 Future work

Based on the research work performed and presented herein, further work is recommended in the following areas:

1. Studies have shown that increased hurricane activity is possible as a result of the changing global climate change and may have a substantial impact on the damage and loss estimation in coastal areas. Hence, additional research is required to investigate the effects of climate change on the risk assessment of structures subject to weather-related hazards such as hurricanes and to incorporate these effects into the PBHE framework.

2. Fragility curves are one of the fundamental elements of the PBHE framework, providing the link between the response and the damage of the structure and its components. The study and the development of fragility curves for different structural and non-structural components of
different structural and infrastructure systems subject to different hazards is, thus, a necessary requirement for the success and applicability of the PBHE framework proposed in this study.

3. In this research work, a frequency domain approach was used for the structural analysis of tall buildings. However, a time domain approach could be required, e.g., to include the effects of fatigue loads and/or nonlinearities in the response of the buildings. Research in this direction is recommended.

4. Each analysis phase of the PBHE framework involves different models with different levels of accuracy. Thus, there is a clear need to acknowledge and properly address epistemic uncertainties within the framework. An interesting research direction would be to incorporate model uncertainty into the framework and include confidence intervals for the results.

5. Post-hurricane damage studies have indicated that light vegetation are one of the primary source of windborne debris in residential areas. The debris generation and trajectory model for these kinds of windborne debris should be investigated and incorporated into the framework for a more realistic assessment of windborne debris hazard.

6. Studies have indicated that windborne debris causes extensive damage to the facade of tall buildings exposed to high winds, and the rainfall ingression through the openings cause significant interior loss. Hence, it would be very important to include the effects of other hazard sources such as windborne debris, flood, and rainfall for the accurate risk assessment of tall buildings.

7. In this study, Gaussian copulas and Frank’s copulas were adopted for constructing joint probability distributions of random variables with known marginal distribution and correlation coefficients. The efficiency of different copulas, e.g., in modeling the dependence structure of the variables for use in the faster re-analysis multi-layer MCS method should be investigated.
8. The models used for characterizing the hazards may affect the final result from the PBHE framework. Hence, the effects of using hazard models of different complexity (e.g., using a joint probability distribution model for wind and surge height instead of empirical equations, direct modeling of hurricane trajectories instead of simulation of the peak wind speed based on historical records) should be investigated.

9. Sensitivity analysis could be used to identify the important parameters involved in a risk assessment/loss analysis, as well as to quantify how much the variability of these factors could affect the final result of a probabilistic loss analysis.
APPENDIX A

PERFORMANCE-BASED HURRICANE ENGINEERING (PBHE) FRAMEWORK

A.1 Introduction

Performance-Based Engineering (PBE) is a general methodology that (1) defines the performance objectives for structural systems during their design life, (2) provides criteria and methods for verifying the achievement of the performance objectives, and (3) offers appropriate methodologies to improve the design of structural systems. In the last two decades, significant research efforts have been devoted to the development of PBE in earthquake engineering (Ellingwood 2006, Porter 2003), and have led, e.g., to the Performance-Based Earthquake Engineering (PBEE) framework implemented by the Pacific Earthquake Engineering Research (PEER) Center (Cornell and Krawlinker 2000, Porter 2003,). More recently, the civil engineering community has shown significant interest toward the possible development and extension of PBE to other subfields of structural engineering (Augusti and Ciampoli 2008). In particular, Performance-Based Blast Engineering has received considerable attention in the US after the terrorist attacks of September 11, 2001 (Hamburger and Whittaker 2003). Other PBE examples are Performance-Based Fire Engineering (Lamont and Rini 2008), Performance-Based Tsunami Engineering (Riggs et al. 2008), and Performance-Based Wind Engineering (PBWE) (Petrini 2009, Ciampoli and Petrini 2012). In earthquake engineering, modern design codes have gradually substituted prescriptive approaches with PBEE procedures for the design of new facilities and the retrofit of existing ones (ATC 1997, ATC 2005). PBEE has been shown to facilitate design and construction of structural systems based on a realistic and reliable assessment of the risk associated with seismic hazard, thus leading to a more efficient use of resources for construction, maintenance, and retrofit of structures (Krishnan et al. 2006, Stojadinovic et al. 2009).
The advantages demonstrated by a PBE approach to civil engineering provide a strong motivation to develop a PBE methodology for structures subjected to hurricanes. The need for assessing and improving the resilience of the built environment subjected to hurricane hazard is widely recognized. Some initial interest in PBE has been expressed in hurricane engineering (van de Lindt and Dao 2009, Barbato et al. 2011, Kareem and McCullough 2011), but a complete and rigorous framework is still needed.

The development of a Performance-Based Hurricane Engineering (PBHE) methodology presents several additional challenges when compared to other existing PBE methodologies. In fact, while other PBE methodologies focus on single hazards, the landfall of a hurricane involves different hazard sources (wind, windborne debris, flood, and rain) that interact to generate the hazard scenario for a given structure and to determine its global risk. Thus, hurricanes can be viewed, and must be analyzed, as multi-hazard scenarios. In addition, monetary losses due to structural and non-structural damage assume more relevance for hurricane events than for other types of hazard (e.g., earthquakes) for which no (or very short) warning is available. Therefore, for hurricane hazard, performance levels related to limitation of the monetary losses due to damage may be required for a large portion of existing or newly designed structures.

During the last decade, significant attention has been also devoted to multi-hazard scenarios (Wen 2001, Whittaker et al. 2003, Li and Ellingwood 2009). Multi-hazard scenarios raise non-trivial issues mainly related to the following three problems: (1) modeling the interaction among concurrent sources of hazard; (2) calibrating design values having comparable occurrence rates for different hazards; and (3) balancing the design in order to attain similar safety levels with regard to multi-hazard scenarios implying hazards that, if taken separately, would drive design solutions
in different (and even opposite) directions (e.g., increasing the elevation of the structure as a safe
guard against flood may result in increased wind loads).

In this chapter, the PBE approach is formally extended to develop a fully probabilistic PBHE
methodology. The interaction among the multiple hazards is discussed, and a scheme for
representing the uncertainties from all pertinent sources and their propagation through a
probabilistic performance assessment analysis is proposed. Analytical models of the relevant
environmental phenomena generated by hurricane events are briefly described. The chapter
includes suggestions for candidate parameters for the probabilistic characterization of: (1) the
interaction between the structure and the hazard sources; (2) the structural response; (3) the
resulting structural damage; and (4) the consequences of the structural damage. The proposed
approach is illustrated through an application focused on the performance assessment of a
residential building subjected to both wind and windborne debris hazard.

A.2 Proposed PBHE framework

In a PBE approach, the structural risk is conventionally measured by the probability of exceeding
(within a given reference period usually taken as one year) a specified value of a decision variable,
$DV$, corresponding to a target performance. Each $DV$ is a measurable attribute of a specific
structural performance and can be defined in terms of cost/benefit for the users and/or the society
(e.g., loss of human lives, economic losses, exceedance of safety/serviceability limit states). An
assessment based on PBE provides a probabilistic description of the appropriate $DV$ for different
design choices in order to allow a rational decision among different design options.

