Multi-Hazard Vulnerability Assessment of Monopile Foundation Offshore Wind Turbines

Jharna Pokhrel
South Dakota State University

Follow this and additional works at: https://openprairie.sdstate.edu/etd

Part of the Civil and Environmental Engineering Commons

Recommended Citation
https://openprairie.sdstate.edu/etd/2454

This Thesis - Open Access is brought to you for free and open access by Open PRAIRIE: Open Public Research Access Institutional Repository and Information Exchange. It has been accepted for inclusion in Electronic Theses and Dissertations by an authorized administrator of Open PRAIRIE: Open Public Research Access Institutional Repository and Information Exchange. For more information, please contact michael.biondo@sdstate.edu.
MULTI-HAZARD VULNERABILITY ASSESSMENT OF MONOPILE FOUNDATION OFFSHORE WIND TURBINES

BY

JHARNA POKHREL

A thesis submitted in partial fulfillment of the requirements for the

Master of Science

Major in Civil Engineering

South Dakota State University

2018
MULTI-HAZARD VULNERABILITY ASSESSMENT OF MONOPILE FOUNDATION OFFSHORE WIND TURBINES

This thesis is approved as a creditable and independent investigation by a candidate for the Master of Science in Civil Engineering degree and is acceptable for meeting the thesis requirements for this degree. Acceptance of this thesis does not imply that the conclusions reached by the candidate are necessarily the conclusions of the major department.

Dr. Junwon Seo,
Thesis Advisor

Dr. Nadim Wehbe,
Department Head
Civil and Environmental Engineering

Dean, Graduate School
ACKNOWLEDGMENTS

I would like to express my sincere gratitude to my advisor, Dr. Junwon Seo, for his wisdom, guidance, and encouragement throughout my studies. I appreciate the opportunity to work and learn from him in the last few years at South Dakota State University. His supervision provided me the motivation to explore the concept relating to this field of study and helped me greatly improve my writing skills. I also want to thank Dr. Nadim Wehbe and Dr. Katherine Malone for serving on my committee.

I want to extend my appreciation to Dr. Nadim Wehbe, Dr. Mostafa Tazarv, and Dr. Suzette Burckhard for providing assistance and helping me overcome challenges during my Master’s program.

I would like to express love for my husband, Ujjwol Tamrakar for always being supportive, caring and providing advice for this thesis. I would like to thank my parents Kamal Kumar Pokharel and Sanu Pokharel, my sisters Barsha Pokharel and Elisha Tamrakar, and my parents-in-law Dr. Indraman Tamrakar and Pabitra Tamrakar for their love, support, motivation, and encouragement during my studies. I wish to provide special thanks to Abhilasha Bajracharya, Jyotshna Pokharel, Manisha Maharjan, Prajina Tandukar, and Rina Koirala for providing valuable suggestions, encouragement, and motivation even in my lowest to help me achieve my goals.

I also would like to thank Benjamin Litchy for helping me out in my simulation. I thank Dr. Jason Jonkman of National Renewable Energy Laboratory for providing advice on wind simulation.
# CONTENTS

LIST OF FIGURES ................................................................................................. viii
LIST OF TABLES .................................................................................................... xiii
ABSTRACT .............................................................................................................. xiv

INTRODUCTION ...................................................................................................... 1

RESEARCH OBJECTIVES ..................................................................................... 2

SCOPE OF RESEARCH ......................................................................................... 2

THESIS OUTLINE ................................................................................................... 3

1 CHAPTER 1 NATURAL HAZARD VULNERABILITY

QUANTIFICATION OF OFFSHORE WIND TURBINE IN SHALLOW WATER

1.1 Abstract .............................................................................................................. 5

1.2 Introduction ...................................................................................................... 6

1.3 Literature Review ............................................................................................ 8

1.3.1 Computational Modeling and Simulation ............................................... 8

1.3.2 Reliability Analysis of Offshore Wind Turbines ................................ 10

1.3.3 Summary .................................................................................................... 11

1.4 Studied Offshore Wind Turbine ..................................................................... 12

1.4.1 Offshore Wind Turbine Description ...................................................... 12

1.4.2 Reliability Analysis .................................................................................. 15

1.5 Modeling Approach for Wave Vulnerability Analysis .................................. 17

1.5.1 Morrison’s Equation ............................................................................... 17

1.5.2 Finite Element Analysis ........................................................................ 19
3.8.2 Effect of Structural Parameters under Wind and Wave Loading 149

3.9 Conclusions and Future Work................................................................. 154
LIST OF FIGURES

Figure 1.1: Schematic of the 5 MW OWT model under distributed wind and wave loads. ................................................................. 14

Figure 1.2: Sample profile for total drag, total inertial, and total wave forces acting at MSL at a wave height of 27 m. .............................................................. 19

Figure 1.3: Representative schematic for the FEA simulation in SAP2000 with wave loads applied to the structure. ................................................................. 21

Figure 1.4: Fragility curve of OBM at mudline with varying wave heights........ 22

Figure 1.5: Peak OBMs of the OWT model at the mudline observed over wave height obtained using three approaches. ................................................................. 23

Figure 1.6: Fragility Curve of OBM at mudline with varying wave heights........ 25

Figure 1.7: Blade root moment of an OWT for a wave height of 5 m and a) a turbulent wind speed of 25 m/s and b) varying wind speed. ........................................ 29

Figure 1.8: Variation of blade-tip displacement observed with change in mean hub-height wind speed and significant wave height. ........................................ 30

Figure 1.9: Variation in OBM observed in three directions over simulation time. ......................................................................................... 31

Figure 1.10: Observed OBM in fore-aft direction with variation in mean hub-height wind speed and significant wave height. ........................................ 32

Figure 1.11: 3-Dimensional fragility surface observed for OBM at mudline with respect to variation in wind speed and wave height. ........................................ 33
Figure 1.12: Comparison of fragility curves of OWT for mudline OBM with respect to wave height: a) regular and irregular wave and b) different wind speed for an irregular wave. ................................................................. 35

Figure 2.1: Schematic representation of Offshore Wind Turbine with structural properties and environmental loadings parameters................................................................. 57

Figure 2.2: Comparison between tower top deflections observed using SMLR model and FAST simulation with respect to a) hub height; b) monopole thickness; c) wind speed; and d) wave height. ................................................................. 74

Figure 2.3: Comparison between mudline flexure observed using SMLR model and FAST simulation with respect to a) hub height; b) monopole depth; c) wind speed; and d) wave height. ................................................................. 75

Figure 2.4: Comparison between mudline shear observed using SMLR model and FAST simulation with respect to a) hub height; b) monopole thickness; c) wind speed; and d) wave height. ................................................................. 76

Figure 2.5: Comparison between blade tip deflections observed using SMLR model and FAST simulation with respect to a) hub height; b) monopole thickness; c) wind speed; and d) wave height. ................................................................. 77

Figure 2.6: Probabilistic 3-D fore-aft tower top deflection fragility surface observed for the Offshore Wind Turbine subjected to change in wind speed and wave height parameter................................................................. 80

Figure 2.7: Probabilistic 3-D fore-aft bending moment fragility surface observed for the offshore wind turbine subjected to change in wind speed and wave height parameter................................................................. 81
Figure 2.8: Probabilistic 3-D fore-aft shear force fragility surface observed for the offshore wind turbine subjected to change in wind speed and wave height parameter. 82

Figure 2.9: Probabilistic 3-D blade tip deflection fragility surface observed for the offshore wind turbine subjected to change in wind speed and wave height parameter. 83

Figure 2.11: Effect of significant input parameters on monopile OWT fragility at median wind speed value for flexural and deflection failure. 84

Figure 2.12: Effect of tower base diameter on monopile OWT fragility with the increase in wind speed: a) flexure and b) deflection. 85

Figure 2.13: Effect of significant input parameters on monopile OWT fragility at median wave height value for flexural and deflection failure. 87

Figure 2.14: Effect of input parameter on monopile OWT fragility with the increase in wave height: a) hub height for flexure and b) tower base diameter for deflection. 88

Figure 3.1: Schematic of Offshore Wind turbine with structural properties and environmental parameters. 112

Figure 3.2: Representative Pareto plot of the deflection response for the screening analysis. 120

Figure 3.3: Comparison of tower top deflection observed using the SMLR, FAST, and RSM approach with respect to a) hub height; b) monopile thickness; c) wind speed; and d) wave height. 131

Figure 3.4: Comparison of mudline flexure observed using the SMLR, FAST, and RSM approach with respect to a) hub height; b) monopile thickness; c) wind speed; and d) wave height. 132
Figure 3.5: Comparison of mudline shear force observed using the SMLR, FAST, and RSM approach with respect to a) hub height; b) monopile thickness; c) wind speed; and d) wave height .......................................................... 132

Figure 3.6: Comparison of blade tip deflection observed using the SMLR, FAST, and RSM approach with respect to a) hub height; b) monopile thickness; c) wind speed; and d) wave height .......................................................... 133

Figure 3.7: Fragility curve for a monopile 5 MW OWT tower top deflection: a) with respect to wind speed and b) with respect to wave height ........................................ 135

Figure 3.8: Fragility curve for a monopile 5 MW OWT mudline bending moment: a) with respect to wind speed and b) with respect to wave height. ............... 136

Figure 3.9: Fragility curve for a monopile 5 MW OWT mudline shear force: a) with respect to wind speed and b) with respect to wave height................................. 137

Figure 3.10: Fragility curve for a monopile 5 MW OWT blade tip deflection: a) with respect to wind speed and b) with respect to wave height................................. 138

Figure 3.11: 3-Dimensional fore-aft tower top deflection fragility surface observed for the OWT subjected to variation in input parameters: a) wind speed and wave height; b) monopile thickness and hub height; c) hub height and tower base thickness, and d) water depth and rotor diameter ................................................................. 142

Figure 3.12: 3-Dimensional fore-aft mudline flexural fragility surface observed for the OWT with variation in input parameters: a) wind speed and wave height; b) monopile thickness and hub height; c) hub height and tower base thickness, and d) rotor diameter and water depth ................................................................. 144

Figure 3.13: 3-Dimensional fore-aft mudline shear fragility surface observed for the OWT with variation in input parameters: a) wind speed and wave height; b) monopile
thickness and hub height; c) hub height and tower base thickness, and d) rotor diameter and water depth.

Figure 3.14: 3-Dimensional blade tip deflection fragility surface observed for the OWT with variation in input parameters: a) wind speed and wave height; b) rotor diameter and hub height.

Figure 3.15: Effect of significant input parameters on 25% monopile OWT fragility behavior for deflection, flexure, and shear failure a) with respect to wind speed and b) with respect to wave height.
LIST OF TABLES

Table 1.1: Properties of NREL 5 MW Baseline OWT model used in this study . 13
Table 2.1: Range of parameters for experimental design ................................. 62
Table 2.2. Sample Latin Hypercube Sampling combination .............................. 66
Table 2.3. Comparison of statistical performance for the deformation demand models .................................................................................................................. 70
Table 2.4. Explanatory function using only influencing parameters obtained from MLR analysis ........................................................................................................ 72
Table 3.1. Monopile OWT input parameters ...................................................... 116
Table 3.2. Significant monopile OWT input parameter for the observed response .................................................................................................................. 120
Table 3.3. Central Composite Design table for observed and predicted response of OWT .................................................................................................................. 122
Table 3.4. Summary of the statistical measure of error observed from SMLR and RSM models........................................................................................................ 129
Table 3.5. Summary of the response observed from SMLR and RSM modeling approach ............................................................................................................. 129
Table 3.6. Summary of the response observed for structural parameters at 25% exceedance probability using RSM model ....................................................... 150
Table 3.7. Summary of the response observed for structural parameters at 25% exceedance probability using RSM model ....................................................... 151
ABSTRACT

MULTI-HAZARD VULNERABILITY ASSESSMENT OF MONOPILE FOUNDATION OFFSHORE WIND TURBINES

JHARNA POKHREL

2018

Offshore Wind Turbines (OWTs) have increased in popularity because of numerous technological advancements in the sector of renewable energy. With such an increase in the number of installments, there is a need for multi-hazard vulnerability assessment of the OWT at a given site. Because of hurricane and tsunami loads pertaining to the offshore site, structural failure analysis with respect to wind and wave loads at the critical section of the OWT is required. The existing simulation methods for the failure assessment are computationally expensive and require many simulations to estimate the multi-hazard behavior. The goal of this thesis was to study multi-hazard behavior of the monopile OWTs. The multi-hazard vulnerability analysis was performed for a 5 MW OWT simulation model developed by National Renewable Energy Laboratory (NREL). Specifically, FAST v8, a simulator developed by NREL was used to perform the simulations for the coupled dynamic response of the structure. In pursuit of the goal, this thesis also aims at developing two surrogate models to estimate the response of OWTs for risk assessment at the low computational cost. Surrogate models replace the traditionally used tedious nonlinear aero-hydrodynamic simulations without loss of accuracy and less computational effort. The surrogate models created were Stepwise Multiple Linear Regression (SMLR) and second order polynomial Response Surface Metamodels (RSMs). Results from each of the models were compared with the observed
FAST responses. It was concluded that the RSM model capable of representing nonlinear behavior of the response offered more accurate results with less computational effort when compared to SMLR. Then, vulnerability analysis performed for multi-hazard loadings revealed flexural failure was the most critical failure at multi-hazard loading scenarios among others, including deflection and shear. More rigorous analysis accounting for the variation in both structural and multi-hazard loading parameters was performed. The result emphasized on the importance of considering uncertainties in structural and loading parameters to improve structural reliability of monopile OWTs.
INTRODUCTION

Increasing demand for renewable energy has favored widespread installation of Offshore Wind Turbines (OWTs). With a major share of the population residing in coastal regions, which have relatively higher wind speeds, the scope of OWTs is promising in the United States (U.S.). The estimated total potential of offshore wind power in the U.S. is 1070 GW, at water depths of less than 30 meters. Most of the existing OWTs are currently installed at these water depths. The stiffness and strength of the tower in these shallow water regions are commonly supported by a monopile foundation. Despite their advantages, the OWTs are subjected to different hazards, such as hurricane and tsunamis, causing significant damage which may lead to the collapse of the structure.

To date, limited studies exist on failure estimation of the monopile OWTs with respect to multi-wind-and-wave loading conditions. The recent landfall of Hurricane Harvey (Category 4) and Hurricane Irma (Category 5) along the east coast of the U.S., caused vast damage to the existing OWTs. Because of the higher potential of wind energy in the hazard-prone region, there always exists risks associating to the structural performance of those turbines. This results in a higher construction and maintenance cost. Recent studies have shown that approximately 10 failures per turbine occur each year in wind farms, which has led to the need for frequent conditional monitoring. One of the evident solutions to reduce frequent monitoring is to perform reliability assessment of the structure to identify the failure probability depending on the site conditions. However, the available computation tools that perform such simulations are time-consuming and
require a large number of simulations to predict the structural behavior of the OWTs in multi wind-and-wave loading scenarios. The usage of surrogate models provides a more efficient prediction to determine the vulnerability at the critical sections of the monopile OWTs.

RESEARCH OBJECTIVES

The objective of this study is to perform a structural vulnerability assessment of monopile offshore wind turbine in multi-hazard wind and wave loading condition. Such assessment performed at critical sections of the structure helps in determining risks associated in the hazard-prone region. Surrogate modeling approach, involving the interaction of input parameters, could be beneficial for such assessment by minimizing the use of computationally expensive nonlinear simulations. Therefore, surrogate models in the identification of vulnerability could be beneficial in further maintenance and installation of the future OWTs.

SCOPE OF RESEARCH

The scope of work can be listed as follows:

- Perform structural fragility analysis of a 5 MW monopile offshore wind turbine under wave loadings and multi wind-and-wave loadings to determine the flexural behavior of each loading scenario at the critical sections of the tower.
• Develop surrogate models utilizing all available specifications for 5 MW wind turbine to predict the peak response at critical sections of the OWTs.

• Compare the surrogate models developed using the same range of input parameters but employing different modeling approaches using statistical analysis techniques.

• Develop 3-Dimensional fragility curve to determine the multi-hazard behavior at the critical sections of the OWTs.

• Perform multi-hazard vulnerability analysis to determine the critical parameter for each observed response.

THESIS OUTLINE

The thesis is organized into three different chapters, resembling three research chapters, on vulnerability assessment of monopile foundation OWTs in multi-hazard loading scenarios. Chapter One details three computational approaches to determine wave fragility analysis of a 5 MW OWT model provided by National Renewable Energy Laboratory (NREL). Further, using a more rational computational approach having the capability to perform nonlinear aero-hydrodynamic analysis, a multi-hazard fragility surface was developed for combined wind-and-wave loading condition. Chapter Two presents the stepwise multiple linear regression (SMLR) approach-based demand model to predict the peak response at critical tower top deflection, mudline flexure, and mudline shear response of the OWTs. Those demand models were used to estimate the vulnerability at those critical sections in multi wind-and-wave loading scenarios. Additionally, critical input parameter affecting the observed response was determined in
terms of wind speed and wave height to represent critical parameter affecting wind prone and wave prone offshore site. Chapter Three details the response surface metamodel (RSM) approach to develop demand models, at the critical sections discussed in Chapter Two. The RSM and SMLR models were compared to determine better model based on the statistical performance, computational time, and accuracy. Further vulnerability analysis of the OWT was performed with respect to significant input parameters to identify the effect of those parameters on the structural fragility analysis.
1 CHAPTER 1 NATURAL HAZARD VULNERABILITY QUANTIFICATION OF 
OFFSHORE WIND TURBINE IN SHALLOW WATER

1.1 Abstract

Offshore Wind Turbines (OWTs) are prone to different types of natural hazards related to wind and wave loads, causing different levels of structural damage. This chapter aims at simulating various wind and wave loads acting on the OWT and performing its vulnerability analysis in the form of fragility curves. The OWT used for the analysis is National Renewable Energy Laboratory (NREL) baseline 5-MW OWT installed in 20m water depth. Initially, the analysis accounting for variability in only wave characteristics was done due to high computational cost by performing the three different approaches: 1) Morison’s Equation; 2) Finite Element Analysis (FEA) using SAP2000; and 3) Computer Aided Engineering (CAE) tools using Fatigue, Aerodynamics, Structures, and Turbulence (FAST) code. The results from each approach coupled with First Order Reliability Method (FORM) were used to develop wave fragility curves, indicating that the FAST approach involving an interaction of aero-hydro-sub-dynamics within the OWT model, resulting in a reasonable conservative range in the fragility curve. With the results, the FAST approach was used to assess multiple wind-and-wave hazard vulnerability of the OWT. To that end, an extreme turbulent model (ETM) coupled with irregular waves determined based on Pierson-Moskowitz (JONSWAP) spectrum was utilized. The OWT was simulated, considering the extreme loading scenarios specified by the International Electro-technical Commission (IEC) Design Standard that takes variability of both wind and waves into consideration. Structural responses of the OWT were captured at various critical locations across the
OWT. While evaluating the OWT’s failure mechanism, the resulting flexural demands at the mudline were found to be critical, and the FORM with these demands was applied to create the multi-hazard fragility curves. The multi-hazard fragility analysis revealed that the exceeding probability increased when there is an increase in both wind speed and wave height, especially above 12 m/s, while the wave height has less impact on the probability until the wave height of 10 m is reached. Through the comparison of regular and irregular wave loading fragility data, the significant difference in the exceeding probability was also found due to the gap in regular and irregular wave characteristics.

1.2 Introduction

Offshore wind farms have potential to become a major source of energy in the near future. The offshore wind farm has several advantages relative to the onshore wind farms (Bilgili et al. 2011; Esteban et al. 2011) because it has the potential to install higher capacities of Offshore Wind Turbines (OWTs) in high wind coastal regions. Some other advantages include larger available areas for the offshore wind farms, higher wind speeds with an increase in distance from the shore, lower disturbance due to the noises produced while in operation, and less harm to the environment (IEA 2008; Markard and Petersen 2009) when compared to the onshore wind farms.

A recent study shows that with the advancement of the production of wind energy in the United States (US), the OWTs are able to cover a broad range of the coastal areas, where a majority of the population resides (Musial 2007). It is estimated that the US alone has the potential to produce nearly 4,150 GW of offshore wind power and 1,070 GW can be produced from existing OWTs in shallow water regions (Musial and Butterfield 2004). Because of huge amounts of energy in shallow water, most of the
existing OWTs reside in a water depth of less than 30 m, mainly due to lower maintenance and installation cost and less expertise in deep water (Lozano-Minguez et al. 2011; Lombardi et al. 2013).

Typically, the OWTs in shallow water is supported by a monopile foundation consisting of a circular hollow steel tube embedded into the seabed, which extends above the sea level. The monopile OWTs have been subjected to stochastic wind and wave loads. In particular, wave forces on the OWTs are one of the dominant loadings in the offshore environment (U.S. Army Corps of Engineers 2006; Wei et al. 2014; Seo et al. 2017) causing significant damage to the components (Jha et al. 2008; Yu et al. 2011; Seo et al. 2015). The majority of the OWT studies (Bisoi and Haldar 2015; Schløer et al. 2016; Zhang et al., 2018) have used linear wave theory as suggested in work by Arany et al. (2015a).