A PBE procedure for structures subject to hurricane hazard can be decomposed into elementary
phases that must be carried out in sequence. Perhaps the most important expected feature of the
procedure is the qualitative independence of each phase from the others (i.e., the choice of the
parameters that are characteristic for a given phase is independent from the parameters adopted in the previous phases). The probabilistic PBHE framework proposed in this chapter is based on the total probability theorem, similar to the PEER PBEE and the PBWE frameworks. The structural risk is defined in terms of a given $DV$ as follows

$$G(DV) = \int \int \int G(DV|DM) \cdot f(DM|EDP) \cdot f(EDP|IM,IP,SP) \cdot f(IP|IM,SP) \cdot f(IM) \cdot f(SP) \cdot dDM \cdot dEDP \cdot dIP \cdot dIM \cdot dSP$$  \hspace{1cm} \text{(A.1)}$$

where: $G(\cdot)$ = complementary cumulative distribution function, and $G(\cdot|\cdot)$ = conditional complementary cumulative distribution function; $f(\cdot)$ = probability density function, and $f(\cdot|\cdot)$ = conditional probability density function; $DM$ = damage measure (i.e., a parameter describing the physical damage to the structure); $EDP$ = engineering demand parameter (i.e., a parameter describing the structural response for the performance evaluation); $IM$ = vector of intensity measures (i.e., the parameters characterizing the environmental hazard); $SP$ = vector of structural parameters (i.e., the parameters describing the relevant properties of the structural system and non-environmental actions); and $IP$ = vector of interaction parameters (i.e., the parameters describing the interaction phenomena between the environment and the structure). In Eq. (A.1), $IM$ and $SP$ are assumed as uncorrelated and independent of $IP$, while $IP$ is dependent on both $IM$ and $SP$. The extension to the case of vectors of $DM$ and $EDP$ is straightforward.

By means of Eq.(A.1), the problem of risk assessment is disaggregated into the following tasks (see Figure A.1): (1) hazard analysis, (2) structural characterization, (3) interaction analysis, (4) structural analysis, (5) damage analysis, and (6) loss analysis. Detailed explanation of steps (1), (4), (5), and (6) can be found in the PBEE literature, while steps (2) and (3) have been introduced in PBWE to rigorously model the effects on the structural response of the interaction between the structural system and the environment (e.g., the aerodynamic effects, see (Ciampoli et al. 2011)).
In particular, the probabilistic hazard analysis phase (i.e., the probabilistic characterization of \( IM \)) can be performed by using the (joint) probability density function \( f(IM) \). The \( IM \) should be chosen as strictly independent on the investigated structure. Thus, the probabilistic information about \( IM \) should be provided by meteorologists, climatologists, and other experts in atmospheric sciences, while the engineers have the task of clarifying what information is needed.

![Diagram of probabilistic analysis components in the proposed PBHE framework.]

**Figure A.1** Probabilistic analysis components in the proposed PBHE framework.

### A.3 Characterization of Uncertainties

The PBHE framework described in the previous section requires the identification of the uncertainties that affect the structural performance and the evaluation of the interaction phenomena occurring among the different hazards and the structure. It is noted here that uncertainties can be classified into two different categories, i.e., aleatoric uncertainties, which are due to natural variability of physical, geometrical, and mechanical properties, and epistemic uncertainties, which are due to lack of knowledge, imprecise modeling, and limited statistical information (Melchers 2002). Aleatoric uncertainties are inherent in nature and, thus, are virtually irreducible. On the
contrary, epistemic uncertainties can and should be reduced as much as possible, e.g., by implementing more accurate and realistic models. While epistemic uncertainties can significantly affect the confidence on the end results of the PBHE framework proposed in this study, their detailed study is beyond the scope of this chapter.

This paper focuses on the characterization of the hazard for a single structure located in a hurricane-prone region. Three different zones can be distinguished (Ciampoli et al. 2011) (see Figure A.2):

1. The *environment*, i.e., the space surrounding the structure but sufficiently far from it, where the parameters describing the wind field and the other hurricane-related environmental actions are not influenced by the presence of the structure itself.

2. The *exchange zone*, i.e., the space immediately surrounding the structure, where the structural configuration and the environmental action are strongly correlated, and the interaction between the structure and the environmental agents, as well as the presence of adjacent structures, cannot be disregarded.

3. The *structural system*, which includes the structure (characterized by a set of uncertain parameters collected in a vector $S$) as well as the non-environmental actions and/or elements that can modify the structural behaviour (characterized by a set of uncertain parameters collected in a vector $A$).

Hereinafter, the uncertain basic parameters of interest describing the environmental actions in the environment are collected in the vector IM; the uncertain basic parameters describing the structural system and non-environmental actions or devices applied to the structure are collected in the vector $SP$; and the uncertain (usually derived) parameters of interest in the exchange zone are collected in the vector $IP$ (Figure A.2). Examples of $IP$ are the aerodynamic and hydrodynamic coefficients,
as well as the parameters defining the impact energy of windborne debris. The uncertain parameters $IP$ describing the exchange zone can be chosen so that they do not affect directly the uncertain parameters characterizing both the environment ($IM$) and the structural system ($SP$). Instead, uncertainty propagation from the structural system and the environment to the exchange zone is likely (e.g., the integrity of the building envelope affects the values of the wind pressure acting on the building surfaces).

Figure A.2 Identification of the uncertain parameters needed to describe the interaction between environment and structure in PBHE.

In Figure A.2, the different sources of uncertainties corresponding to the environment, the structural system, and the exchange zone are shown, and the different hazard sources and their interaction identified. The environmental hazard due to a hurricane event in a specified geographic region is generated by the following four main sources of hazard:

1. Hurricane strong winds (described by the uncertain vector $W$), which can produce wind damage (wind hazard).

2. Water bodies (described by the uncertain vector $F$), which can produce flood damage (flood hazard).
3. Sources of windborne debris (described by the uncertain vector $D$), which can produce windborne debris damage (windborne debris hazard).

4. Rainfall rates (described by the uncertain parameter vector $RA$), which can induce flash flooding and direct damage to the interiors of building when the building envelope has been breached (rainfall hazard).

These various sources of hazard usually interact to produce the actual hurricane hazard for a given structure. Typical examples are the interaction between wind and waves in offshore sites, or the interaction between storm surge and wind in coastal regions. A set of uncertain parameters included in vectors $W$, $F$, $D$, and $RA$ must be selected in order to describe the multiple hazards using $IM$. This set must accurately describe all pertinent hazard sources and must be as small as possible in order to be both “sufficient” and “efficient” (Luco and Cornell 2007). An analogous selection operation (for vectors $A$ and $S$) is needed to describe the structural behavior by $SP$. To derive the probabilistic characterization of the hurricane actions on a structural system, the proposed PBHE methodology also requires the identification of the vector $IP$ of the stochastic parameters describing the interaction between the environment and the structural system in the exchange zone.

A.4 Multi-Hazard Characterization of Hurricane Events

Unlike other existing PBE engineering methodologies, in which only a single hazard source is considered (e.g., PBEE and PBWE, which consider earthquake and wind hazard only, respectively), the proposed PBHE framework innovatively accounts for concurrent and interacting hazard sources, i.e., storm surge and water bodies that can cause flooding, windborne debris, rainfall, and strong winds. It also accounts for the possible sequential effects of these distinct
hazards. The multi-hazard nature of the phenomena related to hurricanes and their effects on the built environment can manifest in the following three different modalities (Petrini and Palmeri 2012):

1. **Independent hazards**, when different hazards affect the structure independently. For example, windborne debris and flood hazard can be considered as independent of each other because no mutual interaction between the two hazards has the effect of modifying the intensity of the corresponding actions. These hazards can occur individually or simultaneously.

2. **Interacting hazards**, when the actions produced on a structure by different hazards are interdependent (e.g., wind and windborne debris hazards).