The OWT installation in shallow water demands a significant amount of research to be done beforehand regarding the site conditions, particularly associated to uncertainties in both wind and wave loadings (Henderson et al., 2003). Despite the advantages of OWTs, there exist risks associated with various extreme conditions, such as a hurricane and tsunamis, causing critical damage to the collapse of the monopile OWTs (Musial et al. 2006; Sørensen 2009). Recent Hurricane Harvey (Category 4 hurricane) and Hurricane Irma (Category 5 hurricane) along the North Atlantic zone made a landfall on the east coast of the US and caused vast damage to the existing OWTs. This problem necessitates an adequate vulnerability assessment to be performed to quantify risk possessed by the OWTs in such a region experiencing wind and wave hazards (Toner and Mathies 2002; Musial and Ram 2010).
The ultimate goal of this chapter is to simulate various wind and wave loads acting on an OWT in shallow water and quantify its vulnerability in the form of fragility curves. This chapter is divided into six sections with this section. The second section presents the literature review related to the computational modeling approaches and reliability assessment for OWTs’ fragility analysis. The third section provides an overview of the OWT considered for the analysis. The fourth section focuses on the wave fragility analysis using three computational approaches: 1) Morison’s equation, 2) Finite Element Analysis using SAP2000, and 3) Computer Aided Engineering (CAE) tool using Fatigue, Aerodynamics, Structures, and Turbulence (FAST) (Jonkman and Buhl 2005). The fifth section deals with the multiple wind-and-wave fragility analysis using the FAST approach. The sixth section includes conclusions and future work.

1.3 Literature Review

Researchers (Valamanesh et al. 2014, Hallowell et al. 2016) over the years have applied different approaches to simulate the wind and wave loadings pertaining to the offshore environments and perform reliability analysis to quantify the structural behavior of monopile OWTs under the extreme loading conditions including the effects of stochastic wind and wave loadings. The following section details the previous studies performed for computational modeling and simulation approaches and reliability assessment of the OWTs.

1.3.1 Computational Modeling and Simulation

Various computational approaches (Haritos 1995; Chandrasekaram et al. 2004; Raheem et al. 2012; Chitziioannou et al. 2015, Seo et al. 2015) have shown popularity in recent offshore research to investigate the behavior of OWTs under ocean environmental
conditions. Some studies on the simulation of wave loads acting on the OWT are based on the empirical Morison’s equation developed by Morison et al. (1950). This equation follows that the wave-induced force on the structure includes drag and inertia coefficients reflecting wave characteristics, and this can be applied to determine the hydrodynamic force with respect to the mudline (Borgmann 1958; Coakrabarti et al. 1975). The equation also involves nonlinear relative velocity components, which could be time-consuming, therefore researchers over years have modified the equation into a more simplified linearized model, considering site-conditions. The simplified model has proven to be efficient in computational time (Haritos 1995; Chandrasekaram et al. 2004).

Other studies have been performed on generating wave loads acting on the OWTs to assess their behaviors using FEA employing SAP2000 (SAP2000 Manual), which has an ability to perform dynamic wave loading phenomena on the substructure (Raheem et al. 2012; Chitzioannou et al. 2015, Seo et al. 2015). These studies have successfully determined the response of the OWT, while employing hydrodynamic loads, such as wave loads on the foundation. Recently, Seo et al. (2015) studied the wave-induced behavior of the monopile OWT using SAP2000, revealing that the wave load had a high impact on the structural response, such as deflection.

Some studies (Barj et al. 2014; Mardfekri and Gardoni 2013, Carswell et al. 2014; Krathe and Kaynia 2015) have employed the FAST to simulate multiple wind-and-wave dynamic loadings on the OWT. It was found that the FAST was proven as an effective tool in modeling and analyzing the OWT subjected to stochastic wind and wave loads at a time domain. Other studies (Asareh et al. 2016; Valamanesh et al. 2015; Taflanidis et al. 2013; Myers et al. 2013; Jiang et al. 2017) have been also done in the area of risk and
vulnerability assessment for the 5MW OWT model via the FAST. For example, Valamanesh et al. (2015) performed a multivariate analysis of the OWT under various wind and wave loads considering peak spectral period, so as to assess extreme storm conditions. The results emphasized the importance of determination of a suitable range of peak spectral periods.

1.3.2 Reliability Analysis of Offshore Wind Turbines

Reliability analysis of the OWTs has been widely incorporated in the design of the structure to identify potential risk observed due to the wind and wave-related hazards. A number of studies (U.S. Army Corps of Engineers 2006; Sorensen et al. 2008; Golafshani et al. 2011; Kim and Kim 2014) have been performed to study the wave loadings affecting OWTs’ structural behavior. It was found that the wave loads are non-proportional with the increase in wave heights, requiring more in-depth research to identify their effect over the range of different wave heights.

Several studies (Quilligan et al. 2012; Rendon and Manuel 2014, Kim et al. 2014; Mardfekri and Gardoni 2015; Wei et al. 2016) have demonstrated that the fragility analysis is considered useful in the assessment of risk associated with the wind and wave loadings. For example, Quilligan et al. (2012) applied a probabilistic approach, using the theorem of total probability, to develop displacement based fragility curves of 5MW wind turbine towers subjected to variability in wind speed and turbulence. The parameters of towers were defined as probabilistic variables. It was concluded that varying turbulence caused a noticeable effect on structural failure probability. Mardfekri and Gardoni (2015) also performed the fragility analysis in terms of deflection and moment demands of a 5MW OWT subjected to seismic and wind loads. A probabilistic
demand model was developed using the response observed from FAST simulation along with detailed analysis of 3-dimensional (3-D) finite element model, and accordingly, the fragility curve was generated using Monte Carlo simulation technique for the operational wind speed. It was reported that the annual failure probability is higher than the target safety level recommended by the OWT design guidelines and further suggested for such analysis in design purposes. Recently, Wei et al. (2016) estimated the vulnerability of the OWT jacket foundation under extreme wind and wave loadings using the pushover analysis in developing relevant fragility curves corresponding to their mean return wave period. The result highlighted that the failure probability is highly sensitive to the wave height and demonstrated the importance of wave height in design.

Other studies (Rendon and Manuel 2014, Agarwal and Manuel 2008, Coe et al 2018, Moriarty and Hansen 2005) have been done to use statistical techniques necessary for the OWTs’ behaviors. Rendon and Manuel (2014) used statistical load interpolation technique to predict long-term behaviors of the 5MW OWT monopile foundation with different wind and wave loadings scenarios by performing aero-hydrodynamic simulation in FAST. The study demonstrated the importance of multi-wind-and-wave load variability in predicting the long-term structural behavior for design while examining the fore-aft tower overturning bending moments (OBM) at mudline and out-of-plane moment (OPM) at the blade root.

1.3.3 Summary

Based on the literature review, there exist a number of studies on the overall behavior assessment of the OWT using the predefined computational approaches. However, there has been no side-by-side comparison of the observed response among
these approaches in terms of fragility curves to identify a reasonable approach for the risk analysis of OWTs when subjected to wave loadings. The literature also lacks the fragility behavior of the OWT with respect to multiple wind-and-wave loadings at the critical section.

1.4 Studied Offshore Wind Turbine

This section provides the description of OWT model utilized for the simulation. An overview of the geometry and materials is provided along with an in-depth discussion of the associated reliability analysis technique utilized for the fragility curve development.

1.4.1 Offshore Wind Turbine Description

The monopile OWT referred to an “NREL offshore 5 MW baseline wind turbine” model developed by National Renewable Energy Laboratory (NREL) (Jonkman et al. 2009) was selected for the study because the number of studies (Passon et al. 2007, Jonkman et al. 2008, Shirzadeh et al. 2013) have been done for its feasibility verification and public availability of its dimensions and structural characteristics. A turbine is a variable-speed machine having rotor speed of 12.1 rpm and rated wind speed of 11.4 m/s. The OWT tower was made with a structural steel having a cylindrical cross-section that was linearly tapered with varying diameter and thickness. The density of 8,500 kg/m³, which is higher than typical steel’s (density =7850 kg/m³) accounts for paint, bolt, welds, and flanges (Jonkman et al. 2009). The monopile foundation was a cylindrical steel cross-section having same properties as in the tower. The monopile had a constant diameter and thickness. The specifications of OWT model including the tower dimension is shown in Table 1.1. All the additional detailed structural, sectional, and dynamic properties of the
blade and tower regarding the design of the NREL offshore 5 MW baseline wind turbine can be found in Jonkman et al. (2009). A geometrical representation of the OWT with distributed wind and wave loadings on the structure is shown in Figure 1.1.

Table 1.1: Properties of NREL 5 MW Baseline OWT model used in this study

<table>
<thead>
<tr>
<th>Properties</th>
<th>NREL Baseline OWT Model</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rating</td>
<td>5 MW</td>
</tr>
<tr>
<td>Rotor Orientation</td>
<td>Upwind</td>
</tr>
<tr>
<td>Number of Blades</td>
<td>3</td>
</tr>
<tr>
<td>Rotor Diameter</td>
<td>126 m</td>
</tr>
<tr>
<td>Hub Diameter</td>
<td>3 m</td>
</tr>
<tr>
<td>Hub Height</td>
<td>90 m</td>
</tr>
<tr>
<td>Tower base diameter, thickness</td>
<td>6 m, 0.027 m</td>
</tr>
<tr>
<td>Tower top diameter, thickness</td>
<td>3.87 m, 0.019 m</td>
</tr>
<tr>
<td>Rotor Mass</td>
<td>110,000 kg</td>
</tr>
<tr>
<td>Nacelle Mass</td>
<td>240,000 kg</td>
</tr>
<tr>
<td>Tower Density</td>
<td>8500 kg/m³</td>
</tr>
<tr>
<td>Young's Modulus of Elasticity</td>
<td>210 GPa</td>
</tr>
<tr>
<td>Shear Modulus</td>
<td>80.8 GPa</td>
</tr>
<tr>
<td>Depth of Monopile</td>
<td>20 m</td>
</tr>
<tr>
<td>Monopile diameter, thickness</td>
<td>6 m, 0.06 m</td>
</tr>
</tbody>
</table>
The wind and wave loads on the OWT was characterized according to International Electrotechnical Commission (IEC) standards for offshore wind turbines, including IEC 61400-1 (IEC 61400-1 2005) and 61400-3 (IEC 61400-3 2009), and Det Norske Veritas (DNV) guideline commonly known as DNV-OS-J101 (DNV 2014) for offshore wind turbines. Design Load Case (DLC) pertaining to ultimate limit state, and a 50-year return period environmental loads were applied to the structure for determining the peak response of the OWT model.
1.4.2 Reliability Analysis

To investigate the probabilistic failure phenomenon with the variation in wind and wave characteristics, the reliability technique that has been widely used in past studies (Thoft-Christensen and Baker 1982; Madsen et al. 1986; Sorensen 2004) was utilized. One common method to perform such analysis is by means of First Order Reliability Method (FORM) (Thoft-Christensen and Baker 1982; Bjerager 1991; Zhang et al. 2015, Tarp-Johnsen et al. 2003; Sorensen and Tarp-Johansen 2005). FORM regarded as the reliable simulation data-based reliability analysis technique (Thoft-Christensen and Baker 1982; Bjerager 1991; Zhang et al. 2015) was utilized for the fragility analysis. In detail, FORM is an analytical approximation method where structural reliability is computed as the integration of uncertain quantities, $X$, wave loads or multiple wind speed and wave loads, over the failure region modeled by the following function as represented in Eq. (1.1).

$$P_f = \int F(X) \, dX$$

(1.1)

The reliability of the structure ($P_r$) is the ability of the structure to withstand a specified load for a certain period. It can be expressed in terms of exceedance probability ($P_f$) of the structure as shown in Eq. (1.2).

$$P_r = 1 - P_f$$

(1.2)

The reliability of the structure is determined by initially defining a Limit State (LS) function or g-function referring to structural failure. The LS function is expressed as a function of structural capacity ($R$) and imposed loads ($S$). The LS function for the
exceedance probability estimation (Tarp-Johansen et al. 2003; Sorensen and Tarp-Johansen 2005) is shown in Eq. (1.3).

\[ g(z,X) = R_z (z,F_y) - S \]  \hspace{1cm} (1.3)

where \( R_z = z \times F_y \) is the flexural resistance as a function of the plastic section modulus and the design value of yield strength of the tower, while \( S \) is the bending moment at the critical section of the tower. To create the flexural fragility analysis, \( R_z \) represents the available bending strength, whereas \( S \) denotes the bending demand resulting from the applied loads.

For a specific LS of interest, structural failure of the system occurs when \( g(z,X) < 0 \). The LS function is modeled as a lognormal function because of its likelihood to follow a lognormal distribution (Czarnecki and Nowak 2008; Nejad et al. 2014; Wang and Kulhawy 2008). Thus, the exceedance probability data points required for the generation of wave fragility curves was determined using reliability index (\( \beta \)) of the OWT. It is noteworthy that the reliability index is a useful indicator in computing the exceedance probability for measuring the probabilistic safety margin of the structure at a given wave load condition. The index can be expressed in Eq. (1.4).

\[ \beta = \frac{\ln \left( \frac{\mu_R}{\mu_S} \sqrt{\frac{1 + V_s^2}{1 + V_R^2}} \right)}{\sqrt{\ln((1 + V_s^2)(1 + V_R^2))}} \]  \hspace{1cm} (1.4)

where \( \mu_R \) and \( \mu_S \) represents mean for the structural capacity and imposed load and \( V_R \) and \( V_S \) are the corresponding coefficient of variation, respectively. The exceedance probability is determined by performing standardized normal distribution of the reliability index and is expressed as shown in Eq. (1.5).
Modeling Approach for Wave Vulnerability Analysis

The OWT model is simulated using three different computational approaches for varying wave heights to investigate the flexural demands at the mudline affecting the fragility behavior. A brief description and discussion along with fragility data comparison for each approach are presented in the following subsections.

1.5.1 Morrison’s Equation

As the first approach, the Morison’s equation is used. This equation involves the determination of static hydrodynamic forces comprising of regular wave acting on the OWT structure (Arany et al. 2015b). The OWT model resembles “NREL Baseline 5 MW OWT” model defined by NREL. There are several theories that describe the shape and kinematics of regular wave on the structure. The basic theory that is commonly used in the calculation of wave force in offshore structures is dependent on water depth \(d\), wave height \(h\), and wave period \(t\). To characterize the wave-induced vulnerability, wave height parameter was considered as a variable. The Morison’s equation involved hydrodynamic drag \(C_d\) and inertia coefficient \(C_m\) for estimating the wave force on the OWT. The inertia coefficient depends on several parameters such as Keulegan-Carpenter number, Reynolds number, roughness parameters, and interaction parameters (Morison et al. 1950). The resulting wave force due to non-breaking waves composed of drag and inertial components is shown in Eq. (1.6).

\[
F_t(z, t) = \frac{1}{2} \rho_w D_p C_D u(z, t) |u(z, t)| + C_m \rho_w A_p \dot{u}(z, t) \tag{1.6}
\]
where $\rho_w$ is the density of water (kg/m$^3$), $D_p$ is the diameter of the monopile (m), $C_D$ is
the drag coefficient of the support structure (generally taken between 0.7 and 1.2) (DNV, 2014), $C_m$ is the inertia coefficient (suggested values between 1.5 to 2) (DNV 2014), $A_p$ is the cross-sectional area of the monopile (m$^2$). The velocity function $u(z, t)$ and acceleration function $\dot{u}(z, t)$ is shown in Eqns. (1.7) and (1.8), respectively.

$$u(z, t) = \frac{\pi H_{1/3} \cosh(k(s + z))}{T \sinh(kS)} \cos\left(\frac{2\pi t}{T}\right)$$

(1.7)

$$\dot{u}(z, t) = \frac{2\pi^2 H_{1/3} \cosh(k(s + z))}{T^2 \sinh(kS)} \sin\left(\frac{2\pi t}{T}\right)$$

(1.8)

where $k = \frac{2\pi}{\lambda}$ is the wave number (cycles/m), $\lambda$ is the wavelength of the sea waves (m),
$z$ is the vertical coordinate (m), $t$ is time (seconds), $H_{1/3}$ is the significant wave height
(average of the one-third highest wave height) (m) and $T$ is the wave period (sec). The
expressions for wave number and significant wave height is shown in Eqns (1.9) and
(1.10), respectively.

$$\omega^2 = gk \tanh(kS)$$

(1.9)

$$H_{1/3} = 1.67 \times 10^{-7} \frac{U_{10}^2}{g} F$$

(1.10)

where $\omega$ is the angular frequency (rad/s), $U_{10}$ is the wind speed at a 10 m height above
sea level (m/s), $F$ is the fetch (m), $g$ is the gravitational constant.

Wave forces for drag and inertia components at a wave height of 27 m are
determined throughout the Morrison’s equation as shown in Figure 1.2. The shape of the
total wave force follows the same pattern as in the total inertial force. The peak inertial
force obtained is $4.84 \times 10^4$ KN, whereas the peak drag force is $0.38 \times 10^4$ KN. It
indicated that the wave load for the OWT model was dominated by the peak inertial force.

Figure 1.2: Sample profile for total drag, total inertial, and total wave forces acting at MSL at a wave height of 27 m.

1.5.2 Finite Element Analysis

The second approach involves the application of FEA methodology to determine the flexural demand at the mudline section of the monopile OWT using SAP2000 (SAP2000 Manual). The base material used in the model is American Society for Testing and Materials (ASTM) (ASTM 2004) A572Gr50 steel. The OWT model resembles “NREL Baseline 5 MW OWT” model defined by NREL. For the modeling, the OWT was broadly classified into superstructure and substructure systems. The superstructure comprises of tower and rotor, while the substructure consists of monopile foundation extending from the base of the tower to 20 m beneath the mudline. The OWT was modeled with frame elements. The dead load associated with the superstructure were
centered at the top of the tower acting along the vertically downward direction. The wave loads, in addition to buoyancy forces, were applied following the American Petroleum Institute Working Stress Design (API WSD) guideline (API RP2A-WSD 2007) above the mudline for each wave height. To eliminate complexity in determining wave characteristics, the unidirectional wave was assumed with kinetics factor of 1.0. To account for soil-monopile interactions, translational springs were also attached to the substructure beneath the mudline and the soil for the modeling was assumed to be uniform sandy soil with spring stiffness of 42 MPa (Salgado et al. 2000). The monopile OWT model was simulated at a wave period of 10 seconds to determine the peak value of OBM at mudline for each wave height. A representative schematic of the OWT model is shown in Figure 1.3.
Figure 1.3: Representative schematic for the FEA simulation in SAP2000 with wave loads applied to the structure.

1.5.3 FAST

The third approach to calculate flexural demand (i.e., bending moment) at the critical mudline region was done by CAE tool, developed by NREL known as FAST v8. The selected monopile 5 MW OWT model resembles “NREL Baseline 5 MW OWT Test 19” model available in the FAST software. To directly compare the results from the first and second approaches, the loading scenarios were kept similar with wave load dominating the structural failure. For the hydrodynamic loading on the monopile, a linear regular unidirectional wave was applied to the model using HydroDyn (Jonkman et al. 2014) features coupled with FAST. The HydroDyn results in time-domain hydrodynamic
loads at the mudline region for a given wave loading condition. The FAST simulation was able to predict the bend moments of the OWT in three directions, which are fore-aft, side-to-side, and yaw directions. For the simulation, the peak value of bending moments from the three directions at the mudline region was determined. To show the time-elapsed OBM distribution along the three directions at the mudline, Figure 1.4 shows the representative OBM distribution for a given wave height of 27 m. Due to the unidirectional wave loadings, the OBM profile shows significant variation with respect to time in the fore-aft direction, while the OBM variation is lower in side-to-side and yaw directions. The peak value of a fore-aft bending moment, side-to-side bending moment, and yaw moment at mudline are $3.29 \times 10^8$ KN-m, $2.13 \times 10^7$ KN-m, and $1.13 \times 10^6$ KN-m, respectively.

Figure 1.4: Fragility curve of OBM at mudline with varying wave heights.
1.5.4 Comparison of Simulation and Fragility Data

Simulation data from each approach were graphically compared in terms of peak OBM values. The peak OBM values are shown in Figure 1.5. It can be noticed that an increase in wave height causes the increment in the peak OBM values for all three approaches. It is further observed that the Morison’s and FAST approaches exhibit the similar trend for all given wave heights, while the FEA approach resulted in relatively lower OBM values up to the wave height of 33 m. Above the wave height of 33 m, the peak OBM values from the FEA approach exceeded those from Morison’s and FAST. The difference could be attributed to the fact that the FEA approach involved idealized linear-elastic failure mechanism.

![Figure 1.5: Peak OBM values of the OWT model at the mudline observed over wave height obtained using three approaches.](image)

In addition to the comparison of simulation data, vulnerability data obtained from all three approaches were compared in the form of fragility curves representing the OWT exceedance probability data points with respect to a wave height as shown in Figure 1.6.
The figure indicates that the increase in a wave height induces the increase in the exceedance probability for all the three approaches. Interestingly, the exceedance probability values resulting from the FEA and Morison approaches reach nearly 1.0 when the wave height is 35m, while that for FAST is close to 0.96. It can be observed that the curves using Morison’s equation and FAST model show similar behavior until wave height reaches 20 m. Above 20 m, the exceedance probability resulting from Morison’s equation is slightly higher than that of the FAST model. The fragility behavior from the FEA model up to 22 m is almost a flat line, resembling zero exceedance probability. The exceedance probability then accelerates rapidly when the wave height is increased above 22 m of wave height. It can be deduced that OBM in the mudline increases significantly when the wave height reaches higher than 22m, causing higher exceedance probability. Such a large difference in the wave fragility occurs from the fact that the model developed in FEA involved frame elements representing ideally linear elastic-plastic failure mechanism, causing a sharp increase in FEA fragility curve.

Additional exceedance probability point as a reference value is included in Figure 1.6 to be compared with the computational fragility curves. The point of $6.87 \times 10^{-4}$ was determined based on the Tarp-Johansen et al. (2003) outlined the ISO standard 19902 wave load of return period 50 years. The point for the design standpoint is almost zero due to the extremely less conservative load and high resistance assumptions specified in Tarp-Johansen et al. (2003).
On comparison of median values gained from each fragility curve, it can be found that median wave height values corresponding to the median exceedance probability (50%) from the Morison’s equation, FEA model, FAST model are 24m, 29m, and 25m, respectively. Generally, the difference in the median exceedance probability can be associated with the fact that each approach used its own modeling and simulation assumptions. The Morison’s equation followed the classical mathematical function for hydrodynamic loadings, while FEA model used API specification, and the FAST model used HydroDyn functionality with consideration of linear wave. The dynamics effect is not included in Morison’s equation, whereas both FEA and FAST models included the dynamics involved in the wave height, causing Morison’s equation to be most conservative than others. From the figure, FEA experiences the exceedance probability of 0.005 at 23 m reached to 0.996 at 35 m wave height, while FAST has that of 0.311 at 23 m reaching 0.956 at 35 m. The increase in wave height beyond 23 m causes a significant
increment in the FEA and FAST fragility curves. With the increase in wave height beyond 23 m to 35 m, the exceedance probability increased by 67.3% in the FAST fragility curve as opposed to 99.5% for the FEA based curve. It can be concluded that FAST with reasonably conservative fragility results, which lies between Morison and FEA curves, could be more useful in assessing the structural vulnerability of OWTs in extreme hazard offshore environment.

1.6 FAST Approach for Multi-Hazard Fragility Analysis

OWTs experience structural damage to collapse due to multi wind-and-wave loadings rather than single wave loads. It is necessary to perform multi-hazard fragility analysis capable of estimating the stochastic exceedance mechanisms of the OWTs in the vicinity of multiple loading conditions. Because high computation costs required for the multi-hazard fragility analysis from all the approaches and FAST resulted in reasonably conservative wave fragility results, FAST was selected for further analysis. The following subsections for the FAST modeling, simulation, and fragility data are provided.

1.6.1 FAST Modeling

To investigate realistic multi-hazard OWT behaviors, the target site is defined as the Nantucket Sound, Massachusetts, U.S. The site considered is Cape Wind project, which is a favorable site for the first offshore wind farm in the U.S. The site has an ideal wind and wave characteristics for the future wind farms, with a wind speed ranging from 0.36 m/s to 40.28 m/s with a mean value of 8.8 m/s, wind gust ranging from 2.6 m/s to 18.3 m/s with mean 4.47 m/s, wave height ranging from 0.3 to 3.5 m with a mean value of 1.4 m, a wave period from 3.5 to 9.5 sec with a mean value of 6 secs, and a wavelength of 19 to 94 m with a mean value of 50 m (Swanson and Subbaya 2006).
For wind loadings in the site, a 10-minutes mean hub height wind speed was defined following a Rayleigh distribution. It implies that the wind load on the tower can be experienced in the lateral direction. To generate a hub-height wind speed, the Extreme Turbulence Model (ETM) as specified in IEC 61400-3 (IEC 61400-3 2009) was employed. The value of ETM parameter \( c \) was taken as 2.0. The wind profile followed a power-law profile on the rotor disk and logarithmic profile along the tower height. To account for turbulence in wind speed, IEC Class A (IEC 61400-3 2009) wind regime was assumed for the site, with Class A being the most turbulent. Kaimal power spectral density functions (Jonkman 2009) were employed to describe the turbulence model with a power-law exponent of 0.2 and surface roughness of 0.03 m.

For the wave loading, wave kinematics and hydrodynamic loads were represented by JONSWAP spectrum as defined in IEC 61400-3 (IEC 2009). To eliminate complexities in the multiple hydrodynamic loads, current and ice loads were ignored, and the wave propagation direction was assumed unidirectional. Given wave height and the spectral period for each time-domain simulation, FAST established the target wave spectrum and then randomly generated linear irregular waves.

FAST provided a number of interfaces for allowing interaction between external load and structural properties. For the wind loading, the program TurbSim (Jonkman, 2009) simulator was used in extension to the FAST software. The wind velocity was simulated in a 2-Dimensional (2D) grid representing the rotor plane. The turbulence was applied at each grid and then was added to the mean wind speed. The mean wind velocity ranged from 3 m/s to 30 m/s in the analysis. The aerodynamic forces along the OWT were determined using AeroDyn feature (Moriarty and Hansen 2005) in the FAST
simulator, which takes into consideration the aero-elastic behavior. The hydrodynamic loadings were simulated using the HydroDyn (Jonkman et al. 2014), and SubDyn (Damiani et al. 2015) was used to simulate the multi-hazard behavior of the substructure.

1.6.2 Blade Response

A blade is considered as the most flexible component of the OWT structure (Arrigan et al. 2011). When subjected to turbulent wind flow, the tip of a blade experienced a large amount of deflection during the multi-wind-and-wave hazard simulation, causing, in some extreme cases, critical impacts (e.g., strikes and collision) and severe damage on the OWT. The response in terms of the bending moment at the blade root was observed in FAST in two directions as Out-of-Plane root Moment (OPM) and In-Plane root Moment (IPM). The OPM is the moment induced on the root of the blade due to wind loadings, while the IPM is moment observed due to its self-weight. To illustrate the profile of the OPM and IPM at a given loading condition, the respective OPM and IPM for a specific turbulent hub-height wind speed of 25 m/s and significant wave height of 5 m is shown in Figure 1.7(a). The variability is observed for OPM and IPM over the simulation time. It is apparent that the effect due to OPM is significantly higher in comparison to that for IPM. The peak value of OPM observed is $1.91 \times 10^4$ KN-m, while the value for IPM is $1.08 \times 10^2$ KN-m for the given wind and wave scenario.
Figure 1.7: Blade root moment of an OWT for a wave height of 5 m and a) a turbulent wind speed of 25 m/s and b) varying wind speed.

To observe the variability in peak OPM and IPM with respect to wind speed, the peak value of OPM and IPM is plotted over the range of different wind speeds in Figure 1.7(b). It can be observed that the wind speed causes an increment in both the OPM and IPM response. The peak value of OPM and IPM is observed as $2.62 \times 10^4$ KN-m and $1.06 \times 10^4$ KN-m, respectively at a wind speed of 35 m/s. A large difference between OPM is observed at a wind speeds of 5 m/s and 10 m/s, with values of $8.73 \times 10^3$ KN-m and $2.62 \times 10^4$ KN-m, respectively.

Because of the significantly higher value of OPM bending moment in comparison to IPM in Figure 1.7(b), further investigation is done to explore the effect of both wind speed and wave height on the blade response in terms of out-of-plane blade tip deflection. The blade-tip deflection is the amount of displacement observed at the tip of the blade in out-of-plane, in-plane, and the twisting directions of the OWT under the action of multiple wind-and-wave loadings with the vertical self-weight. Figure 1.8 represents a 3-D graphical representation of the out-of-plane blade tip deflection with the variation in
wind and wave parameters. It is obvious that the wind increment caused the blade tip deflection to increase, while the increment in a wave height had little impact on the response in terms of blade tip deflection. Specifically, with an increase in the wave height, the blade tip deflection observed at 6 m/s wind speed is 3.63 m at 0 m wave height, which rises to 4.67 m at the wave height of 30 m with an increase of 25.06% in the observed tip deflection. At a higher wind speed of 30 m/s, the tip deflection at the wave height of 0 m is 14.11 m, which increased to 14.45 m at the wave height of 30 m with an increase of 2.39% in the observed response. Due to the higher inclination of blade tip deflection with respect to wind speed variation, it can be deduced that the blade response is dominated by the hub-height wind speed rather than wave height as expected.

Figure 1.8: Variation of blade-tip displacement observed with change in mean hub-height wind speed and significant wave height.

1.6.3 Tower Response

It is well known that the base of the OWT tower at the mudline region exhibits higher bending moment due to the combined action of wind and wave forces (Arany et al.
Multiple simulations were performed to determine the peak tower bending moment that was conditional on the wind speed and wave height.

The response resulted from the FAST simulation was recorded for the mudline OBM of the OWT. The OBM variation profile over simulation time at the mudline for a wind speed of 25 m/s and significant wave height of 5 m is shown in Figure 1.9. It can be seen that peak OBM increases with an increase in wind speed. It appears that the effect of OBM is insensitive to wave height below 10 m, but the increment of wave height above 10 m caused OBM profile to increase for all values of wind speed. The peak value of OBM is observed at 30 m/s wind speed and wave height of 15 m. It can be interpreted that wind speed has a major impact on mudline while the wave height has a minor impact on the observed response for OBM.

Figure 1.9: Variation in OBM observed in three directions over simulation time.
To further observe the effect of the fore-aft directional bending moment with respect to combined wind speed and wave height variation, a 3-D surface plot is developed for the observed peak fore-aft OBM with variation in wind speed and wave height as shown in Figure 1.10. It can be seen that peak OBM increases with an increase in wind speed. It appears that the effect of OBM is insensitive to wave height below 10 m, but the increment of wave height above 10 m caused OBM profile to increase for all values of wind speed. The peak value of OBM is observed at 30 m/s wind speed and wave height of 15 m. It can be interpreted that wind speed has a major impact on mudline while the wave height has a minor impact on the observed response for OBM.

Figure 1.10: Observed OBM in fore-aft direction with variation in mean hub-height wind speed and significant wave height.

1.6.4 Fragility Behavior

To look into multi-hazard risk associated with the OWT subjected to wind and wave loads, the fragility surface with respect to variation in wind speed and wave height
was created. As mentioned earlier, the blade tip displacement was found to be insensitive to wave height, while variation in wind speed and wave height for fore-aft direction mudline OBM caused some level of variability in the observed response. Therefore, the fragility analysis for multiple wind-and-wave loadings was performed for only fore-aft OBM at the mudline. In this analysis, the computed OBM at the mudline was used in the predefined LS function as shown in Eq. (1.1), and the reliability index was calculated for each simulation. Finally, the exceedance probabilities in the form of fragility surface for the given loading scenarios were determined.

![3-Dimensional fragility surface observed for OBM at mudline with respect to variation in wind speed and wave height](image)

**Figure 1.11:** 3-Dimensional fragility surface observed for OBM at mudline with respect to variation in wind speed and wave height.

Figure 1.11 represents 3D fragility surface generated for OBM at mudline with consideration of both wind speed and wave height. It can be seen that the fragility surface has a small increment in the exceedance probability until wind speed reaches a wind speed of 11.5 m/s. Further increment in wind speed caused the exceedance probability to increase for all wave heights. At a low wind speed of 6 m/s, the increase in wave height
causes the exceedance probability to increase from $4.71 \times 10^{-13}$ at 0 m wave height to 0.06 at 15 m wave height. On the other hand, for the high wind speed of 30 m/s, the increase in wave height caused an increment in the probability from 0.51 at 0 m wave height to 0.68 at a wave height of 15 m. It can be further observed that considering wind speed of less than 12 m/s and wave height below 10 m, the increase in exceedance probability is 49% observed between 0 m wave height and 6 m wave height at a wind speed of 12 m/s. It can be marked that a value of wind speed (30 m/s) and wave height (15 m) can result in significantly higher exceedance probability of 0.68. For the considered Nantucket region having a peak wave height of 3.5 m, the fragility surface exhibits a median exceedance probability when the wind speed in the site exceeds 28 m/s as shown by a red dotted line in Figure 1.11.

1.6.5 Comparison of Fragility Curves

The comparison of OBM fragility curves at mudline for single regular wave loads and multiple linear irregular wind-and-wave loads utilizing FAST approach is shown in Figure 1.12. In Figure 1.12(a), the comparison on exceeding probability over the range of wave height is done for the regular and irregular wave condition. It was observed that the exceedance probability of regular wave is less than that of the irregular wave until the wave height of 22 m is reached. Above 22 m wave height, the exceedance probability for the regular wave increases rapidly with the increase in wave height reaching the exceedance probability of 0.80 at a wave height of 30 m. The mean and standard deviation of the moment due to irregular wave are lower than those for the regular wave due to irregular wave characteristics. This trend can be observed in the past work by Li and Lin (2012).
Figure 1.12: Comparison of fragility curves of OWT for mudline OBM with respect to wave height: a) regular and irregular wave and b) different wind speed for an irregular wave.

Additional fragility curves observed in Figure 1.12(b) are developed for linear irregular wave loading conditions considering the turbulent wind speed between 6 m/s and 30 m/s, highlighting the importance of wind effect under the wave conditions. It is clear that the increase in wind speed caused the exceedance probability to increase for a particular wave height. For instance, at a wave height of 30 m, the exceedance probability of 0.58 is observed for 6 m/s wind speed, which increased to 0.87 for 30 m/s wind speed. The increase in exceedance probability values with the increase in wind speed for different wave height is observed. The increase in wave height from 24 m to 30 m at a constant wind speed of 12 m/s cause 8.31% relative increment in the exceedance probability/ while increase in wind speed from 18 m/s to 30 m/s at a constant 30 m wave height caused 14.56% increase in exceedance probability. The percentage increment in exceedance probability reduces on increase in wave height for a particular wind speed with 35.10% when wind speed increased from 18 m/s to 30 m/s at 6 m of wave height,
whereas the increment is reduced to 14.56% at a wave height of 30 m. It can, therefore, be concluded that the increase in both turbulent wind speed and irregular wave height increases the exceedance probability of the structure, an extreme wave height has a significant contribution to the exceeding probability of the structure at the mudline region.

1.7 Conclusions and Future Work

Structural fragility analyses of the 5 MW monopile Offshore Wind Turbine (OWT) under different wave conditions and multi-wind and wave loads were performed. The wave fragility analysis was first performed using three computational simulation approaches, including Morison’s equation, FEA, and FAST. Each computational approach in addition to the reliability technique was used to determine the exceedance probability estimates for Overturning Bending Moment (OBM) demands at a critical mudline region of the monopile OWT. From the detailed comparison among all three approaches, the FEA approach resulted in the least conservative wave height value (29 m), while the Morison’s equation resulted with the most conservative wave height (24 m) followed by the FAST with a wave height value of 25 m being reasonably conservative. This can be attributed to the fact that Morison’s equation produced larger OBM values than the others with consideration to static behavior for the hydrodynamic forces and FEA model considered the linear-elastic-plastic failure mechanism causing a sharp rise in the fragility curves at a high wave height above 20 m. On the other hand, the FAST model had a capability to involve an interaction of aero-hydro-sub-dynamics within the OWT model, resulting in a moderate conservative range in the fragility curve.
The FAST was further used in multi-wind-and-wave simulations to analyze the response of blade and tower of the OWT. For varying amounts of wind speed and wave height, the blade-tip deflection increased with the considered range of wind speed but remained insensitive to wave height. The OBM profile at the mudline region of the tower significantly changed to both wind speed and wave height variation. Therefore, OBM fragility analysis on the OWT model was done to evaluate its probabilistic structural performance under combined wind and wave loading conditions. It was observed that the exceedance probability increased with incremental wind speed and wave height. For the considered Nantucket region, the exceedance probability increased by 50% when the wind speed exceeded a value of 28 m/s. The comparison of regular wave and irregular wave fragility curve revealed that the exceedance probability determined for the irregular wave is relatively less than that of the regular wave.

Although the fragility data for the OWT were used to estimate its natural hazard vulnerability, irregular wave characteristics associated with wind loads were neglected. Therefore, the effects of irregular wave prevailing in shallow water depth can be considered for future work. Further, an additional study to validate the accuracy of three different approaches with experimental data is needed. The vulnerability and simulation of the OWT subjected to irregular turbulences with above 30 m/s wind speed and breaking wave with above 15 m of wave height were not investigated; thus, such analysis is needed.
REFERENCES


*National Renewable Energy Lab.* (NREL), Golden, CO (United States); 2015.

analysis and its application to performance-based assessment of jacket

characteristics and impact loads on offshore wind turbines supported by monopiles.”


International Electrotechnical Commission

Electrotechnical Commission.

standards for offshore wind turbine applications in the US Atlantic Ocean: Phase
II.” *Contract*, 303, 275-3000.


Jonkman, J (2010). “NWTC design codes (FAST).” *NWTC Design Codes (FAST)*, NREL, Boulder, CO.


2 CHAPTER 2 STATISTICAL REGRESSION MODEL FOR FRAGILITY ESTIMATES OF OFFSHORE WIND TURBINES SUBJECTED TO EXTREME AERO-HYDRO DYNAMIC LOADS

2.1 Abstract

Structural analysis of Offshore Wind Turbines (OWTs) subjected to extreme wind and wave loading conditions is computationally expensive. Several variables are involved in determining the turbine response which favors a surrogate modeling technique to predict the critical response of the structure to perform the fragility analysis. This chapter develops a regression-based model to estimate the responses of an OWT such as the tower top deflection, the mudline flexure, mudline shear, and blade-tip deflection response of the monopile foundation OWTs when subjected to multi-wind-and-wave loads. The developed models allow for failure analysis of OWTs without the need to perform the complicated nonlinear analysis. The dataset needed to develop the surrogate model was acquired through simulations performed in a simulator developed by National Renewable Energy Laboratory (NREL), known as Fatigue, Aerodynamic, Structures, and Turbulence (FAST). To perform simulations, a wide range of structural and loading properties of existing 5 MW OWTs were considered. Latin Hypercube Sampling (LHS) method was used to design simulation experiments using 20 input parameters to develop 120 different configurations of OWTs. The model was then fitted using a Stepwise Multiple Linear Regression (SMLR) approach to eliminate insignificant parameters among the input parameters. Then by using the significant input parameters, explanatory functions were developed based on the laws of mechanics to fit a regression model. Further, an SMLR approach was again applied for screening unimportant explanatory
functions to develop a more accurate probabilistic model for deformation, flexure, and shear responses. The developed models were then used to perform fragility surface analysis for the OWTs to determine exceeding probability pertaining to wind speed and wave height variation. It was observed that for a given value of wind speed and wave height, the flexural exceedance probability was the most critical among others, while the shear exceedance probability was the least critical. It was also observed that the blade-tip deflection was highly sensitive to the wind speed parameter. Also, the hub height and the tower base diameter were the most critical parameters amongst others when observing median exceedance probability when observed in terms of wind speed and wave height, respectively. It was further observed that modification in the structural properties is recommended to improve the performance of OWTs in multi-hazard loading condition.

2.2 Introduction

Increasing demands for renewable energy in the upcoming decades necessitates the installment of wind turbines. Offshore Wind Turbines (OWTs) installment is favored over the onshore counterparts because of several advantages such as less visual impacts, minimal impact on birdlife, and lower noise and land conflicts (Henderson et al. 2003, Breton and Moe, 2009). Despite being 150% more costly than the onshore wind turbines, the OWTs have the distinct advantage of being exposed to larger installation areas and relatively higher wind speeds, resulting in higher energy yields. However, the OWTs are subjected to external loadings from various environmental sources such as wind, waves, and current (Jonkman, 2008, Agarwal and Manuel 2008, Musial and Ram, 2010) which need careful consideration for their design. The OWTs subjected to multiple loadings must have a high structural capacity to withstand such loadings leading to critical
structure failures (Bossanyi 2003, Bussel 2001, Echavarria et al 2008, Arwade et al. 2011). Reliability assessment of OWTs is thus crucial to estimate the exceedance probability of these structures and to minimize construction and maintenance costs (Madsen et al. 1986, Thoft-Christense and Murotsu, 1986). The reliability of a component is defined as the probability that it will perform its required function under the given set of loading conditions for a specified period. There have been many studies on the reliability assessment of the blades of the wind turbines (Toft et al. 2011, Toft and Sorensen 2011), yet the support structure comprising of tower and foundation has received less attention in this field. This study focuses on the reliability of the support structure to observe the response of the OWT to different loadings.

The support structure of the OWT in this study comprises of a monopile foundation connected rigidly to the steel tower. Such a configuration is typically common for shallow water depths of less than 30 m (Musial et al. 2006). The OWTs are subjected to various forms of hazards including hurricanes and typhoons among others. Research studies suggest that the exceedance probability estimation using reliability analysis ensures the safety of monopile OWTs under such extreme loading conditions (Mathisen and Ronold, 2004, Cossa et al, 2011). The reliability assessment should account for all possible hazards to predict the damage at critical sections of the structure. This assessment could be important in optimizing the design of OWTs and maximizing their power output while lowering the maintenance costs.