3. **Hazard chains**, when the effects of some hazards modify sequentially the effects of other hazards. For example, the actions on a structure due to windborne debris can damage the structural envelope and increases the vulnerability of the structure to strong winds.

In the proposed framework, the first two cases (i.e., independent and interacting hazards) are treated within the hazard analysis, by assuming proper interaction models between the hazards (e.g., by using a proper joint probability distribution function to describe the variability of the IM for different hazards (Petrini and Palemeri 2012, Toro et al. 2010). The study of hazard chains requires modeling the structural system configuration and properties as a function of the level of structural damage caused by the different hazards. In particular, the presence of a hazard chain implies that the SP can change as a consequence of DM exceeding certain thresholds. Thus, structural characterization, interaction analysis, and structural analysis cannot be carried out without any information or assumption on the values of DM. It is noteworthy that the proposed probabilistic approach is consistent with the state-of-the-art in hurricane hazard and loss modeling, which can be identified with the HAZUS® methodology (FEMA 2007).
However, the proposed PBHE framework presents the following major differences when compared with the HAZUS methodology:

1. HAZUS is a GIS-based natural hazard assessment software used for the regional risk and loss assessment of structures. Although it is possible to use the HAZUS software for individual buildings, the corresponding results can only estimate the average loss for the class of buildings that are similar to the one under consideration. By contrast, the proposed PBHE framework is specifically developed to account for the characteristics of an individual building. Thus, it has the potential to provide more accurate results.

2. HAZUS is not intended for use as a structural design tool. On the contrary, the proposed PBHE framework is the first step toward a performance-based design methodology, which includes the performance-based assessment procedure described in this chapter.

3. HAZUS approximates the multi-hazard nature of the hurricane events as a simple superposition of various effects produced by different sources of hazards, i.e., wind, windborne debris, flood and rainfall. The proposed PBHE framework directly models the multi-hazard nature of hurricanes by taking into account also the effects due to the interaction between different hazard sources.

4. The proposed PBHE framework is significantly more flexible than HAZUS, since it is based on the total probability theorem, which allows for independence of the different analysis components. This property permits to take advantage of the state-of-the-art knowledge in the research subfields involved in the assessment and design of structures located in hurricane prone regions, e.g., in climatology, structural analysis, structural design, material technology, and loss analysis.
A.5 Performance Expectations

In PBE, several performance expectation levels are defined based on the severity of structural and non-structural damage and the corresponding losses (e.g., see Petrini 2009, Van de Lindt and Dao 2009, Wen 2001). Commonly, two main performance expectation levels with each level having different performance objectives are identified (Petrini 2009, Ciampoli et al. 2011), i.e., a high level performance expectation (related to serviceability requirements) and a low level performance expectation (related to structural safety and/or integrity). For the PBHE framework proposed in this chapter, additional considerations are needed to account for the fact that early warning of the population is possible in case of hurricane hazard, in contrast with other hazards (like the seismic hazard) for which warning is impossible or very limited. Thus, empty buildings during the hurricane transit are not rare. In this situation, significant losses due to the damage to non-structural components (e.g., building envelope, interiors of the building) can occur without problems for people (because occupants left the building) or for the structural integrity (because the structural parts do not suffer damages). In view of this consideration, an additional intermediate performance level related to non-structural damage is introduced.

The three performance expectation levels can be further subdivided in sub-levels or performance levels, e.g., the high performance expectation level for a building can be related to occupant comfort (higher) and/or continued occupancy (lower). Moreover, different performance expectation levels need to be defined for different structural typologies (e.g., buildings, bridges). A possible list of performance expectations for buildings and their short description are provided in Table A.1.
Table A.1 Classification of building performance expectation for PBHE.

<table>
<thead>
<tr>
<th>Category</th>
<th>Level</th>
<th>Description</th>
<th>Damage level</th>
</tr>
</thead>
<tbody>
<tr>
<td>High: comfort and safety of occupants</td>
<td>Occupant comfort</td>
<td>No or little discomfort to the building occupants</td>
<td>No damage</td>
</tr>
<tr>
<td></td>
<td>Continued occupancy</td>
<td>No threat to safety of building occupants, small economic losses</td>
<td>Minor exterior damage, no interior damage</td>
</tr>
<tr>
<td>Intermediate: damage to non-structural elements</td>
<td>Limited damage to envelope/content</td>
<td>No threat to safety of building occupants, some economic losses</td>
<td>Exterior damage, minor interior damage</td>
</tr>
<tr>
<td></td>
<td>Extensive damage to envelope/content</td>
<td>Safety of building occupants is jeopardized, significant economic losses</td>
<td>Significant exterior and interior damage</td>
</tr>
<tr>
<td>Low: structural integrity</td>
<td>Structural damage</td>
<td>Structural integrity is jeopardized, reduced safety</td>
<td>Structural components and/or connections are damaged</td>
</tr>
<tr>
<td></td>
<td>Extensive structural damage</td>
<td>Visible signs of structural distress, structure is not safe</td>
<td>Loss of integrity of structural components, significant reduction or loss of bearing load capacity</td>
</tr>
</tbody>
</table>

A.6 Description of the Analysis Steps

This section presents a brief description of the analysis steps of the PBHE framework. Particular emphasis is given to the differences between the PBHE framework and other existing PBE frameworks.

A.6.1 Hazard analysis

The hazard analysis provides the probabilistic description of the intensity measures $IM$. A comprehensive vector $IM$ is obtained by considering the components of the basic random parameter vectors $W, F, D$, and $RA$ that describe the different sources of hurricane hazard (Figure A.2). It is noteworthy that, for a specific structure $s$ and a specific performance objective $p$, the
elements of $IM$ that do not represent a relevant hazard for $s$ and/or have small influence on $p$ can be neglected or treated as deterministic. The reduced vector $IM_{s,p}^{(s,p)}$ (i.e., the vector $IM$ specialized for the structure $s$ and the performance $p$) can be used more efficiently than the vector $IM$ at a negligible loss of sufficiency. For example, the flooding hazard can be neglected in the case of structures that are sufficiently far from water bodies.

The selection of the $IM$ components strictly depends on the choice of the (usually deterministic) models used to describe the environmental phenomena related to the various hazards, as illustrated in the available technical literature. In general, the key parameters of these models are treated as stochastic variables. In this chapter, the $IM$ components are identified by selecting state-of-the-art models for hurricane-related environmental phenomena. While the selected $IM$ depend on the specific models, the approach proposed here to identify $IM$ is general and can also be applied to different hazard models.

### A.1.1.1 Wind field and wind hazard characterization

A model of the wind field associated with a hurricane is needed to characterize the wind hazard. The following three methodologies (of increasing complexity) can be adopted to define the hurricane wind field and the corresponding hazard (FEMA 2007):

1. The statistical description of the gust wind velocity, $V$, at the structural location is directly derived from existing data by fitting a proper probability distribution (Li 2005).

2. The site specific statistics of some fundamental hurricane parameters are obtained, and a Monte Carlo approach is used to sample these parameters from the statistical information. Using the sampled values, a mathematical representation of the hurricane is obtained, and the statistics of the parameters that describe the hurricane actions are evaluated for the structure of interest at its specific location (Vickery and Twisdale 1995).
3. The full track of the hurricane is modelled, from its initiation over the ocean until final
dissipation (Jakobsen and Madsen 2004). Several tracks are simulated. The statistics of the
parameters describing the hurricane actions are estimated from the parameter values in each
simulated track for the structure of interest at its specific location.