The variability associated with load parameter estimation can compromise the accuracy of such reliability assessment techniques. To account for associated variability, dynamic response analysis that considers wind, wave, and current-related variability,
needs to be performed. Computational simulation based probabilistic techniques have been considered a powerful approach in assessing the performance of structures, considering uncertainty related to the environmental conditions (Barata et al. 2002, Taflanidis et al. 2009, Papadrakakis and Lagaros 2002, Taflanidis et al. 2013, Marfekri and Gardoni, 2015). Specifically, for analysis of the OWTs, FAST simulator (Jonkman et al. 2005) has been extensively used to simulate the aerodynamics and structural dynamics of wind turbines (Marfekri and Gardoni 2013, Jonkman and Musial, 2010, Passon et al. 2007, Schepers et al. 2002, Jonkman et al 2008, Rendon and Manuel, 2014). The accuracy of the simulator has been validated for the OWT design by carefully considering the site conditions (Camp 2003). To account for the uncertainties inherent in the structural and loading properties of a site, and to determine the OWTs response to the change in those properties for each site condition requires numerous simulations to be carried out.

To reduce the computational burden of performing numerous simulations, this chapter develops a surrogate regression model to predict the deflection, flexure, and shear responses of the monopile OWTs subjected to extreme wind, wave, and current loads. The probabilistic models will then be used in developing the fragility surface of the OWTs with respect to wind speed and wave height variation. This chapter is subdivided into five sections. The second section explains the background and existing literature for probabilistic assessment of OWTs. The third section discusses the environmental loadings for the OWT simulation. The fourth section presents the procedure for the experimental design and generation of analytical data to predict the behavior of OWTs. The fifth section covers the model development and validation for each mode of failure.
The sixth section performs a graphical comparison between the responses observed from the SMLR model and FAST simulation. The seventh section develops fragility surfaces for deformation, moment, and shear demands. The eighth section summarizes the results of the analysis and the ninth section includes the conclusions and the future work.

2.3 Background and Literature Review

Recently, due to the advancement of computational techniques to perform combined aero- hydrodynamic analysis prevailing to offshore environment, numerous software packages are available for such simulations. Because of the involvement of multibody dynamics within the turbine elements, the wind turbine simulation tools have gained popularity among researchers to study the behavior of the turbine under the different wind, wave, current, and earthquake loadings. All the relevant information relating to the computational probabilistic reliability analysis is presented in the following subsections.

2.3.1 Computational Simulation Tools for Aero-hydrodynamic Loadings

Numerous software tools have been developed for aero-hydrodynamic analysis of wind turbines. Some of the tools include ADAMS (Latino and Handen 2001), GH Bladed (Bossanyi 2009), FLEX5 (Øye 1999), HAWC (Larsen and Handen 2007), PHATAS (Lindenburg 2012), and FAST (Jonkman and Buhl 2005). Such computational tools have been used by many research studies (Seidel et al 2005, Agarwal and Manuel 2009, Haselbach et al. 2013, Mardfekri and Gardoni 2013, Asareh et al. 2016, Koukoura et al. 2016) to perform load and response estimation for fixed bottom foundation OWTs under the combined wind and wave loadings. FAST is one such computation tool developed by National Renewable Energy Laboratory (NREL) for aeroelastic analysis of horizontal-
axis wind turbines. The FAST simulator has the capability to model complex wind turbines having a combination of several rigid and flexible bodies connected to each other using several degrees of freedom. Because of its ability to perform coupled aero-hydro-dynamic simulation over time domain, it has been used widely for simulation studies of fixed bottom OWT (Myers et al. 2015, Hafele et al. 2016, Mo et al. 2017, Carswell et al. 2015, Abdelkader et al. 2017).

2.3.2 Probabilistic Assessment

There have been several studies conducted to perform a probabilistic assessment of wind turbines to predict the extreme loads at critical sections (Madsen et al. 1998, Agarwal and Manuel, 2008, Manual et al. 2001, Nielson and Sorensen 2011). For instance, Mardfekri and Gardoni (2013) conducted a probabilistic assessment monopile OWTs and developed probabilistic demand models for deformation, shear, and moment subjected to wind, wave, and current loadings. The study used the response observed from the 3D nonlinear finite element analysis to validate the probabilistic response and then generated the fragility curves under operational wind speed and wave height. A similar approach was used by Fallon (2012) to develop a probabilistic demand model of an asymmetric offshore jacket platform under serviceability and ultimate limit state. The demand model was further utilized to perform fragility analysis as well as sensitivity analysis with respect to wave height to predict the optimal location of drill pipe.

Additionally, Taflanidis et al. (2013) developed a probabilistic framework based on the probabilistic characterization of uncertainty in the model for risk assessment under extreme wind and wave conditions. Sensitivity analysis was performed, which highlighted the importance of wind speed and wave height under such loadings. Dong et
al. (2011) developed a joint probabilistic model incorporating mean wind speed, significant wave height, and peak spectral period for fatigue analysis of fixed offshore wind turbines. The study showed wind load was a dominant factor for fatigue damage. Avossa et al. (2017) developed a probabilistic framework for onshore wind turbine subjected to combined wind and seismic loads for vulnerability assessment for parked and operating wind condition. Wei et al. (2016) performed a performance-based assessment to calculate the extreme OWT response resulting from nonlinear elastic analysis, followed by fragility curves for damage, yield and collapse damage states for jacket foundation OWT. Wei et al. (2014) developed wind-and-wave induced demand on OWT by applying nonlinear static analysis i.e., Incremental Wind Wave Analysis to determine the structural response to increasing wave heights and wind speeds. Rendon and Manuel (2014) predicted long-term behavior of the 5 MW OWT highlighting the importance of variability in multi-wind-and-wave loads in such analysis. The research studies have concluded that the probabilistic assessment could be an effective technique for predicting the exceedance probability of OWT, and the fragility curve is an efficient representation of exceedance probability of the structure.

2.3.3 Reliability Assessment

Researchers have investigated the reliability of wind turbine based on historical data relating to their failures and the relevant cost associated with them (Walford 2006, Tavner et al. 2007, Sorensen 2009). Those studies highlighted the importance of such analysis in reducing the maintenance cost. Structural reliability is related to determination and prediction of exceedance probability relating to limit states of a structure at a given loading condition (Choi et al. 2006). Sorensen and Toft (2010) developed a methodology
for the probabilistic design of wind turbines resulting in high reliability and low cost by the consideration of random variables to model for uncertainty in the computational model. Carswell et al. (2014) performed reliability analysis of OWT monopile foundation using probabilistic methods. All the findings suggest the effectiveness of probabilistic design method to achieve structural reliability.

Fragility analysis has been used extensively for assessing the vulnerability of the OWT structures, as it offers an exceedance probability data over a range of potential loadings (Cheng et al. 2002, Vahdatirad et al. 2014, Quilligan et al. 2012, Myers et al. 2012, Mardfekri and Gardoni 2013). For instance, Cheng et al. (2002) studied the extreme response of OWTs under operational wind and wave loading using the maximum peak over threshold method and estimated structural fragility by applying Bayesian reliability approach. Vahdatirad et al. (2014) performed reliability analysis, using the probabilistic finite element model to characterize the uncertainties relating to the structural parameters, to analyze the deflection, bearing capacity, and stiffness of a monopile foundation. The study resulted in the effectiveness of such analysis to calibrate the code-based design procedure. Quilligan et al. (2012) applied the probability approach, using the theory of total probability, for the fragility analysis of the onshore wind turbine as a function of tower material type, hub-height, and wind speed. The fragility curve developed for the range of hub-height wind speed showed failure probability of tower type with respect to wind speed. Myers et al. (2012) determined fragility curve of 2.4 MW onshore wind turbine tower for yielding using incremental dynamic analysis as a function of ground motion intensity.
2.3.4 Summary

Based on the literature review, several studies have attempted to develop probabilistic models to determine exceedance probability estimates of OWTs. However, there has not been a detailed study available to date, to predict the response for more common 5 MW OWTs utilizing detailed structural and environmental parameters available, to estimate multi-hazard exceedance probability.

2.4 Theoretical Aspects of Load Modeling in FAST

The OWTs are designed to be installed in an offshore environment that is several distances from the coast. It is exposed to numerous stochastic loadings, which must be resisted by the structure. The schematic representation of OWT showing the structural properties along with the loading properties of the structure is shown in Figure 2.1; the loads comprises mainly of aerodynamic and hydrodynamic loadings. Those slender cantilever OWT tower in the vicinity of external loadings results in critical deflection at the tower top, mudline flexure, mudline shear force, and blade tip deflection. This study analyzes the peak responses observed in the OWT structure from the non-linear aerodynamic simulation at those critical sections of the structure.

As mentioned in the previous sections, the wind turbine simulator tool, FAST is used for the dynamic simulation of the OWT considered in this study. The FAST models a tower and the three blades as a cantilever beam rigidly fixed to the ground and the rotor hub, respectively. The tower flexibility can be determined in either transverse or longitudinal direction with respect to the wind and can be represented by two fore-aft and two side-to-side modes for mode shapes. A coupled model of the OWT structure in the FAST simulator was used to define the wind and wave force time-histories and to
determine the peak response of the structure on such combined loadings. The techniques for defining the aerodynamic and hydrodynamic loads are defined briefly in the following subsections.

Figure 2.1: Schematic representation of Offshore Wind Turbine with structural properties and environmental loadings parameters.

2.4.1 Modeling of Aerodynamic Loads

The time-history analysis of wind loads at the hub-height of the turbine was performed using a TurbSim simulator (Jonkman 2009). TurbSim is a stochastic, full-
field, and turbulent wind generating simulator which is widely used in conjunction with
FAST to represent turbulent wind conditions. For the analysis, IEC Kaimal spectral
model as defined in IEC 61400-3 was used (IEC, 2009). The Kaimal spectrum is
dependent on wind speed at hub height ($V_{hub}$) for the three wind components in u, v, and
w direction is shown in Eq. (2.1).

$$\frac{f_v S_k(f_v)}{\sigma_k^2} = \frac{4 f_v L_k}{V_{hub}^5} \left(1 + 6 f_v L_k V_{hub}^5\right)^{\frac{3}{5}} \quad (2.1)$$

where $k$ is an index referring to the direction of wind speed, $f_v$ is the frequency, $S_k$ is the
single-sided velocity component spectrum, $\sigma_k$ is the standard deviation of the wind
component, and $L_k$ is the integral scale parameter for wind component. Additionally, IEC
wind coherence applies two-dimensional Taylor Frozen theory in the longitudinal
direction to account for spatial correlation structure of the longitudinal velocity
component as shown in Eq. (2.2).

$$\text{Coh} (r, f) = \exp \left\{ -12 \frac{f_v}{V_{hub}} \left( \frac{0.12 r}{L_{sc}} \right)^2 \right\} \quad (2.2)$$

where $r$ is the magnitude of separation vector which is normal to the average wind
direction, $L_{sc}$ is the coherence scale parameter.

For the turbulence model, the Normal Turbulence Model (NTM) was used to
define turbulence intensity. The IEC guidelines specify wind speed in NTM as a 10-
minute mean wind speed at hub height. The wind profile type was taken as IEC profile,
which is the power-law profile on the rotor disk and logarithmic profile elsewhere. The
power-law exponent as stated in the IEC guidelines is taken as 0.14 for NTM model with the surface roughness length as 0.03.

For the aerodynamic forces on the rotating blades, FAST used another internal subroutine AeroDyn module (Moriarty and Hansen 2005) to calculate the blade wind load. The result from the AeroDyn simulation was then applied to the tower as an external load in addition to wave and current loads. The static force on the vertical wind tower member can be expressed as shown in Eq. (2.3).

\[
F = \frac{1}{2} \rho u^2 C_s A
\]  

(2.3)

where \( \rho \) is the mass density of air, \( u \) is the mean wind speed at hub-height, \( C_s \) is the shape coefficient of circular section which depends on Reynolds number and is set to 0.5 for circular sections, and \( A \) is the projected area of the tower facing the incoming wind. The static force was then superimposed with the aerodynamic loads obtained from FAST simulation.

2.4.2 Modeling of Hydrodynamic Loads

The hydrodynamic loading due to waves and current was performed in a HydroDyn module (Jonkman et al. 2014) of the FAST. For the monopile OWT, the modeling assumed strip-theory approach as recommended for fixed-bottom substructure (Song et al. 2012). The incident wave kinematic model for linear irregular waves followed the Joint North Sea Wave Project (JONSWAP) spectrum. The sea wave load on the monopile following the JONSWAP spectrum (\( S_{\eta \eta} \)) has the form as in Eq. (2.4).
\[ S_m(f) = 0.3125 H_s^2 T_p \left( \frac{f}{f_p} \right)^{-5} \exp \left( -1.25 \left( \frac{f}{f_p} \right)^{-4} \right) (1 - 0.287 \ln v) v \left[ \exp \left( -0.5 \left( \frac{f}{f_p} \right)^2 \right) \right] \]  

(2.4)

where \( H_s \) is the significant wave height, \( T_p \) is the peak spectral period, \( v \) is the peak-shape parameter (generally taken as 3.3), \( f \) is the frequency (in Hz), \( f_p \) is the peak frequency \( \left( = \frac{1}{T_p} \right) \), and \( \sigma \) is 0.07 for \( f \leq f_p \) and 0.09 for \( f > f_p \).

The velocity potential of water particles adopts the Laplace equation to simulate the stochastic ocean waves as shown in Eq. (2.5).

\[ \phi(x, z, t) = \frac{gH_s}{2\omega} \frac{\cosh k(d + z)}{\cosh kd} \sin(kx - \omega t) \]  

(2.5)

where \( x \) and \( z \) are the horizontal and vertical directions, respectively; \( \omega \) is the angular sea wave frequency \( \left( = gk \tan(kh) \right) \) (in rad/s); \( k \) is the sea wave number \( \left( = \frac{2\pi}{L} \right) \); \( L \) is the wave length, \( d \) is the depth of water. On differentiating the velocity potential, the velocity and acceleration of water particles were determined. The origin of \( z \) axis is selected at the MSL.

The Morison equation was then used to determine the hydrodynamic forces following the DNV specification for the design of OWT. The horizontal force acting on the small section of cylinder \( dz \) at any structural depth of \( z \) can be written as shown in Eq. (2.6):  

\[ dF(z, t) = C_M \rho \frac{\pi}{4} d_p^2 \ddot{u}(z, t) dz + C_D d_p \frac{1}{2} \rho u(z, t) |u(z, t)| dz \]  

(2.6)

where \( C_M \) and \( C_D \) are the inertia coefficient and the drag coefficient, respectively; \( d_p \) is the monopile diameter, \( \rho \) is the density of water. The bending moment on the structure at depth \( z \) can be determined by integrating the force as expressed as shown in Eq. (2.7).
\[ M(t) = \int_0^d zdF(z, t) \] (2.7)

To find the water velocity due to current, which FAST refers to as the sub-surface model, is defined following the power law as shown in Eq. (2.8):

\[ U_{ss}(z) = U_{0ss} \left( \frac{z + d}{d} \right)^{1/7} \] (2.8)

where \( z \) is the depth considered below MSL (negative downward), \( d \) is the depth of water, and \( U_{0ss} \) is the current velocity at MSL. The current was predicted using Morison’s equation as shown in Eq. (2.6).

2.5 Computational Model Development

A set of configurations consisting of different structural and loading parameters were developed to perform multiple simulations. All the publicly available specifications of 5 MW wind turbine model were recorded from the manufacturers of wind turbines and from the previous literature, to estimate the range of the structural components. The extreme range of environmental loads such as wind speed at 75 m/s resembles the extreme wind velocities of Category 5 hurricane (Bell and Montgomery 2008) and turbulence intensity of 0.16 resembles the highest turbulence of IEC wind turbine Class A (IEC 2009). Using the predefined range of structural and environmental properties for 5 MW OWTs, an experimental design technique was used in generating representative configurations. All the possible range of OWTs for the simulation is shown in Table 2.1. Running simulations by varying all the input parameters is practically infeasible. Therefore, Latin Hypercube Sampling (LHS) technique was used to intelligently design the experiments. Then, for each experimental run, the model was linearized to determine
the mode shapes for each experiment. The experiments were then simulated to determine
the peak response of the OWT at the critical section.

Table 2.1: Range of parameters for experimental design

<table>
<thead>
<tr>
<th>Geometrical and mechanical properties for experimental design</th>
<th>Design Parameters</th>
<th>Abbrev.</th>
<th>Ranges</th>
<th>Unit</th>
<th>References</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rotor Mass</td>
<td>R_m</td>
<td>110-120</td>
<td>tons</td>
<td></td>
<td>Various turbine specifications</td>
</tr>
<tr>
<td>Nacelle Mass</td>
<td>N_m</td>
<td>240-290</td>
<td>tons</td>
<td></td>
<td>Various turbine specifications</td>
</tr>
<tr>
<td>Damping Ratio</td>
<td>DR</td>
<td>0.01-0.05</td>
<td></td>
<td></td>
<td>Fontana et al. (2015)</td>
</tr>
<tr>
<td>Hub Height</td>
<td>H_H</td>
<td>80-140</td>
<td>m</td>
<td></td>
<td>Uraz (2011)</td>
</tr>
<tr>
<td>Monopile Depth</td>
<td>M_D</td>
<td>10-50</td>
<td>m</td>
<td></td>
<td>Mardfekri et al. (2015)</td>
</tr>
<tr>
<td>Monopile thickness</td>
<td>M_T</td>
<td>0.068-0.15</td>
<td>m</td>
<td></td>
<td>Rahman and Achmus (2005)</td>
</tr>
<tr>
<td>Tower Base Diameter</td>
<td>T_BD</td>
<td>5.6-7.6</td>
<td>m</td>
<td></td>
<td>Engstrom et al, (2010)</td>
</tr>
<tr>
<td>Tower Top Diameter</td>
<td>T_TD</td>
<td>3.8-5</td>
<td>m</td>
<td></td>
<td>Engstrom et al, (2010)</td>
</tr>
<tr>
<td>Tower Top Thickness</td>
<td>T_H</td>
<td>0.019-0.02</td>
<td>m</td>
<td></td>
<td>Engstrom et al, (2010)</td>
</tr>
<tr>
<td>Tower Base Thickness</td>
<td>T_B</td>
<td>0.027-0.068</td>
<td>m</td>
<td></td>
<td>Engstrom et al, (2010), Uraz</td>
</tr>
<tr>
<td>Steel Type</td>
<td>S</td>
<td>S275, S355, S460, S690</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Young's Modulus</td>
<td>E_s</td>
<td>190-210</td>
<td>GPa</td>
<td></td>
<td>Veljkovic et al. (2012)</td>
</tr>
<tr>
<td>Shear Modulus</td>
<td>V_s</td>
<td>73-78</td>
<td>GPa</td>
<td></td>
<td>Veljkovic et al. (2012)</td>
</tr>
<tr>
<td>Density</td>
<td>D</td>
<td>8100-8600</td>
<td>kg/m^3</td>
<td></td>
<td>Veljkovic et al. (2012)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Loading Parameters</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Wave Height</td>
<td>W_H</td>
<td>1-20</td>
<td>m</td>
<td></td>
<td>Fallon M.B. (2012)</td>
</tr>
<tr>
<td>Water Depth</td>
<td>D</td>
<td>20-30</td>
<td>m</td>
<td></td>
<td>Mardfekri et al. (2015)</td>
</tr>
<tr>
<td>Turbulence Intensity</td>
<td>T_I</td>
<td>0-0.16</td>
<td>m</td>
<td></td>
<td>Mardfekri et al. (2015)</td>
</tr>
<tr>
<td>Wind Speed</td>
<td>W_S</td>
<td>3-30</td>
<td>m/s</td>
<td></td>
<td>Mardfekri et al. (2015)</td>
</tr>
<tr>
<td>Current Velocity</td>
<td>C_S</td>
<td>0.03-2.5</td>
<td>m/s</td>
<td></td>
<td>Fallon M.B. (2012)</td>
</tr>
<tr>
<td>Peak Spectral Wave Period</td>
<td>T_P</td>
<td>3.54-4.56 *</td>
<td>sec</td>
<td></td>
<td>IEC 61400-3 (2009)</td>
</tr>
</tbody>
</table>

Note: Various turbine specifications refer to the NREL (Jonkman et al. 2009), and

2.5.1 Latin Hypercube Sampling

There are many sampling techniques available to reduce experiment sizes, and
thereby reducing the computational cost. Latin Hypercube Sampling (LHS), referred to as
space-filling design, is a sampling technique developed by McKay et al. (1979) which is
commonly used for experiments relating to sensitivity and uncertainty analysis. It is based on the Latin square design, which specifies a single sample in each row and column. The field of structural engineering has been recently exposed to this technique for reliability analysis (Mardfekri and Gardoni 2013, Bernier et al. 2015, Mangalathu et al. 2017). The LHS technique, for ‘n’ sample size and ‘k’ number of variables, generates an n by k matrix having equal probability where each column is a random permutation of \{1, 2, ..., n\} and each row forms a k-tuple of the LHS combination. In the present study, a total of 120 configurations were generated with 20 different structural and loading parameters.

2.5.2 Linearization

Each simulation configuration consists of different structural properties such as hub height and tower dimensions which will result in different mode shapes in a dynamic loading condition. The linearization of the modified structure is, therefore, necessary to determine the system response i.e., tower mass and bending stiffness. Linearization is also required for Eigen-analysis i.e., to compute structural tower bending modes (Jonkman et al. 2008). In general, the linearization process comprises of two steps: 1) determining an operating point, about which the model will be linearized, and 2) linearize the complete nonlinear aeroelastic model about the operating point.

The linearization in FAST v7 can develop second-order linearized representation (generating tower mass, stiffness and damping matrix) of the nonlinear aeroelastic OWT model while FAST v8 is capable of developing only the first order linearized representation (generating state matrix). Therefore, we employed linearization in FAST v7 which resulted in tower mass and stiffness matrix, which was required for the further
analysis. Linearization analysis is performed by FAST following the complete nonlinear aeroelastic equations of motion of the form as shown in Eq. (2.9) described by Jonkman and Buhl (2005).

\[ M\ddot{\mathbf{q}} + C\dot{\mathbf{q}} + K\mathbf{q} = \mathbf{F}_g \]  

(2.9)

where \( M \) is the mass matrix, \( C \) is the damping matrix, \( K \) is the stiffness matrix, \( \mathbf{q}, \dot{\mathbf{q}}, \) and \( \ddot{\mathbf{q}} \) are the vector of displacements, velocity, and acceleration associated with each DOF, respectively, and \( \mathbf{F}_g \) is the generalized force vector associated with external loads.

The linearization analysis resulted in the tower top mass, the moment of inertia along side-to-side, fore-aft, and twist directions and the cross moment of inertia with respect to reference axes which were later used in an input file for BModes. BModes is a finite-element code resulting in dynamically coupled modes for the wind turbine model (Bir, 2005). The tower in BModes was modeled as a cantilever beam having fixed support at the mudline with a lumped mass at the tower top. The output from the BModes added to ModeShapePolyfitting.xls sheet resulted in the mode shape of the tower.

2.5.3 Simulation Experiment

The FAST code was utilized for the simulation of each linearized OWT subjected to the aero-hydrodynamic loadings to determine the responses at the critical mudline region of the turbine. The inflow turbulent wind conditions were generated by TurbSim, the aerodynamic forces were computed using AeroDyn, the tower structural dynamics using ElastoDyn, the hydrodynamic loadings along the support structure by HydroDyn which uses linear wave kinematics to solve wave kinematics, and the foundation behavior using SubDyn (Damiani et al. 2015).
The sample LHS combinations for the model is shown in Table 2.2. The value in each combination represents the selected value of each input parameter for FAST simulation. For example, in combination 1, the following values are taken to run non-linear simulation in FAST: wave height is 13 m, depth of water is 27 m, monopile depth is 72 m, rotor diameter is 130 m, hub height is 107 m, turbulence intensity is 0.14, wind speed is 16 m/s, current speed is 0.3 m/s, wave spectral period is 17.9 sec, damping ratio is 0.020, monopile thickness is 0.11 m, tower top diameter is 4.10 m, tower top thickness is 0.019 m, tower base diameter is 6.47 m, tower base thickness is 0.041 m, rotor mass is 113 tons, nacelle mass is 267 tons, modulus of elasticity is 204 kg/m3, shear modulus is 75 kg/m3, and steel density is 8365 kg/m3. Each simulation yielded in the desired OWT response such as tower top deflection, mudline bending moment, and mudline shear force, and the peak response was utilized to fit the probabilistic model.
Table 2.2. Sample Latin Hypercube Sampling combination

| Comb. | W_H | D   | M_D | RD  | H_H | TI  | W_S | C_S | T_P | DR  | M_T | T_TD | T_H | T_HD | T_Ht | Rm  | Nm  | Es  | Vs  | D   |
|-------|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|-----|
| 1     | 13  | 27  | 72  | 130 | 107 | 0.14| 16  | 0.3 | 17.9| 0.020| 0.11| 4.10| 0.019| 6.47| 0.041| 113 | 267 | 204 | 75 | 8365|
| 2     | 4   | 25  | 36  | 133 | 121 | 0.03| 48  | 0.1 | 4.9 | 0.015| 0.13| 4.71| 0.019| 6.37| 0.048| 117 | 249 | 210 | 74 | 8583|
| 3     | 7   | 20  | 65  | 119 | 95  | 0.06| 33  | 1.8 | 14.9| 0.020| 0.08| 4.12| 0.019| 6.84| 0.048| 113 | 243 | 208 | 76 | 8222|
| 4     | 11  | 22  | 41  | 149 | 103 | 0.15| 30  | 0.4 | 8.8 | 0.013| 0.09| 4.79| 0.019| 7.40| 0.062| 117 | 256 | 200 | 78 | 8134|
| 5     | 3   | 27  | 45  | 146 | 129 | 0.09| 52  | 0.2 | 7.7 | 0.015| 0.09| 4.42| 0.019| 7.20| 0.047| 116 | 282 | 200 | 77 | 8571|
| 6     | 19  | 25  | 62  | 144 | 123 | 0.06| 24  | 2.4 | 4.0 | 0.015| 0.07| 4.84| 0.020| 6.21| 0.054| 118 | 248 | 203 | 76 | 8121|
| 7     | 17  | 26  | 71  | 125 | 107 | 0.10| 62  | 0.6 | 4.2 | 0.019| 0.12| 4.22| 0.020| 6.41| 0.034| 120 | 247 | 207 | 77 | 8386|
| 8     | 5   | 29  | 65  | 131 | 97  | 0.15| 67  | 1.7 | 11.6| 0.011| 0.10| 4.67| 0.019| 5.73| 0.063| 115 | 279 | 209 | 77 | 8230|
| 9     | 19  | 28  | 63  | 117 | 98  | 0.02| 29  | 0.0 | 18.2| 0.016| 0.07| 3.98| 0.019| 5.84| 0.042| 116 | 277 | 207 | 74 | 8167|
| 10    | 12  | 22  | 46  | 151 | 135 | 0.13| 5   | 1.1 | 6.0 | 0.014| 0.15| 4.13| 0.019| 5.87| 0.052| 120 | 286 | 201 | 77 | 8524|
|       |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |     |
| 111   | 1   | 27  | 41  | 126 | 117 | 0.09| 41  | 1.4 | 7.8 | 0.011| 0.09| 4.57| 0.020| 6.42| 0.057| 117 | 255 | 191 | 77 | 8150|
| 112   | 20  | 22  | 48  | 130 | 117 | 0.00| 73  | 0.5 | 17.8| 0.014| 0.07| 4.07| 0.019| 7.28| 0.052| 118 | 253 | 196 | 75 | 8424|
| 113   | 3   | 29  | 57  | 140 | 81  | 0.11| 58  | 2.3 | 8.2 | 0.012| 0.12| 4.20| 0.020| 6.04| 0.051| 114 | 283 | 191 | 74 | 8209|
| 114   | 17  | 25  | 46  | 146 | 127 | 0.09| 3   | 0.4 | 17.5| 0.016| 0.09| 4.81| 0.019| 5.65| 0.062| 119 | 243 | 195 | 78 | 8512|
| 115   | 15  | 21  | 57  | 146 | 84  | 0.12| 30  | 0.5 | 15.0| 0.016| 0.07| 4.29| 0.020| 5.85| 0.046| 116 | 280 | 204 | 78 | 8453|
| 116   | 16  | 22  | 63  | 141 | 94  | 0.01| 15  | 1.3 | 5.2 | 0.012| 0.11| 4.70| 0.020| 6.79| 0.040| 117 | 241 | 206 | 74 | 8600|
| 117   | 8   | 21  | 62  | 120 | 91  | 0.01| 53  | 0.7 | 12.4| 0.014| 0.11| 3.91| 0.019| 7.38| 0.035| 118 | 279 | 191 | 77 | 8533|
| 118   | 7   | 28  | 73  | 133 | 132 | 0.10| 57  | 1.9 | 14.5| 0.010| 0.11| 4.92| 0.019| 5.97| 0.046| 120 | 257 | 196 | 77 | 8499|
| 119   | 14  | 25  | 42  | 149 | 132 | 0.03| 19  | 1.7 | 15.6| 0.013| 0.12| 3.87| 0.020| 7.25| 0.044| 113 | 262 | 199 | 74 | 8243|
| 120   | 9   | 27  | 71  | 141 | 82  | 0.01| 45  | 0.9 | 6.2 | 0.011| 0.14| 4.73| 0.019| 6.52| 0.041| 118 | 260 | 197 | 75 | 8398|
2.5.4 Probabilistic Model Development and Validation

Multiple Linear Regression (MLR) is a common approach to develop a response model for the functional relationship between the variables of interest (Chatterjee and Hadi, 2015, Smith 1999) and to estimate the statistical significance of each individual variable (Cirilovic et al. 2014, Sharma and Singh 2018). It is a specific form of the regression model, where the linear parametric function would be utilized to model a response. The method has shown its effectiveness in the construction industry (Attalla and Hegazy 2003, Lowe et al. 2006, Sadrmontazi et al 2013, Asadi et al. 2014, Jafarzadeh et al. 2014, Khademe et al. 2017). A generalized MLR model can be formulated as shown in Eq. (2.10).

\[ y = \beta_0 + \beta_1 X_1 + \beta_2 X_2 + \cdots + \beta_k X_k + \varepsilon \]  

(2.10)

where \( y \) is the response variable, \( X_i \) is the input parameters, \( \beta_0 \) and \( \beta_i \) represents intercept and regression coefficients, respectively, and \( \varepsilon \) is the random error.

The subsections explain the modeling methodology along with parameter selection process. Furthermore, cross-validation of the developed model is also presented. All the model development and validation task were performed using the R programming environment (Team 2015).

2.5.5 Stepwise Multiple Linear Regression

In general, for the 20 input variables considered in the study, a total of \( 2^{20} = 1048576 \) possible regression models can be developed. Examining all possible models is not practically feasible, therefore, a model development procedure employs the stepwise multiple linear regression (SMLR) algorithms to develop a reduced model.
(Mohsenijan et al. 2016, Jafarzadeh et al. 2014). It is a widely used method in reducing the number of parameters without compromising the prediction accuracy of the model (Kutner 2004, Barrett and Gray 1994, Silva et al. 2013, Mohsenijam et al. 2016).

The SMLR technique starts with the estimation of the most influencing input parameter to develop a model based on statistical outcomes. The coefficient of regression ($R^2$) for a model provides a correlation between the observed response and the predicted response. It should be higher (nearly equal to 1) for a particular model to be considered accurate. However, it might be misleading in some cases having a higher number of input parameters, as the $R^2$ value increases with increase in input parameters (Jongman et al. 1995). In such case, adjusted-R$^2$ ($R^2_a$) is introduced for evaluating the goodness of fit. The $R^2_a$ is a better measure of fitted model with potential to contain more significant parameters in the model than the $R^2$.

The SMLR process begins with a model with just the intercept $\beta_0$. An input parameter is then added to the model if the resulting $R^2_a$ value is higher than in the previous model with just the intercept. This process continues until the model with highest $R^2_a$ is found.

Additional approaches such as determination of Akaike Information Criterion (AIC), the Bayesian Information Criterion (BIC) statistics following Choi et al. 2013 and Holiday et al. 1995 were performed to determine the better performing model for each approach.

Using SMLR approach along with AIC and BIC criterion, three models were selected including the full model to compare the errors observed from each model. All
the insignificant parameters which have no correlation to the observed response are eliminated in this SMLR algorithm, and three models having relatively higher $R_{\text{a}}^2$ values are further analyzed for the model validation. The full model (FM), reduced model following AIC criterion (RM1), and reduced model following BIC criterion (RM2) are shown as a function of associated input parameters in Eqs. (2.11)-(2.13), respectively.

$$FM = f(H_w, W_d, M_d, RD, H_t, TI, W_s, C_s, T_p, DR, M_T, T_{TD}, T_{TT}, T_{BD}, T_{BT}, M_R, M_N, E_s, V_s, D)$$  \hspace{1cm} (2.11)

$$RM_1 = f(H_w, W_d, H_t, TI, W_s, M_T, T_{BD}, T_{BT})$$  \hspace{1cm} (2.12)

$$RM_2 = f(H_w, M_d, H_t, W_s, M_T, T_{BD})$$  \hspace{1cm} (2.13)

2.5.6 Cross-Validation

Cross-validation is one of the most useful methods for determining the accuracy of a fitted model (Stone 1974). Leave-Out-One Cross-Validation (LOOCV) is one of the methods used to determine the predictive performance of a fitted model (Hawkins et al. 2003, Cawley and Talbot, 2003, Wong 2015, Jafarzadeh et al. 2014). This technique is useful when the number of observations is limited. In other cases, where there are a large number of observations, the LOOCV approach can get computationally expensive. However, in our case with 120 simulation results, LOOCV approach is applicable.

The LOOCV technique follows that the given data set of N observations is divided into N-1 observations commonly known as the “training set” to establish the model. Its validity is then checked on the remaining one observation called the “test set” for each observation (Cheung and Skitmore, 2006). The training set of data is involved in calibrating the model while the test set is applied for validation. The predictive accuracy of all the reduced models is checked in terms of cross-validation (CV) errors for each model. The relative comparison of the CV errors of the different models will
determine which model performs better. The model having lower cross-validation error is considered as a more accurate representation of calibrated MLR model, which is then utilized in the fragility estimation.

The resulting models for tower top deflection along with its respective statistical output for model selection is shown in Table 2.3. It can be observed that the $R^2$, AIC and CV error for RM$_1$ is relatively higher than for FM. However, RM$_2$ has the lowest BIC value amongst all, therefore it is taken into consideration. From the cross-validation result, it can be concluded that RM$_1$ performed statistically better than the others, resulting in low CV error, which is used for the further analysis. Similar behavior was observed for the moment and the shear demand model. The accuracy of the model was observed to be relatively low because of the limited number of simulations of a large number of input parameters. This result can be improved by increasing the number of simulations run per input parameter.

<table>
<thead>
<tr>
<th>Model</th>
<th>$R^2$</th>
<th>Adj. $R^2$</th>
<th>CV error</th>
<th>AIC</th>
<th>BIC</th>
</tr>
</thead>
<tbody>
<tr>
<td>FM</td>
<td>0.76</td>
<td>0.72</td>
<td>0.166</td>
<td>99.27</td>
<td>160.59</td>
</tr>
<tr>
<td>RM$_1$</td>
<td>0.75</td>
<td>0.73</td>
<td>0.125</td>
<td>86.08</td>
<td>116.74</td>
</tr>
<tr>
<td>RM$_2$</td>
<td>0.73</td>
<td>0.71</td>
<td>0.55</td>
<td>89.52</td>
<td>111.82</td>
</tr>
</tbody>
</table>

After the SMLR analysis and the cross validation, the predicted response of the OWTs was found to be highly affected by the variation of following parameters: significant wave height ($H_W$), monopile depth ($M_D$), rotor diameter ($R_D$), hub height ($H_H$), turbulence intensity ($T_I$), wind speed ($W_S$), current velocity ($C_S$), monopile thickness ($M_t$), tower base diameter ($T_{BD}$), tower base thickness ($T_{BT}$), rotor mass ($M_R$), and nacelle mass ($M_N$). These parameters were further grouped in terms of explanatory functions.
2.5.7 Explanatory Functions

Explanatory functions are derived for the development of probabilistic demand model utilizing the significant input variables obtained from the regression analysis. The functions were developed following the law of mechanics along with engineering judgment to improve the model accuracy. The developed explanatory function is shown in Table 2.4. In formulating the model, a logarithmic transformation of the data is employed to reduce the skewness of the data (Mardfekri and Gardoni 2013).

The term $H_1$-$H_3$ were selected following Mardfekri and Gardoni (2013) to characterize the influence of wind and wave parameters as a function of hub height ($H_{HI}$), wave height ($H_W$), turbulence intensity ($TI$), and current speed ($CS$). The function $H_4$ was selected to capture the potential influence of rotor diameter ($RD$) and hub height ($H_{HI}$) following Mardfekri and Gardoni (2013) and recommended by Tempel and Molenaar (2002) and Kaiser and Snyder (2012). The effect of extreme wave height, as specified by Dolan et al. (2004) as a design driver for the design of monopile, was expressed in terms of monopile thickness ($MT$) and monopile depth ($MD$) in the function $H_5$ and $H_6$, respectively. Malhotra (2011) specified the increase in tower height (here referred as hub height ($H_{HI}$) affects the design of foundation, therefore, its effect with monopile depth ($MD$) and monopile thickness ($MT$) was considered in the function $H_7$ and $H_8$, respectively. Bisoi and Haldar (2014) concluded that the change in diameter and thickness of both tower and monopile affected the soil stiffness, stating the importance of such parameters in monopile and tower design. The explanatory functions $H_9$-$H_{11}$ considered the possible influence of tower and monopile diameter and thickness. Finally,
the influence of rotor nacelle assembly mass was considered in the function $H_{12}$ as suggested by Segeren and Diepeveen (2014).

<table>
<thead>
<tr>
<th>Explanatory functions</th>
<th>Formula</th>
</tr>
</thead>
<tbody>
<tr>
<td>$H_1$</td>
<td>$\ln\left(\frac{H_w}{H_H}\right)$</td>
</tr>
<tr>
<td>$H_2$</td>
<td>$\ln(TI)$</td>
</tr>
<tr>
<td>$H_3$</td>
<td>$\ln\left(\frac{W_S}{C_S}\right)$</td>
</tr>
<tr>
<td>$H_4$</td>
<td>$\ln\left(\frac{RD}{H_H}\right)$</td>
</tr>
<tr>
<td>$H_5$</td>
<td>$\ln\left(\frac{M_T}{H_w}\right)$</td>
</tr>
<tr>
<td>$H_6$</td>
<td>$\ln\left(\frac{H_w}{M_D}\right)$</td>
</tr>
<tr>
<td>$H_7$</td>
<td>$\ln\left(\frac{M_D}{H_H}\right)$</td>
</tr>
<tr>
<td>$H_8$</td>
<td>$\ln\left(\frac{M_T}{M_D}\right)$</td>
</tr>
<tr>
<td>$H_9$</td>
<td>$\ln\left(\frac{T_{BD}}{M_D}\right)$</td>
</tr>
<tr>
<td>$H_{10}$</td>
<td>$\ln\left(\frac{T_{BT}}{M_T}\right)$</td>
</tr>
<tr>
<td>$H_{11}$</td>
<td>$\ln\left(\frac{T_{RD}}{M_T}\right)$</td>
</tr>
<tr>
<td>$H_{12}$</td>
<td>$\ln\left(\frac{M_R}{M_N}\right)$</td>
</tr>
</tbody>
</table>

The SMLR analysis was performed again to eliminate the insignificant explanatory functions for the finalized response model. The proposed model for tower top deflection, mudline flexure, mudline shear, and blade tip deflection can be expressed as shown in Eqs. (2.14) - (2.17), respectively.
\[
\ln(\tilde{y}_b) = -8.037 - 1.30\ln\left(\frac{H_w}{H_H}\right) + 0.22\ln\left(\frac{W_S}{C_S}\right) - 0.69\ln\left(\frac{M_T}{H_w}\right) \\
+ 0.74\ln\left(\frac{H_w}{M_D}\right) - 1.51\ln\left(\frac{T_{BD}}{M_D}\right) - 0.33\ln\left(\frac{T_{BT}}{M_T}\right)
\]

\[
\ln(\tilde{y}_m) = 7.65 + 2.23\ln\left(\frac{H_w}{H_H}\right) + 0.12\ln\left(\frac{W_S}{C_S}\right) - 2.40\ln\left(\frac{RD}{H_H}\right) \\
- 1.99\ln\left(\frac{H_w}{M_D}\right) + 1.85\ln\left(\frac{T_{BD}}{M_D}\right)
\]

\[
\ln(\tilde{y}_v) = 22.43 + 4.60\ln\left(\frac{H_w}{H_H}\right) - 0.08\ln\left(\frac{W_S}{C_S}\right) - 2.45\ln\left(\frac{RD}{H_H}\right) \\
+ 1.22\ln\left(\frac{M_T}{H_w}\right) - 2.88\ln\left(\frac{H_w}{M_D}\right) + 3.61\ln\left(\frac{T_{BD}}{M_D}\right)
\]

\[
\ln(\tilde{y}_{bd}) = 0.4439 - 0.51\ln\left(\frac{W_H}{H_H}\right) + 0.21\ln\left(\frac{W_S}{C_S}\right) + 0.72\ln\left(\frac{R_D}{H_H}\right) \\
+ 0.56\ln\left(\frac{H_w}{M_D}\right) - 0.49\ln\left(\frac{T_{BD}}{M_T}\right)
\]

2.6 Graphical Performance Comparison to FAST

The OWT responses observed from the SMLR models and the FAST simulations are plotted with respect to the significant input parameters. This analysis is helpful in identifying the trend of the responses, i.e., increase or decrease in the observed response for the given input parameter. For the analysis, the parameter of interest is varied while other parameters are maintained a median value in the given range.
Figure 2.2: Comparison between tower top deflections observed using SMLR model and FAST simulation with respect to a) hub height; b) monopole thickness; c) wind speed; and d) wave height.