The different methodologies provide different vectors $IM$. In particular, the first methodology
gives $W = V$, while the other two methodologies give (Barbato et al. 2011):

$$W = \begin{bmatrix} \text{RMW} & V_c & \Delta p_c & B & H^* & z_0 \end{bmatrix}^T \quad (A.2)$$

where: $\text{RMW} =$ radius of maximum wind (defined as the radial distance between the storm center
and the maximum wind location); $V_c =$ translational speed of the center of the storm; $\Delta p_c =$
hurricane central pressure deficit; $B =$ Holland parameter (Vickery et al. 2009); $H^* =$ atmospheric
boundary layer height; $z_0 =$ terrain roughness length; and the superscript $T$ denotes the matrix
transpose operator.

A.6.1.1 Flood hazard characterization

The flood hazard due to the presence of water bodies surrounding the structure depends on the
total water surface elevation with respect to the mean surface, $\eta_{\text{tot}}$, and on the flooding water
velocity, $V_{\text{water}}$ (i.e., the value of the component of the water velocity orthogonal to the flooding
barriers). These two parameters allow for the computation of the volumetric rate of flow and can
be used as synthetic indicators of the flood intensity. The basic parameters characterizing these
indicators can be selected as the components of $F$.

Three main natural phenomena cause water level increase and contribute to flood hazard: the
astronomical tide ($\eta_{\text{tide}}$), the waves ($\eta_{\text{wave}}$), and the storm surge ($\eta_{\text{surge}}$). The total water surface
elevation is the sum of the three contributions at the same instant of time, i.e.,
$$\eta_{\text{tot}} = \eta_{\text{tide}} + \eta_{\text{wave}} + \eta_{\text{surge}}.$$ The flooding water velocity, $V_{\text{water}}$, can be assumed, as a first approximation, equal to the
highest velocity for each of the three considered phenomena ($V_{\text{tide}}$, $V_{\text{wave}}$, and $V_{\text{surge}}$, respectively).

Specific basic parameters subvectors can be defined for each of the three contribution (i.e., $F_{\text{tide}}$, $F_{\text{wave}}$, $F_{\text{surge}}$), and the vector $F$ obtained as the union of the three subvectors.

The flood hazard due to the astronomical tide can be characterized by the two random variables $\eta_{\text{tide}}$ and $V_{\text{tide}}$, i.e., $F_{\text{tide}} = [\eta_{\text{tide}} V_{\text{tide}}]^T$. The individual characterizations of the flood hazard due to waves and storm surge require more detailed considerations. The water level, $\eta_{\text{wave}}$, and the wave speed, $V_{\text{wave}}$, can be directly related to: the water depth, $d$; the wave height, $H$; the wave length, $L$; and the wave period, $T$. The last three quantities can be obtained by propagating in space and time (Dean and Dalrymple 2004) the waves corresponding to a given wave energy density spectrum (e.g., JONSWAP, Hasselmann 1973) valid for the sea waves as determined by the wind field (i.e., $\text{RMW}$, $V_c$, $\Delta p_c$, and $B$), as well as by other parameters (Dean and Dalrymple 2004). A storm surge is defined as the water surface response to wind-induced surface shear stress and pressure fields. Storm surges can produce considerable short-term increases in water level. Current storm surge models are based on the depth-averaged momentum and continuity equations for steady long waves under the hypothesis of incompressible water (Bode 1997).

Based on the existing literature, the following vector $F$ is suggested for a suitable flood hazard characterization

$$F = \left[ \text{RMW} \ V_c \ \Delta p_c \ B \ H^+ \ z_0 \ \eta_{\text{tide}} \ d \ U_{\text{curr}} \ z_b \right]^T \quad (A.3)$$

where $U_{\text{curr}} = \text{current velocity}$, and $z_b = \text{sea bottom friction roughness}$.

### A.6.1.2 Windborne debris hazard characterization

The vector $D$ of intensity measures for windborne debris hazard describes the intensity of the wind field (needed to determine the impact wind speed), and the characteristics of the windborne debris that could affect the structure. The additional parameters needed to describe the debris are: the
density of upwind buildings with respect to the investigated structure, \( n_{\text{buildings}} \); the properties of the different (potential) debris types, e.g., \( M_d \) = mass per unit area of the debris, \( C_{D,d} \) = drag coefficient of the debris (and/or other parameters describing the debris flight characteristics), and \( A_d \) = reference area of the debris; and the resistance model for the missile sources (which contributes to determine the number of windborne debris). The following vector \( D \) is suggested in this study:

\[
D = \begin{bmatrix}
RMW & V_c & \Delta p_c & B & H^* & z_0 & n_{\text{buildings}} & \eta_{\text{vegetation}} & M_d & C_{D,d} & A_d
\end{bmatrix}^T
\]  

(A.4)

A.6.1.3 Rainfall hazard characterization

The high rainfall rate associated with hurricane events can induce significant damage to the interior of buildings when the building envelope has been breached (Huang et al. 2001). To the best of the authors’ knowledge, no analytical rainfall hazard model is available in the technical literature. However, several models based on the interpolation of statistical data define the correlation between the rainfall rate and other fundamental hurricane parameters. One of the more widely accepted and used models is the one implemented in HAZUS® (Vickery et al. 2006, FEMA 2007), which is valid for tropical cyclones. The estimates of rainfall rates resulting from this model are employed in HAZUS® to evaluate the amount of water that enters the buildings through broken windows and glass doors, while they are not used to assess the risk associated with inland flash flooding. Consistently with HAZUS®, this study does not consider inland flash flooding hazard.

The proposed vector \( RA \) of basic random parameters is given by

\[
RA = \begin{bmatrix}
RMW & V_c & \Delta p_c & B & H^* & z_0 & \dot{p}_c
\end{bmatrix}^T
\]  

(A.5)

where \( \dot{p}_c \) is the first time derivative of the hurricane central pressure.
A.6.2 Structural characterization

The step of structural characterization in the PBHE framework provides the probabilistic description of the components of $SP$, which define the geometrical and/or mechanical properties of the structure that characterize its response to environmental and man-made loading. Uncertainties affecting $SP$ are well-known and have been extensively investigated in the past decades for ordinary buildings (Lungu and Rackwitz 2001). They are usually identified as the parameters determining the structural resistance and stiffness (Lee and Rosowsky 2005). However, parameters describing shape, size, and orientation of structural components can also be considered, since they can affect the load acting on the structure. In addition to the above parameters, robustness, connectivity, and redundancy are also critical in the analysis of wind-induced effects on structures. Robustness implies the property of a structure not to respond disproportionately to either abnormal events or initial local failure (Arangio 2012). A general framework, based on the total probability theorem, was proposed in the literature to assess probabilistically the robustness of systems subject to structural damage (Baker et al. 2008, Izzuddin et al. 2012). This framework is consistent with the proposed PBHE framework, e.g., by using as $DV$ the robustness index (Baker et al. 2008). However, the computation of the robustness index for structures subjected to hurricane hazard requires significant research and implementation work, and is outside the scope of this chapter.

Particular attention is needed when a hazard chain is possible or likely. The probabilistic description of $SP$ (e.g., the first- and second-order statistics, as well as the distribution type) needs to be expressed as a function of the damage parameter $DM$. A typical example of this situation occurs when the behavior of buildings subjected to windborne debris hazard is considered (Pinelli et al. 2008). If windows or doors break due to windborne debris impact, the characteristics of the
building envelope (described by \( SP \)) vary, causing a change in the internal pressure coefficients and in the loads acting on the structure (ASCE 2010).

A.6.3 Interaction analysis

The interaction parameters \( IP \) describe the physical interaction between structure and environment, which influences the structural response and performance, as well as the intensity and distributions of the environmental actions as a result of the interaction between the structure and the environment. Typical examples of \( IP \) are the aerodynamic pressure coefficients and the aerodynamic derivatives for dynamic wind actions, the rate of water flow impacting the structure for flooding actions, the kinetic energy and linear momentum of the impacting missile for windborne debris, the wind pressure on the internal and external building surfaces for wind actions on a building envelope, and the rate of water intrusion in a building under strong rain for rainfall action (Dao and Van de Lindt 2012).

In other words, the \( IP \) are parameters that influence the intensity of the environmental actions on the structure, and that depend simultaneously on \( IM \) and \( SP \), as well as on their variability (e.g., the aerodynamic derivatives of a bridge depend both on wind direction and velocity, which are components of \( IM \), and on structural damping, which is a component of \( SP \)). In deterministic terms, this dependency is described by a mechanistic model of the \( IP \) as functions of the \( IM \) and \( SP \) (e.g., see Figure A.3 (a)).

In a probabilistic analysis, the uncertainty of both \( IM \) and \( SP \) must be taken into account in order to obtain the probability distributions of \( IP \), which can be derived by using probability distributions conditional to \( IM \) and \( SP \). The propagation of uncertainties from \( IM \) and \( SP \) to \( IP \) can be performed, e.g., by characterizing the \( IP \) via parametric probabilistic distributions whose parameters are deterministic functions of \( IM \) and \( SP \) (see Ciampoli and Petrini 2012). The conceptual separation
of the interaction analysis from others analysis steps carried out in PBE approaches is an aspect of novelty of the proposed PBHE framework with respect to the original PEER approach. This clear separation between independent parameters (IM and SP) and derived parameters (IP) has also the merit of highlighting the correct direction of uncertainty propagation.

Examples of interaction analysis in structural engineering subfields other than hurricane engineering are: soil-structure interaction analysis in earthquake engineering, fluid-structure interaction analysis in offshore engineering and wind engineering, and heat-transfer analysis in fire engineering.

A.6.4 Structural analysis and damage analysis

The structural analysis phase provides the probabilistic description of a proper EDP, which concisely represents the essential aspects of the structural response for damage and performance evaluation. Examples of EDP are: axial force, shear force, bending moment, and stress state in structural and non-structural elements; response quantities describing the structural motion (deflections, velocities, and accelerations of selected points); structural deformation indices (e.g., interstory or global drift ratio and beam chord rotation).

The damage analysis provides the probabilistic description of DM conditional to the values of EDP. The results of a probabilistic damage analysis are commonly expressed in terms of fragility curves as shown in many recent applications in hurricane engineering (Li 2005, Li and Ellingwood 2006, Shanmugam 2011, Herbin and Barbato 2012). For example, for low-rise wood residential construction, the damage states of interest relate to those components that are essential to maintain the integrity of the building envelope, i.e., roofs, windows, and doors, since the building envelope is the residential construction component that is most vulnerable to hurricane-induced damage (Li and Ellingwood 2006).
In some applications, it is convenient to assume $DM = EDP$, e.g., in the case of low rise gable roof structures, in which the number of lost roof panels due to the uplift pressure generated by hurricane winds can be chosen as both $DM$ and $EDP$. The hurricane wind fragility is then expressed as the cumulative probability distribution conditional to the uplift pressure ($IP$) or to the wind gust velocity ($IM$). In case of hazard chains (e.g., if wind and windborne debris hazards are considered), the representation of the fragility as the probability of $DM$ conditional to $IM$, namely $P(DM|IM)$, can be used to assess the effects of the interaction, e.g., based on the differences between the functions $P(DM|IM)$ obtained by considering undamaged or broken windows (e.g., after a missile impact).

![Diagram](image)

**Figure A.3** Different representations of fragility curves in case of interaction between two hazards: (a) relation between $IP$ and $IM$, (b) fragility curve as a function of $IP$, and (c) fragility curves as functions of $IM$.

Figure A.3 shows two alternative representations of the fragility curve for the roof panel uplift limit state, i.e., $P(DM|IP)$ and $P(DM|IM)$, in the case of low-rise gable-roof buildings under the
The relation between uplift pressure and wind velocity (i.e., $IP = IP(IM)$) is different for the cases “broken” and “unbroken” windows (see, e.g., ASCE standard as shown in Figure A.3 (a)). In fact, this relation must take into account the internal pressurization of the building caused, e.g., by the failure of a door/window due to windborne debris impact (i.e., the chain effect). The fragility curves obtained from the technical literature in the form $P(DM|IP)$ are the same for the two case of “broken” or “unbroken” windows, since they depend only on the properties of the roof panels (see Figure A.3 (b)). Two different $IP$ values (identified in Figure A.3 (a) and (b) as $p_1^{\text{broken}}$ and $p_1^{\text{unbroken}}$) correspond to a given $IM$ value ($V_1$ in Figure A.3 (a) and (c)). Thus, two different fragility curves $P(DM|IM)$ can be built for the two cases of “broken” or “unbroken” windows, which highlight the effects of considering the interaction between wind and windborne debris hazard. In the case of vector $IM$ and $IP$, the fragility can be represented through appropriate fragility surfaces (Seyedi et al. 2010).

### A.6.5 Loss analysis

The loss analysis step gives the estimate of the annual probability of exceedance of $DV$, $G(DV)$, where $DV$ can be used as an indicator for structural risk. Hurricanes are among the most costly natural hazards to impact residential construction in the southeast coastal area of the United States (Li and Ellingwood 2006); thus, $DV$ is usually expressed in monetary terms. It is noteworthy that, from a loss-based design perspective, non-structural and structural damage are both losses; moreover, in addition to direct losses, hurricanes can lead to social disruption for extended periods of time, including the need to relocate building inhabitants (Li et al. 2012).
$DV$ can be chosen as the repair cost related to the hurricane induced damage, or a percentage of the insured value, or the lifetime cost of the structural system, evaluated by taking into account the construction and maintenance costs, the repair costs after an event, the economic losses due to damage (also to building contents), and the loss of functionality (Bjarnadottir et al. 2011). Even in the simplest cases, repair costs are highly uncertain, and updated data from insurance companies are needed to obtain an appropriate probabilistic description of repair costs.

In addition, both ethic and technical problems arise when the $DV$ is related to the loss of human life and/or to a life quality index for the structure subjected to the hurricane. Further research is needed to overcome the technical challenges related to the inclusion of these aspects in evaluating the losses associated with the structural damages and failures due to the hurricanes. In addition, a constructive dialogue is needed among different stakeholders to determine a consensus on when and how to consider life quality indices and costs associated with loss of life into hurricane risk assessment.

### A.7 Application Example

The proposed PBHE framework is illustrated through the risk analysis for a building belonging to a hypothetical residential development, located near the coast in South Florida and composed by 30 identical concrete block gable roof structures (see Figure A.4). This application example seems sufficiently advanced to display some of the specific critical issues of the PBHE framework, and highlights the importance of the interaction between different hazards in a hurricane risk analysis. However, it is also simple enough to avoid the complexities of more realistic applications, thereby maintaining the focus of this chapter on the illustration of the PBHE framework.
The risk analysis is performed for the building identified as “Target” in Figure A.4. The interaction of wind and windborne debris hazards is taken into account. Roof covers are considered as debris sources, whereas the windows and glass doors are considered as debris impact vulnerable components (Lin and Vanmarcke 2008).