Figure 2.2(a)-Figure 2.2(d) shows the tower top deflection response observed from the SMLR model and the FAST simulation for hub height, monopile thickness, wind speed, and wave height, respectively. The increase in hub height increased the tower top deflection of the OWT while the increase in monopile thickness reduced the tower top deflection observed. This can be observed because the increase in hub height increases the height of the tower and by cantilever action the response observed increases. On the other hand, increase in monopile thickness increased the strength of the structure thereby reducing the tower top deflection. The increase in wind speed and wave
height increase the tower top deflection of the structure, because of increase in OWT tower loads.

![Graphs showing comparison between mudline flexure observed using SMLR model and FAST simulation with respect to various parameters.](image)

Figure 2.3: Comparison between mudline flexure observed using SMLR model and FAST simulation with respect to a) hub height; b) monopile depth; c) wind speed; and d) wave height.

A similar representation is shown in Figure 2.3(a)-Figure 2.3(d) for the mudline flexure response observed for the hub height, monopile depth, wind speed, and wave height, respectively. Similar behavior is observed for the mudline flexure response for the hub height, wind speed, and wave height plots. The mudline flexure increases slightly with the increase in monopile depth. Similarly, the mudline shear response is observed in figure 4a-4d using the SMLR model and the FAST simulation for hub height, monopile thickness, wind speed, and wave height, respectively. There is some discrepancy
observed between the response from the SMLR and FAST simulation. This arises because of linear behavior of the input parameters considered in the model development. However, the FAST response followed the similar trend with the increase in input parameter as the SMLR response.

Figure 2.4: Comparison between mudline shear observed using SMLR model and FAST simulation with respect to a) hub height; b) monopole thickness; c) wind speed; and d) wave height.
Figure 2.5: Comparison between blade tip deflections observed using SMLR model and FAST simulation with respect to a) hub height; b) monopole thickness; c) wind speed; and d) wave height.

Figure 2.5 shows the blade tip deflection response and the FAST simulation. It can be observed that with an increase in input parameters, there is some level of variation in the observed response. The observed SMLR responses showed a similar trend as that of FAST simulation data point for the given input parameters. The effect of wind speed is however on the critical side as it is increased from 8.71 m at 3 m/s to 17.13 m at 75 m/s. Thus, it is evident that wind speed is responsible for higher blade tip displacement as expected.
2.7 Fragility Analysis

The fragility analysis of the OWTs is performed by developing wind-and-wave induced fragility curve to observe vulnerability associated with a given wind and wave loading. The fragility of a structure is defined as the conditional probability of a demand of the structure reaching or exceeding a predefined structural capacity. A Limit State (LS) function developed for each failure mode can be represented in terms of the capacity of structure pertaining to each mode of failure \( C_i \) and the demands \( D_i(x, w) \) due to the imposed load, such as material properties \( x \) and loading conditions \( w \). The LS function can be formulated as shown in Eq. (2.17).

\[
g(x,w) = C_i - D_i(x,w) \tag{2.17}
\]

The probability of component failure is defined such that \( g(x,w) > 0 \) represents the exceedance of limit state for each mode of failure. The exceedance probability, therefore, can be determined by integration of probability distribution of \( x \) and \( w \) over the failure region as shown in Eq. (2.18).

\[
P_f = P[g(x,w) < 0] = \int \ldots \int f_x(x,w)dx \tag{2.18}
\]

Monte Carlo Simulation (MCS) is used in determining the exceedance probability. To calculate the randomness inherent to an input function, it randomly runs the simulation by considering the probability density function of input variables. The MCS counts the number of conditions exceeding the limit state and then divides it with the number of simulation runs for each failure mode. Basically, 10,000 runs are performed for accuracy as outlined in numerous previous studies (Au et al. 2007, Choi et al. 2004, Seo and Linzell 2012).
The predictive fragility estimates for the representative OWT plotted as a function of wind speed and wave height, have been developed for deflection, moment, and shear mode of failure. The drift of 5% is considered as deformation capacity following Adhikari et al. (2014) is used to define the serviceability limit state. The limit on blade tip deflection is taken as 5% of the blade length following Young et al. (2010) and Hu et al. (2012). The bending moment capacity is computed using the expression in Eq. (2.19).

\[ C_m = f_y z \]  

(2.19)

where \( f_y \) is the yield bending capacity of the structure and \( z \) is the plastic section modulus. In developing the fragility curve, the OWT structure is made up of S355 steel having a design strength of 410 MPa and coefficient of variation of 5% (Karmazinova and Melcher, 2012). The shear capacity is defined as shown in Eq. (2.20).

\[ C_v = f_y A \left( \frac{3}{4} \right) \left( r_e^2 + r_i^2 \right) / (r_e^2 + r_o r_i + r_i^2) \]  

(2.20)

where \( A \) is the tower base area, \( r_e \) and \( r_i \) are tower external and internal diameter, respectively.

The fragility estimates are shown in the form of fragility surface, which is a three-dimensional representation of the exceedance probability as a function of two critical parameters wind speed, and wave height.

The predicted fore-aft tower top deflection fragility surface is presented as a function of wind speed and wave height in Figure 2.6. For the wind speed in the range of 3 to 75 m/s and wave height in the range of 1 to 20 m, the fragility surface for deflection showed progressive increment in fragility response to both wind and wave parameter. The exceedance probability increased from 0 to 0.15 when the wave height is increased
from 1 to 20 m, at a constant wind speed of 5 m/s. The increment is observed from 0.03 to 0.54 when wind speed is increased from 3 m/s to 75 m/s at a constant wave height of 2 m. This can be explained by the fact that the increase in wind loading causes higher deflection of the tower, while the wave loading does not directly contribute to the tower top deflection. It is noteworthy to observe that the increment in wind speed alone can increase the exceedance probability of the structure by more than 50%. With the combined extreme wind speed of 75 m/s and extreme wave height of 20 m, the exceedance probability reached 0.93.

![Figure 2.6: Probabilistic 3-D fore-aft tower top deflection fragility surface observed for the Offshore Wind Turbine subjected to change in wind speed and wave height parameter.](image)

The peak fore-aft bending moment obtained from the developed probabilistic model is utilized in developing fragility surface as shown in Figure 2.7. For the wind speed ranging from 5 m/s to 75 m/s and wave height ranging from 1 to 20 m, the exceedance probability increased from 0.0001 to 0.98. The flexural moment at the mudline has relatively higher exceedance probability with the variation in wave height.
than for deflection at the tower top. It can be explained by the fact that mudline region experiences additional wave load moment along with the moment due to aerodynamic loads causing relatively higher demands at values of wind and wave load. At a constant wind speed of 5 m/s, the exceedance probability observed is 0.01 at 2 m wave height, which increased to 0.66 at a wave height of 20 m. For a wave height of 2 m, the increase in wind speed from 5 m/s to 75 m/s increased the exceedance probability from 0.01 to 0.26. The increase in wind speed from 5 m/s to 75 m/s caused exceedance probability to increase from 0 to 0.26 at the wave height of 2 m, while the increment is from 0.66 to 0.98 at a 20 m wave height. The effect of wind and wave loads increased the exceedance probability at the mudline region of the OWT, the impact due to wave loads being relatively higher than due to wind loads. It can, therefore, be deduced that wave height plays an important role in estimating flexural failure at the mudline region.

![Figure 2.7: Probabilistic 3-D fore-aft bending moment fragility surface observed for the offshore wind turbine subjected to change in wind speed and wave height parameter.](image)
Figure 2.8 represents a 3-D fragility surface generated for the fore-aft shear forces at mudline with consideration of both wind speed and wave height. The shear fragility behavior is observed to the least critical failure mode. For the extreme wind speed of 75 m/s and extreme wave height of 20 m, the exceedance probability reached 0.16. The increase in wind speed from 5 m/s to 75 m/s at 20 m wave height increased the exceedance probability from 0.03 to 0.16, while the exceedance probability is almost 0 at a wind speed lower than 40 m/s for all values of wave height. Above 40 m/s of wave height, the fragility surface elevates with the increase in wind speed. It can, therefore, be marked that higher wind speed above 40 m/s caused an increase in the exceedance probability of fore-aft shear at the mudline region of the OWT. Also, the shear failure is highly dependent on the wind speed. Excluding wind speed from the probabilistic model could underestimate the wind-related shear failure of the OWT.

Figure 2.8: Probabilistic 3-D fore-aft shear force fragility surface observed for the offshore wind turbine subjected to change in wind speed and wave height parameter.
Figure 2.9 represents a 3-D fragility surface developed for the blade tip deflection with respect to both wind speed and wave height. The blade tip deflection fragility behavior is observed to be affected by the increase in wind speed only. The increase in wave height caused no significant increment in exceedance probability in comparison to the wind speed. The fragility surface increases linearly with the increase in wind speed, reaching an exceedance probability of 0.99 at a wind speed of 75 m/s and wave height of 20 m. Therefore, a significant research should be performed to reduce such deflection while installing OWT in wind-related hazard site.

2.8 Effect of Design Parameters under Wind Speed and Wave Height Variation

To explore the effect of each significant parameter on the exceedance probability estimation, a fragility curve is developed with respect to loading parameters i.e., wind speed and wave height. Fragility analysis is performed for both upper and lower range of significant input parameters to estimate the exceedance probability in terms of wind
speed/wave height. For the fragility curve, the considered input parameter was taken as a constant value (either the upper or lower range) while the other parameters were fixed.

The parameter effects are here analyzed at median exceedance probability to estimate the value of wind speed and wave height causing 50% exceedance probability. In this study, deflection and flexure failure are only considered as shear failure yielded in exceedance probability value of less than 0.5. Also, the blade tip deflection has not been studied in detail, as it is insignificant with the increase or decrease in the given structural parameters and has only shown its effect with the wind speed increment.

Figure 2.10: Effect of significant input parameters on monopile OWT fragility at median wind speed value for flexural and deflection failure.

The effects of each significant input parameter are expressed in terms of median wind speed for deflection and flexure failure in Figure 2.10. The tower base diameter is a critical input parameter for the flexural response, as the median failure is observed at 17 m/s and 47 m/s when the hub height is 5.6 m and 7.6 m, respectively. Similarly, for the deflection exceedance probability, the hub height experiences 50% exceeding probability at a median wind speed of 51 m/s and 44 m/s for hub heights of 80 m and 140 m,
respectively. The increment in monopile depth and tower base diameter is observed to increase the mean wind speed resembling the reduction of exceedance probability with increased dimension. The current speed for both higher and lower range experienced the median exceedance probability beyond the considered range of wind speed for deflection, resulting in being the least critical for deflection failure. Since the rotor diameter is not a significant parameter for deflection in the developed model in Eq. (2.14), its effect is not seen for deflection. Similar behavior is observed for the tower base thickness and monopile thickness, as it is not considered to be significant for the flexural failure. From the above figure, tower base diameter is observed to be critical for both deflection and flexural response experiencing median exceedance probability at relatively lower wind speed than other parameters.

![Diagram](image)

Figure 2.11: Effect of tower base diameter on monopile OWT fragility with the increase in wind speed: a) flexure and b) deflection.

To observe the fragility behavior with wind speed variation for different tower base diameter values, the exceedance probability of upper and lower value of tower base diameter considered as 5.6 m and 7.6 m is observed with an increase in wind speed in
Figure 2.11. The curve indicates that flexural exceedance probability for 5.6 m tower base diameter increases significantly with the increase in wind speed until it reaches failure of 1.0 at 50 m/s. In Figure 10(a), the flexural exceedance probability for 7.6 m tower base diameter rises smoothly with an increase in wind speed and reaches peak exceedance of 0.78 at extreme 75 m/s wind speed. The percent difference for median exceedance probability is nearly 63%. The deflection fragility curve for 5.6 m and 7.6 m tower base diameter is shown in Figure 10(b). The curve represents smooth exceedance increment with wind speed for both hub height values. The peak exceedance probability at 75 m/s for 5.6 m and 7.6 m tower base diameter is observed as 0.99 and 0.85, respectively. The percent difference of 37% was observed for median exceedance probability.

Similar bar plot was developed for the median wave height value for each of the significant input parameters as shown in Figure 2.12. It can be observed that the tower base diameter reaches the median deflection exceedance probability at 14 m and 20 m for the tower base diameter of 5.6 m and 7.6 m, respectively. Similarly, the hub height is observed as the critical parameter for flexure. The flexural median exceedance probability is recorded at 12 m and 8 m for 80 m and 120 m hub height, respectively. The parameters such as monopile depth, hub height, current speed, and tower base thickness are observed to be ineffective within the given range of wave height for deflection with median exceedance observed beyond 20 m wave height. Also, the rotor diameter is ineffective in the developed model for the deflection failure and the tower base thickness for the flexural failure, their effects are not observed in the bar chart. From the bar chart, it is assured that the tower base diameter and hub height is the critical component for the
deflection and flexure failure, respectively, as the median exceedance probability observed at a lower wave height.

Figure 2.12: Effect of significant input parameters on monopile OWT fragility at median wave height value for flexural and deflection failure.

To explore the fragility behavior of tower base diameter on deflection failure and hub height on flexural failure, the fragility curve is developed in Figure 2.13. Figure 2.13(a) shows the flexure fragility curve with an increase in wave height for 80 m and 140 m hub height. The fragility curve accelerates rapidly above 6 m wave height for 140 m hub height and above 8 m for 80 m hub height. There is a uniform difference between the fragility curve for the given tower base diameter, with the difference of nearly 33% at median wave height. Similarly, the deflection fragility curve is shown in Figure 2.13(b) for the 5.6 m and 7.6 m tower base diameter. It is observed that fragility curve reaches maximum exceedance probability at 20 m wave height with values of 0.72 and 0.5 for 5.6
m and 7.6 m tower base diameter, respectively. The difference is uniform for both the range with the observed difference of 30% at median wave height.

From the above discussion, it can be concluded that the tower base diameter and the hub height are observed as the critical parameters among other significant parameters while considering wind speed and wave height as control loading parameters, respectively. Also, the wind speed can be considered as a more critical loading parameter as most of the median exceedance probabilities are observed within the given range of wind speed, but for the wave height, most of the input parameters reached median exceedance probability beyond the given range. Both the bar plots and fragility curve resembled the flexural mudline failure as the critical failure mode since the median exceedance probability is reached at a relatively lower wind speed and wave height value. The existing wind turbine specification is observed to experience median exceedance probability at lower wind and wave height for the flexure, which suggests modification in structural parameters for better performance in multi-hazard wind and wave loads.
2.9 Conclusions and Future Work

This chapter developed statistical regression models for deflection, moment, and shear demands of 5 MW Offshore Wind Turbines (OWTs). To develop the models, Latin Hypercube Sampling (LHS) technique were employed to develop the configurations for the simulations. A total of 120 configurations with 20 input parameters was generated from the sampling technique for the simulation. To determine the most significant parameters for each failure mode, Stepwise Multiple Linear Regression (SMLR) approach was used along with some model validation techniques. Later, explanatory functions were defined using only the identified significant parameters for the probabilistic models. Finally, demand models were developed with the significant explanatory functions.

The developed demand models for deformation, flexure, and shear were further analyzed to generate fragility estimates as a function of wind speed and wave height. The performance of OWT tower for deflection limit and yield limit state was investigated under the combined action of wind speed and wave height. The failure surface for both fore-aft tower-tip displacement and fore-aft mudline overturning bending moment showed significant exceedance probability with incremental wind speed and wave height, while the failure surface for fore-aft shear at the mudline only showed an increase in exceedance probability for wave height greater than 15 m. The wave height of lower than 15 m had no effect on the shear failure surface. For an extreme wind speed of 75 m/s and extreme significant wave height of 30 m, the exceedance probability for tower top deflection, flexure, shear, and blade tip deflection were 0.96, 0.99, 0.35, and 0.99, respectively. The overturning flexural moment was found to be the most critical amongst
other at a given wind speed and wave height. The blade tip deflection is highly sensitive to the wind speed and insensitive to the wave parameter, as expected.

The effect of significant input parameters on OWT fragility was examined for both tower top deflection and flexure failure. Under the increasing wind speed and wave height, the fragility increment was observed for those parameters to identify the critical parameter that affected the exceedance probability. The tower base diameter and hub height played an important role in the fragility behavior when observed with respect to wind speed and wave height variation, respectively. It was observed that increase in the hub height and decrease in the tower base diameter resulted in an increase in the exceedance probability. It can be concluded that existing wind turbines specifications could be modified for structural stability at multi-hazard scenarios.

The proposed model only included a linear effect of the significant parameters. However, a complex structure requires a more advanced approach which considers the interaction between the parameters. Additional analysis of the offshore environment involving ice loads and earthquake loads are needed. Further, a linear irregular wave was considered in the wave phenomena, but the offshore waves are nonlinear breaking waves, which need further research.
REFERENCES


3 CHAPTER 3 SURROGATE MODELING FOR MULTI-HAzaRd
VULNERABILITY ASSESSMENT OF OFFSHORE WIND TURBINES

3.1 Abstract

This chapter deals with surrogate modeling-based multi-hazard vulnerability analysis of monopile offshore wind turbines (OWTs) subjected to stochastic wind and wave loadings. The 5-MW OWT model developed by National Renewable Energy Laboratory (NREL) was used as the baseline model. To perform vulnerability analysis under a wide range of structural parameters, physical configurations of the OWT such as hub height, monopile thickness, rotor diameter, etc. were varied based on available manufacturers’ specifications of typical 5MW OWTs. Two separate surrogate models were then developed using Response Surface Metamodels (RSMs) and Stepwise Multiple Linear Regression (SMLR) approaches. Multiple aero- and hydro-dynamic simulations under wind and wave loadings was performed via Fatigue, Aerodynamics, Structures, and Turbulence (FAST) developed by NREL to determine the peak tower top deflection, mudline flexure, mudline shear, and blade tip deflection response of the OWTs using 20 structural and loading parameters. Based on the simulation results, screening analysis was performed to identify the significant input parameters affecting the response. For the RSM approach, Pareto plots were used to identify the significant parameters. A Central Composite Design (CCD) mechanism was then used to develop the simulation matrix for modeling. Similarly, a stepwise elimination technique was used for identifying the significant parameters in the SMLR approach while Latin Hypercube Sampling (LHS) was employed for simulation matrix development. Statistical and graphical analysis of
the developed models revealed that the RSM approach provided a more accurate prediction of the response with less computational effort. Further fragility curves were developed using the two surrogate models and the FAST simulation data to perform vulnerability analysis. The fragility curves developed using the SMLR approach resulted in a conservative exceedance probability curve. This was because the SMLR model considered only the linear effect of the input parameters. The RSM model on the other hand modeled the nonlinearities between the input parameters, thus resulting in a less critical exceedance probability curve and provided more resemblance to actual FAST simulation’s result. Hence, the RSM approach was adopted to develop the fragility surface and estimate the exceedance probability under the identified structural and loading parameter variations. Results showed that mudline flexural failure was the most critical mode of failure for monopile OWTs. The wind speed was observed to be the most critical loading parameter for vulnerability estimation especially for the deflections, while the effect of the wave height was significantly higher in mudline flexural failure. Further, hub height, rotor diameter, and monopile thickness were observed to be critical structural parameters affecting the exceedance probability. It can, therefore, be concluded that modification of critical parameters depending on the site considered could improve the structural performance of the monopile OWTs in hazardous loading scenarios.

3.2 Introduction

With the world progressing in the sector of renewable energy, wind turbines are gaining widespread popularity. The availability of large areas and higher wind speeds across the shore favors the installment of Offshore Wind Turbines (OWTs). However, the OWTs are subjected to various dynamic forces due to the wind, wave, and current. The
operation and maintenance cost of wind turbines is thus still on the higher side compared to traditional energy sources. Carroll et al. (2016) reported approximately 10 failures per turbine each year existed in wind turbine farms which shows that frequent conditional monitoring is needed. One of the solutions to reduce the frequency at which the turbines need to be assessed for maintenance is to improve system reliability (Echavarria 2009). An accurate assessment of the system reliability can be performed using structural reliability methodology, which has been commonly applied since the 1980’s in assessing the safety of OWTs (Madsen 1987, Melchers and Beck 2017, Sorense and Faber 2002).