A.7.1 Hazard analysis and Structural characterization

In the present study, the 3-second hurricane wind speed $V$ recorded at 10 m above the ground is used as the only component of $W$ for characterizing the wind hazard. For the sake of simplicity, the wind direction variability has been neglected by assuming that the maximum local winds generated by hurricanes act only in the most unfavorable direction for windborne debris hazard (i.e., in the $X$ direction in Figure A.4).

For windborne debris hazard, the considered intensity measures ($IM$) are: the wind speed, $V$; the debris area, $A_d$; and the mass per unit area of debris, $M_d$. It is assumed that the buildings in the benchmark residential development are the only windborne debris source affecting the target structure. Thus, the parameter $n_{buildings}$ (i.e., the density of upwind buildings with respect to the
investigated structure) can be excluded from the $D$ vector. All windborne debris are assumed of sheet type with flight characteristics described by deterministic parameters. The choice of $IM$ is based on damage analysis results available in the literature, which show a strong correlation between the selected parameters and the structural damage produced by wind and windborne debris hazard. A study of sufficiency and efficiency of different potential $IM$ (Luco and Cornell 2007), albeit important, is out of the scope of this study.

Among the wind occurrence models available in the literature (Li and Ellingwood 2006, Yau 2011), the Weibull distribution is adopted here to describe the hurricane wind speed variability (Li 2005). The two-parameter Weibull cumulative distribution function, $F(V)$, is given by:

$$F(V) = 1 - \exp\left[-\left(\frac{V}{a}\right)^b\right]$$  \hspace{1cm} (A.6)

The two shape parameters $a$ and $b$ are site specific and are determined by fitting the hurricane wind speed records provided by the National Institute of Standards and Technology (NIST) to a Weibull distribution. The NIST wind speed records contain data sets of simulated 1-minute hurricane wind speeds at 10 m above the ground in an open terrain near the coastline for locations ranging from milepost 150 (near Port Isabel, TX) to milepost 2850 (near Portland, ME), spaced at 50 nautical mile intervals (92,600 m). Considering South Florida as the location for the case study, the dataset corresponding to milepost 1400 is used for fitting the distribution. The 1-minute hurricane wind speed ($\bar{V}$) dataset is converted into 3-second wind speed as $V = 1.77 \bar{V}$ (Lungu and Rackwitz 2001, ASCE 2010). The two parameter Weibull distribution function is fitted using the converted wind speeds, and the parameters are $a = 25.2447$ m/s and $b = 1.6688$, respectively. The area and mass per unit area of debris are assumed to follow a uniform distribution in the range $[0.108, 0.184]$ m$^2$ and $[10.97, 23.35]$ kg/m$^2$, respectively (Gurley et al. 2005).
The considered structural parameters (SP) are: the wind pressure exposure factor (evaluated at the height \( h \) of the roof of the target building), \( K_h \); the external pressure coefficient, \( G_{Cp} \); and the internal pressure coefficient, \( G_{Cpi} \). The pressure coefficients include the effects of the gust factor \( G \). The topographic factor, \( Kzt \), the wind directionality factor, \( Kd \), are modeled as deterministic and assumed equal to one; whereas the total vulnerable area on each side of each building is assumed equal to 20% of the total wall area. The wind pressure exposure factor \( K_h \) is assumed as normally distributed with a mean value of 0.71 and a coefficient of variation (COV) of 0.19 (Lee and Rosowsky 2005). The characterization of the external and internal pressure coefficients is given in Table A.2 (Li and Ellingwood 2006, Yau 2011).

<table>
<thead>
<tr>
<th>Location/Condition</th>
<th>Mean</th>
<th>COV</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>( G_{Cp} )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roof (near ridge)</td>
<td>-0.855</td>
<td>0.12</td>
<td>Normal</td>
</tr>
<tr>
<td>Roof (away from ridge)</td>
<td>-1.615</td>
<td>0.12</td>
<td>Normal</td>
</tr>
<tr>
<td>Windward Wall</td>
<td>0.95</td>
<td>0.12</td>
<td>Normal</td>
</tr>
<tr>
<td>Leeward wall</td>
<td>-0.76</td>
<td>0.12</td>
<td>Normal</td>
</tr>
<tr>
<td>Side wall</td>
<td>-1.045</td>
<td>0.12</td>
<td>Normal</td>
</tr>
<tr>
<td>( G_{Cpi} )</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Enclosed</td>
<td>0.15</td>
<td>0.33</td>
<td>Normal</td>
</tr>
<tr>
<td>Breached</td>
<td>0.46</td>
<td>0.33</td>
<td>Normal</td>
</tr>
</tbody>
</table>

**A.7.2 Interaction analysis**

The choice of the interaction parameters (\( IP \)) is crucially dependent on the performance levels of interest and the corresponding monitored responses of the structural and non-structural elements. The main parameters controlling the effects of windborne debris impact are the impact linear
momentum, $L_d$, the impact kinetic energy, $E_d$, and the number of impacting debris, $n_d$. It is known from the literature that the impact linear momentum is well correlated with damage for envelope components with a brittle behavior (e.g., doors, windows, see (NAHB 2002, Masters et al. 2010), whereas the impact kinetic energy is better correlated with damage to envelope components with a ductile behavior (e.g., aluminum storm panels, see (Herbin and Barbato 2012)). Hereinafter, it is assumed that the glass windows and doors are unprotected and have a brittle behavior. Based on this assumption, the $IP$ selected in this study are: (1) the linear momentum of the debris at impact, $L_d$, for the windborne debris hazard; and (2) the wind pressure acting on the walls and roof, $p_w$, for the wind hazard.

![Figure A.5 Interaction analysis for windborne debris hazard.](image)

The procedure proposed here for the interaction analysis corresponding to windborne debris hazard is summarized in the flowchart shown in Figure A.5. The input of the interaction analysis for windborne debris hazard is obtained from the hazard analysis and the structural characterization steps. A debris generation model provides the number and type of windborne debris that can affect
the structure under consideration. The debris generation model used in this study is that employed by the Florida Public Hurricane Loss Model (FPHLM), in which the mean percentage of damage to roof covers is based on the simulation results from a component-based pressure induced damage model, and is expressed as a function of the wind speed (Gurley et al. 2005). The number of debris generated from each source house is calculated considering the percentage of roof cover damage at a given wind speed and the geometry of the house (Gurley et al. 2005, Lin and Vanmarcke 2008).

The results of the debris generation model, derived according to the geometry of the considered example case (i.e., density and relative position of debris sources with respect to the target structure), are taken as input for the debris trajectory model (Holmes 2004, Baker 2007, Lin 2007). The debris trajectory model is used to assess if and at which impact velocity a given windborne debris hits the building. In this study, the debris trajectory model provides the landing position of the debris, which is identified by the random variables \( X = \) along-wind flight distance and \( Y = \) across-wind flight distance. The random variables are modeled using a two-dimensional Gaussian distribution described by the following parameters: \( \mu_x = \) mean along-wind flight distance; \( \mu_y = \) mean across-wind flight distance = 0 m; \( \sigma_x = \sigma_y = 0.35 \mu_x = \) standard deviation of the along-wind and across-wind flight distances, respectively (Yau 2011). The parameter \( \mu_x \) is computed as:

\[
\mu_x = \frac{2 M_d}{\rho_a} \left[ \frac{1}{2} C \left( K \cdot T \right)^2 + c_1 \left( K \cdot T \right)^3 + c_2 \left( K \cdot T \right)^4 + c_3 \left( K \cdot T \right)^5 \right]
\]