In recent years, numerous research has been performed for performance assessment of the wind turbine properties using computational tools (Kallehave et al. 2015, Lozano-Minguez et al. 2011, Bazilevs et al. 2011, Ashuri et al. 2016, Yeter et al. 2017). In order to estimate the performance an OWT structure, it is necessary to determine the extreme wind and wave loads which cause structural failure. To that end, many studies relating to reliability assessment has been performed by the usage of complex aero-elastic simulation tools, (Yeter et al. 2017, Mardfekri and Gardoni 2013, Karadeniz et al. 2010) mainly Fatigue, Aerodynamics, Structures, and Turbulence (FAST) simulation tool developed by NREL (Jonkman and Buhl 2005). Fragility analysis is a common approach to visualize the structural vulnerability under such extreme loading conditions. It has been applied to many engineering structures such as bridges (Seo and Park 2017, Kameshwar and Padgett 2014, Seo et al. 2012, Seo and Linzell 2010, Nielson and DesRoches 2007, Choi et al. 2004); buildings (Kirçil and Polat 2006, Rota et al. 2010); and wind turbines (Quilligan et al. 2012, Kim et al. 2014, Mardfekri and Gardoni 2013).
Fragility analysis provides a conditional probability of the likelihood that the structure, or one of its components, will reach or exceed its designed limit state. There exist many kinds of literature regarding the fragility analysis of OWTs (Quilligan et al. 2012, Mardfekri et al. 2013, Kim et al. 2014) whose failure behavior has shown to be dependent on the external loadings. Quilligan et al. (2012) applied a probabilistic approach to compare the relative performance of steel and concrete wind turbine towers having different tower heights under wind speed variation. The fragility curves in terms of tip displacement were developed showing their performance. The results concluded that increase in turbulence level resulted in an increase in maximum tip displacement. However, increasing tower height only caused a minimal increase in the exceedance probability of wind turbines. Mardfekri and Gardoni (2013) performed structural reliability analysis of monopile OWTs using the probabilistic demand models and developed fragility curves utilizing predictive response from the developed model along with Monte Carlo Simulation (MCS). The developed probabilistic demand models were based on results from the FAST simulation and were validated with finite element simulation. The study concluded that the bending mode controlled the fragility behavior of the wind turbine. The wind speed showed the negligible effect on the shear failure mode and the change in wave height did not affect the exceedance probability at higher wind speeds.

The fragility curve developed using surrogate modeling techniques such as Response Surface Metamodels (RSM) has shown many applications to structural reliability estimation (Deng and Cai 2009, Seo and Linzell 2012, Soares et al. 2002, Wong et al. 2005, Gavin and Yau 2008). Surrogate models have a number of distinct
advantages such as the ability to account for the uncertainty within the considered parameters (Youn and Choi 2004, Taflanidis and Cheung 2012, Yang et al. 2015), replace complicated computational simulation models (Maki et al. 2012, Jia and Taflanidis 2013), reduce computational time (Cheng and Si 2008, Kim and Lee 2015), and accuracy in predicted response (Bacharoudis and Philippidis 2013, Toft et al. 2016, Hussan et al. 2017).

This chapter compares two surrogate models to evaluate their efficiency and accuracy for improved multi-hazard vulnerability analysis in terms of fragility surface. The information relating to fragility behavior is helpful for future design of OWTs and in reducing the operational and maintenance cost of OWTs. The chapter is divided into six sections. The second section discusses the nonlinear aero-hydrodynamic analysis in FAST. The third section explains experimental design procedure for developing the RSM functions. The fourth section details the developed surrogate model using RSM and SMLR. The fifth section provides a detail on fragility estimation for vulnerability analysis. The sixth section performs a statistical and graphical comparison between the surrogate models and FAST simulation data. The seventh section develops a fragility surface using RSM function and determines the effect of structural parameters on multi-hazard loading. The eighth section provides conclusions and future work.

3.3 FAST Modeling and Simulation

The FAST code is a nonlinear time-domain simulator which employs multi-body dynamics to perform complex simulations incorporating coupled wind and wave loads. The schematic representation of OWT with loads associated with the offshore environment is shown in Figure 3.1. The FAST environment consists of several simulation interfaces.
allowing dynamic interaction between structural and environmental properties. The following subsections provide an overview of the multi-hazard loadings in FAST.

Figure 3.1: Schematic of Offshore Wind turbine with structural properties and environmental parameters.

3.3.1 Aero-hydro Dynamic Simulation

A Kaimal power spectrum and exponent coherence spectrum was used to define the turbulent wind condition over the rotor plane, using TurbSim (a stochastic, full-field, and turbulent wind generating simulator which is a part of the FAST package) (Jonkman 2009). The turbulent wind condition is defined by the International Electro-technical Commissions (IEC) Kaimal spectral model as defined in IEC 61400-3 (International Electrotechnical Commission, 2009). The Kaimal spectrum is dependent on the wind
speed at hub-height ($V_{hub}$) along with the wind direction and can be expressed as shown in Eq (3.1).

$$\frac{f_v S_k (f_v)}{\sigma_k^2} = \frac{4 f_v L_k}{V_{hub}} \left(1 + 6 f_v L_k V_{hub}\right)^{\frac{5}{3}}$$

(3.1)

where $k$ is an index referring to the direction of the wind, $f_v$ is the frequency in Hertz, $S_k$ is the single-sided velocity component spectrum, $\sigma_k$ is the standard deviation of the wind component, and $L_k$ is the integral scale parameter for wind component. For the coherence spectrum to account for spatial correlation of the longitudinal velocity component, IEC wind coherence follows two-dimensional Taylor Frozen theory in the longitudinal direction as shown in Eq (3.2).

$$Coh (r, f) = \exp \left\{ -12 \sqrt{\left(\frac{f_v}{V_{hub}}\right)^2 + \left(\frac{0.12r}{L_{sc}}\right)^2} \right\}$$

(3.2)

where $r$ is the magnitude of separation vector, which is normal to the average wind direction, $L_{sc}$ is the coherence scale parameter. The wind force generated by the wind turbine acting parallel to the direction of wind flow can be represented as shown in Eq (3.3).

$$F(t) = \frac{1}{2} \rho u^2 C_s A$$

(3.3)

where $\rho$ is the mass density of air, $u$ is the mean wind speed at hub-height, $C_s$ is the shape coefficient of circular section which depends on Reynolds number and is set to 0.5 for circular sections, and $A$ is the projected area of the tower facing the incoming wind.

The response from the TurbSim along with aerodynamic drag force in AeroDyn (Moriarty and Hansen 2005), and structural dynamics in ElastoDyn (Jonkman 2013) were
utilized in determining time-domain stochastic OWT aerodynamic response. AeroDyn is a FAST utility that simulates aerodynamic forces on the rotor blades whereas, ElastoDyn simulates the wind forces on the tower.

The hydrodynamic loading on the monopile generated in HydroDyn module (Jonkman et al. 2014) was performed following irregular long-crested wave for the propagation along with Morison’s equation. The Morison equation determines the hydrodynamic forces following the DNV specification for the design of OWT. The horizontal force acting on the small section of the monopile, \(dz\), at any structural depth of \(z\) can be written as shown in Eq (3.4).

\[
dF(z, t) = C_M \rho \frac{\pi}{4} d_p^2 \ddot{u}(z, t) dz + C_D d_p \frac{1}{2} \rho u(z, t) |u(z, t)| dz
\]

where \(C_M\) and \(C_D\) are inertia coefficient and drag coefficient, respectively; \(d_p\) is the monopile diameter, \(\rho\) is the density of water. The bending moment on the structure at depth, \(z\), can be determined by integrating the force as expressed in Eq (3.5).

\[
M(t) = \int_0^d z dF(z, t)
\]  

The hydrodynamic loading due to water waves for modeling the monopile assumes the strip-theory approach (Song et al. 2012). The incident wave kinematic model for linear irregular waves follows the Joint North Sea Wave Project (JONSWAP) spectrum. The JONSWAP spectrum for wave loading on a monopile follows the form as shown in Eq (3.6).
\[ S_{\eta\eta}(f) = 0.3125 \, H_s^2 \, T_p \left( \frac{f}{f_p} \right)^{-5} \exp \left( -1.25 \left( \frac{f}{f_p} \right)^{-4} \right) (1 - 0.287 \ln v) \exp \left[ -0.5 \left( \frac{f}{f_p \sigma} \right)^2 \right] \] (3.6)

where \( H_s \) is the significant wave height, \( T_p \) is the peak spectral period, \( v \) is the peak shape parameter (generally taken as 3.3), \( f \) is the frequency, \( f_p \) is the peak frequency \( \left( = \frac{1}{f_p} \right) \), and \( \sigma \) is 0.07 for \( f \leq f_p \) and 0.09 for \( f > f_p \). The velocity potential of water particles adopts the Laplace equation to simulate the stochastic ocean waves as shown in in Eq (3.7).

\[ \phi(x, z, t) = \frac{g H_s}{2 \omega} \frac{\cosh k(d + z)}{\cosh kd} \sin(kx - \omega t) \] (3.7)

where \( x \) and \( z \) are the horizontal and vertical directions, respectively; \( \omega \) is the angular sea wave frequency \( \left( = g k \tan(kh) \right) \); \( k \) is the sea wave number \( \left( = \frac{2\pi}{L} \right) \); \( L \) is the wave length, \( d \) is the depth of water. On differentiating the velocity potential, the velocity and acceleration of water particles were determined. The origin of \( z \) axis is selected at the Mean Sea Level (MSL).

For the current loads, the current velocity follows the power law for the distribution of current along the depth of the water as shown in Eq (3.8).

\[ U_{ss}(z) = U_{0ss} \left( \frac{z + d}{d} \right)^{1/7} \] (3.8)

where \( z \) is the depth considered below MSL (negative downward), \( d \) is the depth of water, and \( U_{0ss} \) is the current velocity at MSL. The current force can be calculated using Morison’s equation as shown in Eq (3.4).
The dynamics of the monopile are evaluated in SubDyn (a software utility part of the FAST code which performs the dynamic simulation for monopile sub-structures) (Damiani et al. 2015) which considers that the monopile foundation is clamped to the mudline and is rigidly connected to the tower. For the simulations, a range of input parameters is selected based on the existing literature and publicly available manufacturers’ specifications of 5 MW OWTs as shown in Table 3.1. The input parameters are subdivided into structural parameters and external loading parameters.

Table 3.1. Monopile OWT input parameters

<table>
<thead>
<tr>
<th>Category</th>
<th>Parameters</th>
<th>Variable Level</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Structural Parameters</strong></td>
<td>Hub Height ($H_H$)</td>
<td>80 140</td>
<td>m</td>
</tr>
<tr>
<td></td>
<td>Monopile depth ($M_D$)</td>
<td>10 50</td>
<td>m</td>
</tr>
<tr>
<td></td>
<td>Rotor diameter ($R_D$)</td>
<td>115 151</td>
<td>m</td>
</tr>
<tr>
<td></td>
<td>Tower base diameter ($T_{BD}$)</td>
<td>5.6 7.6</td>
<td>m</td>
</tr>
<tr>
<td></td>
<td>Tower top diameter ($T_{TD}$)</td>
<td>3.8 5</td>
<td>m</td>
</tr>
<tr>
<td></td>
<td>Tower base thickness ($T_{bt}$)</td>
<td>0.027 0.068</td>
<td>m</td>
</tr>
<tr>
<td></td>
<td>Monopile thickness ($M_t$)</td>
<td>0.068 0.15</td>
<td>m</td>
</tr>
<tr>
<td></td>
<td>Tower top thickness ($T_{tt}$)</td>
<td>0.019 0.02</td>
<td>m</td>
</tr>
<tr>
<td></td>
<td>Rotor mass ($R_M$)</td>
<td>110 120</td>
<td>tons</td>
</tr>
<tr>
<td></td>
<td>Nacelle mass ($N_M$)</td>
<td>240 290</td>
<td>tons</td>
</tr>
<tr>
<td></td>
<td>Damping ratio ($DR$)</td>
<td>0.01 0.02</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Youngs modulus ($E_S$)</td>
<td>190 210</td>
<td>kg/m$^3$</td>
</tr>
<tr>
<td></td>
<td>Shear modulus ($V_S$)</td>
<td>73 78</td>
<td>kg/m$^3$</td>
</tr>
<tr>
<td></td>
<td>Density ($D_S$)</td>
<td>8100 8600</td>
<td>kg/m$^3$</td>
</tr>
<tr>
<td><strong>Loading Parameters</strong></td>
<td>Wind Speed ($W_S$)</td>
<td>3 75</td>
<td>m/s</td>
</tr>
<tr>
<td></td>
<td>Turbulence intensity ($TI$)</td>
<td>0 0.16</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>Wave Height ($W_H$)</td>
<td>1 20</td>
<td>m</td>
</tr>
<tr>
<td></td>
<td>Peak spectral period ($T_P$)</td>
<td>1 20</td>
<td>s</td>
</tr>
<tr>
<td></td>
<td>Water Depth ($D$)</td>
<td>20 30</td>
<td>m</td>
</tr>
<tr>
<td></td>
<td>Current Speed ($W_C$)</td>
<td>0.03 2.5</td>
<td>m/s</td>
</tr>
</tbody>
</table>

For each simulation with a change in structural properties such as tower dimensions, it is necessary to recalculate the mode shapes due to the dynamic loadings.
which require linearizing the OWT structure. The linearization of the OWT model is performed by utilizing a linearization feature in FAST and BModes (a finite-element code resulting in dynamically coupled modes for the wind turbine mode) (Bir 2005).

3.3.2 Results

The aero-hydrodynamic simulation performed in FAST results in responses over the simulation time at different sections of the OWT structure. The OWT responses of interest in the chapter are the peak deflection at the tower top, the mudline shear, the mudline shear force, and the blade tip deflection of the structure. For the unidirectional wind and wave load acting on the structure, the fore-aft direction in FAST (the direction along the applied load) has the highest value of deflection, bending moment, and shear force (Bush and Manuel 2009, Shi et al. 2015). The peak response observed from FAST were used in the development of the surrogate models.

3.4 Experimental Design

Response Surface Metamodels (RSM) are second-order polynomial functions representing the functional relationship between the input parameters and the observed response of the OWT structure. The statistical model is responsible for efficiently predicting the response using a set of input parameters. The RSM approach can be generalized in two steps: 1) generating a simulation matrix using experimental design techniques, and 2) fitting the model to represent the observed responses (Seo and Linzell 2012). Central Composite Design (CCD) is one of the popular experimental design technique for RSM development (Seo and Linzell 2013). The CCD technique is considered an extension of the 2k factorial design, where k is the number of input
parameters. The CCD method establishes \(2k+2k+1\) experiment using three levels for each input parameters. CCD was applied in this study as it provides a good agreement between the predictive accuracy and the computational effort (Park and Towashiraporan 2014, Seo and Linzell 2012). After performing a set of nonlinear analysis based on CCD, the RSM model was fitted according to the observed responses obtained from FAST to develop an accurate estimate of the actual behavior. The RSM model can be expressed as shown in Eq (3.9).

\[
\hat{y} = \beta_0 + \sum_{i=1}^{k} \beta_i x_i + \sum_{i=1}^{k} \beta_{ii} x_i^2 + \sum_{i=1}^{k-1} \sum_{j=i+1}^{k} \beta_{ij} x_i x_j + \epsilon
\] (3.9)

where \(\hat{y}\) is the predicted response of the OWT structure (e.g., deflection), \(x_i\) and \(x_j\) are the input parameters (e.g., wind speed, hub height), \(\beta_0, \beta_{ii}, \beta_{ij}\) are regression coefficients determined from surrogate modeling, \(k\) is the number of input parameters and \(\epsilon\) is the random error. The coefficients \(\beta_0, \beta_{ii}, \beta_{ij}\) are calculated using the least square regression technique to fit the response surface approximation to the observed responses.

The critical input parameters which have a significant effect on the observed response have to be identified first through a screening process. The screen analysis was performed in a statistical software called JMP (SAS Institute, 2000). The identified significant parameters were then used to develop an experimental design matrix using CCD.

3.4.1 Screening Analysis

The initial set of simulation matrix was generated using the set of 20 input parameters defined in Table 3.1. The screening analysis identifies the most significant
parameters among this set. The simulation matrix was generated using two-level main effect screening design technique, which uses the maximum and minimum values of each of the 20 input parameters. The screening method was used to establish 30 OWT simulation models, each with varying range of input parameters and was simulated using FAST.

The screening of the significant input parameters was accomplished by employing least square regression of the observed responses. The results from the screening were visualized by generating a Pareto plot to observe the rank of input parameters affecting the response. The Pareto plot lists the most significant parameters of the model. The Pareto plot was developed for deflection, flexure, and shear force responses and the significant parameters for each response was identified. The representative Pareto plot for the peak deflection at tower top is shown in Figure 3.2. The bar plot represents the individual contributions of each input parameter to the predicted response. The dashed curve in the figure represents a cumulative contribution to the overall response, while the vertical dotted line represents the cumulative probability corresponding to each input parameter.
Table 3.2: Significant monopile OWT input parameter for the observed response

<table>
<thead>
<tr>
<th>Observed Response</th>
<th>H_H</th>
<th>W_S</th>
<th>R_D</th>
<th>T_br</th>
<th>W_C</th>
<th>W_H</th>
<th>D</th>
<th>M_t</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tower Top Deflection</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

The significant input parameters observed from the screening analysis for tower top deflection, mudline flexure, mudline shear, and blade tip deflection response is summarized in Table 3.2. The significant parameters: hub height (H_H), rotor diameter (R_D), water depth (D), tower base thickness (T_br), and monopile thickness (M_t), were identified as the significant input parameters relating to structural properties. Similarly, for the loading parameters, wind speed (W_S), wave height (W_H), and current velocity (C_S) were the significant parameters.
### Mudline Bending Moment

- $M_t$
- $H_H$
- $D$
- $W_H$
- $C_S$
- $T_{bt}$
- $W_S$
- $R_D$

### Mudline Shear Force

- $M_t$
- $H_H$
- $C_S$
- $D$
- $T_{bt}$
- $W_H$
- $R_D$
- $W_S$

### Blade Tip Deflection

- $W_S$
- $R_D$
- $H_H$
- $T_{bt}$
- $M_t$
- $C_S$
- $W_H$
- $D$

#### 3.4.2 Simulation Matrix

The simulation matrix used for the development of the RSM model, shown in Table 3.3, illustrates 82 different combination patterns developed using the CCD mechanism. The values in the table represent a value in the selected range for each significant input parameter. In the combination, a ‘-1’ represents the lowest value, a ‘+1’ represents the highest value, and ‘0’ represents the midpoint of the range considered in Table 1. For instance, in combination 1, $W_S = 1$, $C_S = -1$, $W_H = 1$, $H_H = -1$, $D = 1$, $T_{bt} = 1$, $M_t = -1$, and $R_D = 1$, resembles a OWT model with maximum wind speed of 75 m/s, current speed of 0.03 m/s, water depth of 30m, monopile thickness of 0.068 m, and rotor diameter of 151 m. Based on the given combinations, the peak responses of the OWT model are computed from the FAST simulation. The table also shows the peak simulation response as observed from FAST as well as the predicted response observed from the RSM model which is described in the following section.
### Table 3.3. Central Composite Design table for observed and predicted response of OWT

<table>
<thead>
<tr>
<th>Cm.</th>
<th>C1</th>
<th>Ws</th>
<th>Cs</th>
<th>Wm</th>
<th>Ht</th>
<th>D</th>
<th>Tm</th>
<th>Ml</th>
<th>Rd</th>
<th>$y_{b,o}$</th>
<th>$y_{b,p}$</th>
<th>$y_{m,o}$</th>
<th>$y_{m,p}$</th>
<th>$y_{v,o}$</th>
<th>$y_{v,p}$</th>
<th>$y_{b8,o}$</th>
<th>$y_{b8,p}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>8.17</td>
<td>6.04</td>
<td>1.48</td>
<td>3.63</td>
<td>25.24</td>
<td>23.50</td>
<td>19.76</td>
<td>20.46</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>17.05</td>
<td>17.01</td>
<td>1.31</td>
<td>0.53</td>
<td>16.19</td>
<td>10.00</td>
<td>19.4</td>
<td>20.45</td>
</tr>
<tr>
<td>3</td>
<td>1</td>
<td>1</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>-1</td>
<td>2.26</td>
<td>1.43</td>
<td>0.82</td>
<td>0.58</td>
<td>11.44</td>
<td>6.83</td>
<td>15.7</td>
<td>15.57</td>
</tr>
<tr>
<td>4</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>3.53</td>
<td>5.37</td>
<td>0.69</td>
<td>1.40</td>
<td>21.00</td>
<td>37.01</td>
<td>7.17</td>
<td>11.63</td>
</tr>
<tr>
<td>5</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>-1</td>
<td>8.07</td>
<td>7.54</td>
<td>1.43</td>
<td>1.32</td>
<td>14.90</td>
<td>19.67</td>
<td>16.38</td>
<td>15.26</td>
</tr>
<tr>
<td>6</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>3.78</td>
<td>3.55</td>
<td>0.68</td>
<td>0.46</td>
<td>7.32</td>
<td>9.31</td>
<td>16.18</td>
<td>14.59</td>
</tr>
<tr>
<td>7</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>1</td>
<td>0.11</td>
<td>1.17</td>
<td>0.02</td>
<td>0.14</td>
<td>0.99</td>
<td>7.03</td>
<td>1.8</td>
<td>2.51</td>
</tr>
<tr>
<td>8</td>
<td>1</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>-1</td>
<td>7.82</td>
<td>9.68</td>
<td>1.43</td>
<td>0.58</td>
<td>14.81</td>
<td>6.38</td>
<td>16.39</td>
<td>14.80</td>
</tr>
<tr>
<td>75</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>3.20</td>
<td>4.42</td>
<td>9.46</td>
<td>2.57</td>
<td>2.90</td>
<td>4.32</td>
<td>13.6</td>
<td>14.69</td>
</tr>
<tr>
<td>76</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>13.64</td>
<td>13.42</td>
<td>1.14</td>
<td>0.78</td>
<td>9.50</td>
<td>6.87</td>
<td>16.51</td>
<td>16.91</td>
</tr>
<tr>
<td>77</td>
<td>-1</td>
<td>-1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>0.39</td>
<td>1.09</td>
<td>0.05</td>
<td>0.07</td>
<td>0.76</td>
<td>2.86</td>
<td>4.84</td>
<td>7.86</td>
<td></td>
</tr>
<tr>
<td>78</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>1</td>
<td>0.39</td>
<td>2.10</td>
<td>0.05</td>
<td>0.03</td>
<td>0.71</td>
<td>1.98</td>
<td>4.84</td>
<td>5.05</td>
<td></td>
</tr>
<tr>
<td>79</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>0.34</td>
<td>1.49</td>
<td>0.04</td>
<td>0.02</td>
<td>0.96</td>
<td>0.33</td>
<td>1.84</td>
<td>2.74</td>
<td></td>
</tr>
<tr>
<td>80</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>-1</td>
<td>1</td>
<td>1</td>
<td>0.10</td>
<td>1.07</td>
<td>0.04</td>
<td>0.14</td>
<td>1.03</td>
<td>4.25</td>
<td>4.95</td>
<td>3.85</td>
<td></td>
</tr>
<tr>
<td>81</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>1</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>4.10</td>
<td>3.57</td>
<td>0.72</td>
<td>0.62</td>
<td>8.38</td>
<td>7.52</td>
<td>21.19</td>
<td>20.90</td>
<td></td>
</tr>
<tr>
<td>82</td>
<td>-1</td>
<td>-1</td>
<td>1</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>-1</td>
<td>3.89</td>
<td>3.47</td>
<td>0.64</td>
<td>4.55</td>
<td>17.73</td>
<td>19.52</td>
<td>2.21</td>
<td>3.64</td>
<td></td>
</tr>
</tbody>
</table>
3.5 Surrogate Models

This section describes the surrogate models developed for modeling the peak OWT response. The RSM models are developed from the experimental design dataset as shown in Table 3.3. The Stepwise Multiple Linear Regression (SMLR) models that were developed in Chapter 2 are also listed here for the subsequent comparison with FAST simulation, which is compared in section six.