(A.7)

where: \( \rho_a = 1.225 \text{ kg/m}^3 = \text{air density} \); \( K = \frac{\rho_a \cdot V^2}{2 M_d \cdot g} = \text{Tachikawa number} \); \( \bar{T} = \frac{g \cdot T}{V} = \text{normalized time} \); \( g = \text{gravity constant} \); \( T = \text{flight time in seconds} \); and \( C, c_1, c_2, \text{and} c_3 = \text{non-dimensional coefficients that depend on the shape of the debris} \).
The flight time is assumed to follow a uniform distribution in the interval $[1, 2.5]$ s. For the sheet-type debris considered in this study, $C = 0.91$, $c_1 = -0.148$, $c_2 = 0.024$, and $c_3 = -0.0014$. The debris horizontal velocity at impact, $u_d$, is a function of the wind velocity and the distance travelled by the debris (determined by its landing position), and is given by (Lin and Vanmarcke 2008)

$$u_d = V \cdot \left[1 - \exp \left( -\sqrt{2 \cdot C \cdot K \cdot x} \right) \right]$$

(A.8)

where $x = \frac{g \cdot X}{V^2}$ = dimensionless horizontal flight distance of the debris. The debris is assumed to hit the target building if the debris flight distance is larger than the distance between the source and the target building and, at the same time, the landing position falls within the angle of hit (see Figure A.4).

Finally, the debris impact model uses the horizontal component of the missile velocity (obtained from the debris trajectory model) and data related to the missile size and mass (obtained from the debris generation model) to compute the impact linear momentum of the missile (i.e., the linear momentum corresponding to the windborne debris velocity component orthogonal to the impacted surface, conditional to the event of at least one impact on windows). In this study, the debris impact model gives the impact linear momentum as

$$L_d = M_d \cdot A_d \cdot u_d$$

(A.9)

The interaction analysis for the wind hazard provides the statistical characterization of the wind pressure, $p_w$. For the sake of simplicity, in this study, the wind pressure is computed as

$$p_{w,j} = q_h \cdot \left( G_{C_{p,j}} - G_{C_{p,j}} \right)$$

(A.10)

where the wind velocity pressure at a quote $h$, $q_h$, is given by

$$q_h = 0.613 \cdot K_h \cdot K_{zt} \cdot V^2 \quad \text{(units: N/m}^2)$$

(A.11)
\( I = \text{importance factor (assumed here equal to 1)}. \) In Eq. (A.11), \( V \) is measured in m/s and \( q_h \) is measured in N/m². It is noted here that the simplified approach used in this study to perform the interaction analysis for the wind hazard is appropriate for the simple and small structures considered in the example application. However, larger and more complex structures may require a more rigorous approach based on the use of stochastic processes, random fields, and computational fluid dynamics to evaluate the wind effects on the structure.

A.7.3 **Structural analysis and Damage analysis**

In this example application, the structural analysis step is not needed explicitly because the engineering demand parameters (EDP) can be assumed to coincide with the \( IP \). The strengths of glass windows, glass doors, and roof (which are assumed to be the only components that can be damaged) are obtained from empirical relations available in the literature and directly compared to the corresponding \( IP \). The statistics of the damage limit state capacities for different components and limit states are provided in Table A.3 (Yau 2011, Masters et al. 2010).

<table>
<thead>
<tr>
<th>Component</th>
<th>Limit State</th>
<th>Parameter</th>
<th>Mean (unit)</th>
<th>COV</th>
<th>Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof</td>
<td>Uplift</td>
<td>( R_{\text{roof}} )</td>
<td>2762.7 (N/m²)</td>
<td>0.20</td>
<td>Normal</td>
</tr>
<tr>
<td>Windows</td>
<td>Pressure</td>
<td>( R_M )</td>
<td>4998.7 (N/m²)</td>
<td>0.20</td>
<td>Normal</td>
</tr>
<tr>
<td>Windows</td>
<td>Impact</td>
<td>( R_{\text{window}} )</td>
<td>4.72 (kg·m/s)</td>
<td>0.23</td>
<td>Lognormal</td>
</tr>
</tbody>
</table>

Following a procedure commonly used in PBEE, the physical damage conditions are represented using a limit state function \( g \) for each damage limit state, i.e.,

\[
g = DM - IP
\]  

(A.12)

where the demand measure \( DM \) corresponds to the limit state capacity for the given damage limit state. The damage limit states considered here are (1) the breaking of annealed glass
windows/doors, and (2) the uplift of the roof sheathings.

A.7.4 Loss analysis

In this study, the decision variable \((DV)\) is taken as the repair cost of the building, \(RC\), expressed as a percentage of the total cost of the building. The complementary cumulative distribution of \(DV\) can be used for informed risk-management decision (Mitrani-Reiser et al. 2006) and is computed as the convolution integral of the conditional probability of \(DV\) given \(DM\) and the derivative of the complementary cumulative density function of \(DM\) (Yang et al. 2009). Since the repair costs associated with the different component limit states are not independent, the computation of \(G(DV)\) requires the joint probability density function of the repair costs of all component limit states, which is very difficult to obtain. To overcome this difficulty, a very efficient multilayered Monte Carlo simulation (MCS) approach is used in this study to estimate the loss hazard curve (Conte and Zhang 2007). The multilayered MCS approach is able to account for the uncertainty in the various parameters involved in the risk assessment methodology (i.e., \(IM\), \(IP\), \(DM\), and \(DV\)), which are summarized in Table A.4.

The probabilistic hurricane loss analysis is performed for three different scenarios: (1) considering only the losses due to windborne debris hazard (the debris-only scenario); (2) considering only the losses due to wind hazard (the wind-only scenario); and (3) considering the losses due to windborne debris and wind hazards, and the effects of their interaction (the interaction scenario).

In the debris-only scenario, the repair cost is associated to the failure of a glass door or a window due to the windborne debris impact (i.e., \(L_d \geq R_{w_d}\)). No chain reaction is considered, because the failure of a door or a window does not affect the impact linear momentum of the other missiles. In the wind-only scenario, the repair cost is associated to the failure of a glass door and/or a window, as well as to the uplift of the roof due to the wind pressure (i.e., \(p_w \geq R_{\text{window}}\) and/or \(p_w \geq R_{\text{roof}}\)).
### Table A.4 Summary of parameters used in the risk assessment analysis.

<table>
<thead>
<tr>
<th>Analysis Step</th>
<th>Parameters</th>
<th>Symbol</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>$V$</td>
<td>3-second gust wind speed</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$A_d$</td>
<td>Area of debris</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$M_d$</td>
<td>Mass of debris per unit area</td>
</tr>
<tr>
<td>Hazard Analysis</td>
<td>$IM$</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Structural Characterization</td>
<td>$SP$</td>
<td>$K_h$</td>
<td>Wind pressure exposure factor</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$GC_p$</td>
<td>External pressure coefficient</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$GC_{pi}$</td>
<td>Internal pressure coefficient</td>
</tr>
<tr>
<td>Interaction Analysis</td>
<td>$IP$</td>
<td>$L_d$</td>
<td>Impact linear momentum</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$p_w$</td>
<td>Wind pressure on the surface</td>
</tr>
<tr>
<td>Damage Analysis</td>
<td>$DM$</td>
<td>$R_{window}$</td>
<td>Strength for pressure (window)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R_{roof}$</td>
<td>Strength for uplift (roof)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>$R_M$</td>
<td>Strength for impact (window)</td>
</tr>
<tr>
<td>Loss Analysis</td>
<td>$DV$</td>
<td>$RC$</td>
<td>Repair cost (% of total cost)</td>
</tr>
</tbody>
</table>

In this case, a chain reaction is possible because the failure of a glass door or a window produces an internal pressurization of the building and modifies the wind pressure acting on the other doors and windows and on the roof (through the modification of the $GC_{pi}$ parameter from enclosed to breached building, see Table A.2). The interaction scenario considers the failure of glass doors and windows due to both debris impact and wind pressure, as well as the roof uplift due to the wind pressure. In this case, two types of hazard chains are possible, corresponding to the internal pressurization of the building caused by the failure of a door/window due to the windborne debris impact or to the wind pressure. Thus, the two scenarios considering wind-only and debris-only can be obtained as particular cases of the interaction case by neglecting the wind pressure damage on the doors/windows and roof for the debris-only scenario, and the damage on the doors/windows due to windborne debris impact for the wind-only scenario.
Figure A.6 shows the flowchart of the multilayered MCS approach (Conte and Zhang 2007) used for considering the interaction between wind and windborne debris hazards. The number of hurricanes in each year is simulated according to a Poisson random occurrence model with annual occurrence rate obtained from the NIST database.

Figure A.6 Multilayered MCS approach for probabilistic hurricane loss estimation.

For each generated hurricane, a peak wind speed, $V$, is generated according to the Weibull distribution. For each value of $V$, the value of the wind pressure on the doors/windows and the roof is simulated using the pressure coefficients corresponding to the condition of enclosed buildings. The linear momentum is also computed for each debris impact. If the impact linear momentum and/or the wind pressure assume values larger than the corresponding limit state capacity of the glass on any of the four walls, the building envelope is considered to be breached and the internal pressure is modified. The undamaged building components (doors/windows and roof) are checked for further damage due to the modified pressure. A repair cost is then generated for each damaged component according to an appropriate probability distribution. For the sake of simplicity, it is assumed that the repair cost for the breakage of the windows on any side of the building or the uplift of the roof can be represented by a lognormal random variable with mean equal to 20% of the total cost of the building and COV equal to 20%. The total repair cost for the single hurricane
simulation is equal to the sum of all the simulated component repair costs, with a maximum value of 100% (total failure of the building). It is also assumed that the building is fully repaired after each hurricane event.

The single-year simulation is repeated a large number of times (in the example, 10000 samples are used) to estimate the annual probability of exceedance (which coincides with the complementary cumulative distribution function of $DV$) of the total repair cost.

Figure A.7 Annual probability of exceedance of repair cost for different hazard scenarios.

The annual probabilities of exceedance of the repair cost for the target building for the three different scenarios are shown in Figure A.7, using a semi-logarithmic scale. A strong interaction is observed between the wind and the windborne debris hazards. This observation suggests that the multi-hazard nature of hurricane must be taken into account for accurate probabilistic loss analyses.
A.8 Conclusions

In this chapter, a probabilistic Performance-Based Hurricane Engineering (PBHE) framework is proposed and illustrated. The methodology, that can be used to evaluate the structural risk associated with facilities located in hurricane-prone regions, is based on the total probability theorem and builds on techniques already developed and used in other civil engineering subfields. The problem of risk assessment is disaggregated into the following basic probabilistic components: (1) hazard analysis, (2) structural characterization, (3) interaction analysis, (4) structural analysis, (5) damage analysis, and (6) loss analysis. Each of the analysis steps is briefly discussed in this chapter. Particular emphasis is given to the differences between PBHE and other existing performance-based engineering frameworks, e.g., the multi-hazard nature of hurricane events, the presence of interacting hazard, and the focus on high, intermediate, and low performance levels.

The feasibility of the proposed framework is demonstrated through an application example consisting in the risk assessment for a target building in a hypothetical residential development under three different hazard scenarios. It is observed that the interaction between wind and windborne debris hazard can affect significantly the value of the annual probability of exceedance of repair cost. This observation suggests the need to consider the multi-hazard nature of hurricane events for accurate probabilistic loss analysis.

A.9 References

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APPENDIX B

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Vipin Unnikrishnan <vunnik1@tigers.lsu.edu>  Thu, Apr 16, 2015 at 1:06 PM
To: Francesco Petrini <francesco.petrini@uniroma1.it>

Dear Dr. Petrini,

I'm writing this mail with reference to our paper titled “Performance-based hurricane engineering (PBHE) framework” published in Structural Safety as a result of our collaborative research. I'm planning to use that paper as one of my chapters in my dissertation explicitly stating and highlighting my key contribution to that research. So I kindly request you to grant me permission to use the paper and also validate my key contributions to that research.

Thanking you

Vipin Unnithan Unnikrishnan

--
Graduate Student
Department of Civil & Environmental Engineering
LSU, Baton Rouge
(225)-445-9002
April 17, 2015

To whom it may concern:

Mr. Vipin Unnithan Unnikrishnan, has been my co-author for the following journal paper:


I give him the permission of using all the content of the above paper in his dissertation.

I would also validate the contribution of Mr. Unnikrishnan to the collaborative research made for the development of the above paper in the following contributions:

1) identification of the key parameters for the probabilistic characterization of windborne debris hazard;
2) development of the multi-layer Monte Carlo simulation (MCS) method for PBHE;
3) implementation of the PBHE framework and multi-layer MCS method for the application example presented in the paper;
4) investigation of the interaction between hazard sources for the specific application example.

Sincerely yours,

[Signature]

Francesco Petrini, Structural Engineer, Ph.D., P.E.

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Request for the use of manuscript

Vipin Unnikrishnan <vunnik1@tigers.lsu.edu>  Thu, Apr 16, 2015 at 12:05 PM
To: gardoni@illinois.edu
Cc: Michele Barbato <mbarbato@lsu.edu>

Dear Dr. Gardoni,

I'm Vipin Unnikrishnan, Phd student of Dr. Michele Barbato (Louisiana State University, Baton Rouge). I'm writing this mail with reference to the manuscript titled "Performance-Based Hurricane Engineering: A Multi-Hazard Approach" which we have submitted for the ICMAE 2014 edited volume.

I'm preparing for my dissertation defense and I plan to use this paper as one of my chapters in my dissertation. Therefore, I kindly request you to grant me permission to use the same.

Also, can you please update me on the status of the manuscript and the intended title of the edited volume.

I look forward to hearing from you

Thanks & regards

Vipin Unnikrishnan

---
Graduate Student
Department of Civil & Environmental Engineering
LSU, Baton Rouge
(225)-445-9002
Hi Vipin,

I think it would be fine if you use the manuscript as part of your dissertation. Typically there is the option of waiting to make available a thesis/dissertation online or through the library system some a year or so to allow time for the material submitted (or to be submitted) to be published. I imagine LSU has the same option, which I would suggest using.

The tentative title of the edited volume is: “Multi-hazard Approaches to Civil Infrastructure Engineering.” We had to replace a couple of authors so the deadline was pushed a little bit. We should be able to send comments on all the chapters by the beginning of the summer.

I hope my reply is of some help. Best regards,

-- PG

Paolo Gardoni
Associate Professor
Department of Civil and Environmental Engineering (CEE at Illinois: Tackling Society's Most Complex Challenges)
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Vipin Unnithan Unnikrishnan received his Bachelor of Technology degree in civil engineering from the University of Kerala, India in 2005. He completed his Master of Science degree in 2003, from the Indian Institute of Technology Madras, India in civil engineering with a specialization in structural engineering and he worked on the development of seismic fragility curves using response surface methods.

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