3.5.1 Response Surface Methodology

The RSM functions were developed using the eight significant input parameters identified through a three-level CCD in Section 3.4.1. The CCD resulted in 82 different experimental simulations of the OWT that needed to be performed. Nonlinear aerodynamic simulations were carried out for each experimental dataset in FAST. The observed response was then fitted using least squares regression technique. The resulting RSM functions for the tower top deflection, mudline flexure, mudline shear, and blade tip deflection with the interaction of significant parameters are shown in Eq (3.11) -(3.10), respectively.
\[
\hat{y}_B = 3.58 + 2.75W_S + 0.03C_S + 1.15W_H + 2.81H_H - 0.27D + 0.04T_{bt} \\
- 1.22M_t + 0.80R_D - 0.001W_SC_S - 0.87W_SW_H \\
- 0.16C_SW_H + 1.41W_SH_H + 0.002C_SH_H + 0.76W_HH_H \\
+ 0.06W_SD + 0.39C_SD - 0.17W_HD + 0.33H_D \\
- 0.42W_STS_{bt} + 0.06C_STS_{bt} + 0.32W_HT_{bt} - 0.52H_HT_{bt} \\
+ 0.10DT_{bt} - 0.26W_SM_t + 0.17C_SM_t - 0.47W_HM_t \\
+ 0.10DT_{bt} - 0.26W_SM_t + 0.17C_SM_t - 0.47W_HM_t \\
- 0.53H_HM_t + 0.24DM_t - 0.39M_tD + 0.85W_S^2R_D \\
+ 0.37C_SR_D - 0.03W_HR_D - 0.18H_HR_D + 0.73R_D \\
- 0.19T_{bt}R_D - 0.26M_tD - 0.91W_S^2W_S + 0.38C_SC_S \\
+ 0.46W_HW_H + 0.16H_HH_H + 0.49DD + 0.68T_{bt}T_{bt} \\
+ 0.46M_tM_t - 0.08R_D^2D \\
\]

\[
\hat{y}_M = 19.96 + 0.34W_S - 0.02C_S + 0.63W_H - 0.33H_H - 0.15D + 0.08T_{bt} \\
+ 0.06M_t + 0.47R_D + 0.07W_SC_S - 0.80W_SW_H \\
+ 0.004C_SW_H + 0.70W_SH_H + 0.09C_SH_H + 0.08W_H^2H_H \\
- 0.3W_SD + 0.10C_SD + 0.20W_H^2D + 0.15H_DD \\
- 0.40W_STS_{bt} - 0.29C_STS_{bt} + 0.09W_HT_{bt} + 0.04H_HT_{bt} \\
+ 0.27DT_{bt} - 0.05W_SM_t + 0.12C_SM_t - 0.14W_HM_t \\
+ 0.27DT_{bt} - 0.05W_SM_t + 0.12C_SM_t - 0.14W_HM_t \\
- 0.13H_HM_t + 0.05DM_t - 0.4M_tD + 0.27W_SR_D \\
+ 0.16C_SR_D - 0.25W_HR_D - 0.37H_HR_D + 0.43R_DD \\
- 0.31T_{bt}R_D - 0.06M_tD - 0.04W_SW_S + 0.44C_SC_S \\
+ 0.47W_HW_H - 2.07H_HH_H + 0.45DD + 0.4T_{bt}T_{bt} \\
+ 0.4M_tM_t + 0.3R_D^2D \\
\]

\[
\hat{y}_v = 16.04 + 0.78W_S - 0.01C_S + 0.57W_H - 0.55H_H - 0.01D - 0.31T_{bt} \\
- 0.02M_t - 0.40R_D + 0.20W_SC_S - 0.52W_SW_H \\
+ 0.06C_SW_H - 0.08W_SH_H + 0.10C_SH_H + 0.22W_HW_H \\
+ 0.56W_SD + 0.06C_SD + 0.07W_DH_D \\
- 0.44W_ST_{bt} - 0.08C_ST_{bt} + 0.26W_HT_{bt} + 0.38H_HT_{bt} \\
- 0.09DT_{bt} - 0.10W_SM_t + 0.18C_SM_t + 0.08W_HM_t \\
+ 0.01H_HM_t - 0.12DM_t - 0.29M_tT_{bt} - 0.06W_SR_D \\
+ 0.09C_SR_D - 0.23W_HR_D + 0.48H_HR_D - 0.47R_DR_D \\
+ 0.59T_{bt}R_D - 0.25M_tR_D - 0.37W_SW_S - 0.24C_SC_S \\
+ 0.005W_HW_H + 2.02H_HH_H - 0.2DD - 0.31T_{bt}T_{bt} \\
- 0.26M_tM_t - 0.24R_DR_D \tag{3.13}
\]

\[
\hat{y}_{b\delta} = 19.59 + 8.20W_S - 0.13C_S + 0.72W_H + 0.76H_H - 0.49D \\
+ 0.46T_{bt} - 0.75M_t + 4.66R_D - 0.21W_SC_S - 0.61W_SW_H \\
+ 0.12C_SW_H - 0.60W_SH_H + 0.69C_SH_H + 0.61W_HW_H \\
- 0.45W_SD + 0.15C_SD + 0.46W_DH_D \\
- 0.21W_ST_{bt} + 0.37C_ST_{bt} + 0.39W_HT_{bt} - 0.25H_HT_{bt} \\
+ 0.79DT_{bt} - 0.005W_SM_t - 0.28C_SM_t - 0.32W_HM_t \\
- 0.25H_HM_t - 0.05DM_t - 0.85M_tT_{bt} + 2.40W_SR_D \\
+ 0.21C_SR_D + 0.66W_HR_D + 0.18H_HR_D + 0.62R_DR_D \\
- 0.15T_{bt}R_D - 0.86M_tR_D - 7.19W_SW_S + 1.78C_SC_S \\
+ 1.78W_HW_H + 1.51H_HH_H + 1.80DD + 1.73T_{bt}T_{bt} \\
+ 1.80M_tM_t - 9.32R_DR_D \tag{3.14}
\]
3.5.2 Stepwise Multiple Linear Regression Models

The SMLR procedure started with a selection of all the input parameters range as shown in Table 1 to develop a simulation matrix using the Latin Hypercube Sampling (LHS) technique. The nonlinear aero-hydro dynamic simulation was then performed in FAST. The screening analysis was performed through a stepwise regression procedure to eliminate the insignificant input parameters. The significant input parameters were then used to develop explanatory functions based on the law of mechanics and engineering judgment. Finally, multiple linear regression is performed to develop SMLR model for the observed OWTs response. A detailed description of this modeling approach is provided in (). The proposed models for tower top deflection, mudline flexure, mudline shear, and blade deflection are shown in Eq (3.15)-(3.16), respectively.

\[
\ln(\hat{y}_\delta) = -8.037 - 1.30\ln\left(\frac{W_H}{H_H}\right) + 0.22\ln\left(\frac{W_S}{C_s}\right) - 0.69\ln\left(\frac{M_T}{W_H}\right)
+ 0.74\ln\left(\frac{W_H}{M_D}\right) - 1.51\ln\left(\frac{T_{BD}}{M_D}\right) - 0.33\ln\left(\frac{T_{bt}}{M_T}\right)
\]

\( (3.17) \)

\[
\ln(\hat{y}_m) = 7.65 + 2.23\ln\left(\frac{W_H}{H_H}\right) + 0.12\ln\left(\frac{W_S}{C_s}\right) - 2.40\ln\left(\frac{R_D}{H_H}\right)
- 1.99\ln\left(\frac{W_H}{M_D}\right) + 1.85\ln\left(\frac{T_{BD}}{M_D}\right)
\]

\( (3.18) \)

\[
\ln(\hat{y}_v) = 22.43 + 4.60\ln\left(\frac{W_H}{H_H}\right) - 0.08\ln\left(\frac{W_S}{C_s}\right) - 2.45\ln\left(\frac{R_D}{H_H}\right)
+ 1.22\ln\left(\frac{M_T}{W_H}\right) - 2.88\ln\left(\frac{W_H}{M_D}\right) + 3.61\ln\left(\frac{T_{BD}}{M_D}\right)
\]

\( (3.19) \)
\[ \ln(\gamma_{bs}) = 0.4439 - 0.51\ln\left(\frac{W_H}{H_H}\right) + 0.21\ln\left(\frac{W_S}{C_S}\right) + 0.72\ln\left(\frac{R_D}{H_H}\right) \\
+ 0.56\ln\left(\frac{H_W}{M_D}\right) - 0.49\ln\left(\frac{T_{BD}}{M_T}\right) \]

(3.20)

3.6 Fragility Estimates for Vulnerability Analysis

The developed surrogate models were used to generate fragility curves for a wide range of input parameters using the MCS reliability analysis technique to compute the exceedance probabilities for deflection, flexure, and shear failure limit state. The input parameters were considered to have a random uniform distribution to account for the inherent randomness over the given range.

For the deflection at the tower top, the drift of 5% was considered as the deformation capacity following Adhikari et al. (2014) to define the serviceability limit state. The drift ratio is defined as the ratio of the deflection at tower top to the hub height. The limit on blade tip deflection is taken as 5% of the blade length following Young et al. (2010) and Hu et al. (2012). The flexural capacity at the mudline region of the monopile OWT tower is computed using the expression shown in Eq. (3.21)(3.22).

\[ C_m = f_y z \]

(3.22)

where \( f_y \) is the yield strength of the structure and \( z \) is the plastic section modulus of the structure. For the fragility analysis, the OWT structure is considered to be built with S355 steel having a design strength of 410 MPa and having a coefficient of variation of 5% (Karmazinova and Melcher, 2012). The shear capacity is defined as shown in Eq. (3.23).

\[ C_v = f_y A \left(\frac{3}{4}\right) \left(\frac{r_e^2 + r_l^2}{r_e^2 + r_o r_l + r_e^2}\right) \]

(3.24)
where $A$ is the tower base area, $r_e$ and $r_i$ are tower external and internal diameter, respectively.

3.7 Surrogate Results Comparison

This section discusses the statistical comparison of surrogate models with the response observed from the FAST simulation. Additional comparison with respect to the fragility curves developed using two approaches in terms of wind speed and wave height are done.

3.7.1 Statistical Comparison to FAST Data

To statistically determine the performance of the surrogate models, the response from the nonlinear FAST simulations were compared to the surrogate models as shown in Table 3.4. For comparison among the models, three measures are selected to determine the performance, the mean absolute error (MAE), the maximum absolute error (MAX), and the root mean square error (RMSE). The MAE is used to measure the spread of data while including the effect of the total data set. The MAX is the sum of maximum relative error and maximum absolute error. The RMSE is the expected value of the square of the error and useful indicator of the average magnitude of the error. It can be seen that MAE and RMSE in the models are lower than the MAX error. Lower MAE and RMSE indicates a strong association between the observed and predicted response. On comparison of RSM and SMLR model, the statistical error of the RSM model is significantly lower than that of the SMLR. The MAE error in SMLR is above 30% while for the RSM model the highest error is 16.82% for deflection. Similar results are observed for the RMSE and MAX error.
Further comparison was done based on the observed statistical results and computational time required to develop a model. The statistical results from the surrogate models shown in Table 3.5 show that RSM approach resulted in higher $R^2$ value for all the modes of failure, while the $R^2$ value is relatively low for the SMLR approach. Because of the larger set of input parameters and limited computational simulations, the $R^2$ value for the SMLR model was observed to be lower than the RSM model. However, the computational time is also one of the aspects for determining the efficiency of the model. The computational time is less while utilizing RSM approach with the total estimated time of less than 175 h of CPU time, whereas the SMLR approach resulted in total estimated CPU time of over 200 h. The large proportion of the computational time is required to perform the nonlinear aeroelastic simulation in FAST. The additional time was used for the screening and regression analysis which was relatively higher for SMLR.

Table 3.5. Summary of the response observed from SMLR and RSM modeling approach

<table>
<thead>
<tr>
<th>Responses</th>
<th>Multiple R-squared</th>
<th>Computational time</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SMLR</td>
<td>RSM</td>
</tr>
<tr>
<td>Tower Top Deflection</td>
<td>0.77</td>
<td>0.95</td>
</tr>
<tr>
<td>Mudline Flexure</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mudline Shear</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Blade Tip Deflection</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Mudline Flexure  
Mudline Shear  
Blade Tip Deflection  

<table>
<thead>
<tr>
<th></th>
<th>0.24</th>
<th>0.74</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0.35</td>
<td>0.76</td>
</tr>
<tr>
<td></td>
<td>0.37</td>
<td>0.95</td>
</tr>
</tbody>
</table>

3.7.2  Graphical Comparison to FAST Data

Further, to determine the performance of the surrogate models, the responses observed from the FAST simulation and the surrogate models are graphically compared in Figure 3.3-Figure 3.6. Such comparison provides an overall idea of the obtained responses from these methodologies and helps in understanding the predicted response and its deviation from the actual simulations performed in FAST. In the graphical comparison, every other parameter was considered in the median value in the given range of structural and loading parameters.
Figure 3.3: Comparison of tower top deflection observed using the SMLR, FAST, and RSM approach with respect to a) hub height; b) monopile thickness; c) wind speed; and d) wave height.

Figure 3.3(a)-Figure 3.3(d) provides tower top deflection observed from the surrogate models and the FAST simulation data points. The tower top deflection response is observed with respect to hub height, monopile thickness, wind speed, and wave height, respectively. It was observed that all of the models followed a similar trend with the increase in structural/loading parameters. There are some differences observed while observing the deflection response, mainly because of difference in modeling approaches. The plot also shows the FAST simulation response has a closer resemblance to the RSM model than to the SMLR model. This is because the RSM model considers the nonlinear interaction of input parameters to estimate the predicted response.
Figure 3.4: Comparison of mudline flexure observed using the SMLR, FAST, and RSM approach with respect to a) hub height; b) monopile thickness; c) wind speed; and d) wave height.

A similar plot is developed for the mudline flexure as shown in Figure 3.4(a)-Figure 3.4(d) for hub height, monopile thickness, wind speed, and wave height, respectively. It can be observed that SMLR response is not observed for the monopile thickness in Figure 3.4(b), this is because of the fact that the SMLR model did not consider monopile thickness parameter for the mudline flexure. From the figure, the FAST simulation data point is observed to lie closer to the RSM predicted response. Also, the wind speed caused the highest flexural response.

Figure 3.5: Comparison of mudline shear force observed using the SMLR, FAST, and RSM approach with respect to a) hub height; b) monopile thickness; c) wind speed; and d) wave height.
Similarly, the response plot for the mudline shear is shown in Figure 3.5(a)-Figure 3.5(d). The shear response is similar to the flexure response. The RSM response is observed to have a closer resemblance to the FAST data points than the SMLR model.

Figure 3.6: Comparison of blade tip deflection observed using the SMLR, FAST, and RSM approach with respect to a) hub height; b) monopile thickness; c) wind speed; and d) wave height.

Figure 3.6 presents the blade tip deflection response observed from the different approaches. All the three approaches yielded with different responses. However, the response from the FAST simulation matches close to the RSM model response. Also, the blade tip deflection varies to a great extent with the variation in wind speed. Other plots are developed at a median value of wind speed resulting in such higher deflection. Therefore, it can be concluded that wind speed is critical for the blade tip deflection.
3.7.3 Fragility Comparison

The RSM fragility curve was compared with the SMLR fragility curve in this section. Figure 3.7-Figure 3.9 show the comparison of the fragility curves for the tower top deflection, mudline flexure, mudline shear, and blade tip failure developed from the RSM model with the fragility curves developed from SMLR model and FAST simulation. The exceedance probability data points using the FAST simulation is shown to determine the estimated exceedance probability behavior using the non-linear simulation and to compare the result observed from the surrogate models. The wind fragility curve is developed at a constant wave height of 10 m, while the wave fragility curve is observed at a constant wind speed of 30 m/s. The fragility curve using the SMLR model, the RSM model, and the FAST simulation with respect to wind speed is shown in Figure 3.7(a). The peak exceedance probability for deflection is observed to be 0.85 for SMLR approach, while with the RSM model the peak exceedance probability is 0.65 at a wind speed of 75 m/s. The difference in peak exceedance probability is observed to be nearly 23%. This is observed because the SMLR model involved only linear terms of the input parameter without consideration of interaction terms in the model. Similarly, the deflection fragility curve for variation in wave height is shown in Figure 3.7(b). It can be observed that the exceedance probability using SMLR model started from 0.25 at 2 m wave height and increased to 0.70 at 20 m wave height. Similarly, the RSM model started at $6.70 \times 10^{-3}$ at 2 m wave height and increased to 0.36 at a wave height of 20 m. Because of the consideration of the nonlinear behavior of the response in RSM model, the SMLR model resulted in relatively conservative value for exceedance probability than the RSM model.
Figure 3.7: Fragility curve for a monopile 5 MW OWT tower top deflection: a) with respect to wind speed and b) with respect to wave height.

Similar fragility curves for flexure using SMLR model and RSM model are shown in Figure 3.8. Figure 3.8(a) illustrates the flexural fragility behavior with respect to the wind speed. The exceedance probability increases significantly from 0.10 at 5 m/s to 0.85 at 75 m/s for SMLR model with an increase in wind speed, while for the RSM model the probability increased gradually from 0 to its extreme value of 0.44. The percent difference at an extreme wind speed of 75 m/s is nearly 48%. The fragility behavior for wave height variation is shown in Figure 3.8(b). It is apparent that the wave exceedance probability for flexure using RSM shows a gradual increase from 0 with increment in wave height, while the exceedance probability using SMLR model increases significantly from 0.12 at 2 m wave height. The peak flexural exceedance probabilities using SMLR and RSM model were 0.93 and 0.53, respectively, observed at 20 m wave height. The percentage difference for the peak exceedance probability is nearly 43%. This difference is observed because of consideration of quadratic term in the RSM model,
which considered the nonlinear behavior of the observed response resulting in a less
critical exceedance probability, as opposed to the SMLR model.

The fragility curves for shear failure at the mudline region of the OWT using the
SMLR and the RSM model are shown in Figure 3.9. Figure 3.9(a) represents the
exceedance probability with an increase in wind speed. It can be observed that SMLR
model resulted in an increment from 0.05 at 3 m/s to 0.19 at 75 m/s in the fragility curve
while the RSM model resulted in an increase in the shear exceedance probability from
3.40 × 10⁻³ at 3 m/s to 0.14 at 75 m/s. The shear fragility curve for wave height variation
is shown in Figure 3.9(b). The shear exceedance probability with the SMLR model
increased from 0 at 2 m wave height to 0.11 at 20 m wave height. For RSM model, a
similar trend was observed with the shear exceedance probability increasing gradually
from 2.9 × 10⁻³ at 2 m wave height to 8.36 × 10⁻² at 20 m wave height.
Similarly, the fragility curves for the blade tip deflection using the surrogate models are shown in Figure 3.10. The fragility curve with the surrogate models and FAST simulation data with variation in wind speed is shown in Figure 3.10(a). The increase in wind speed increased the exceedance probability using both models. The exceedance probability for the SMLR model increased from 0.04 at 3 m/s to 0.98 at 75 m/s, while the probability using the RSM model increased from 0 at 3 m/s to 0.82 at 75 m/s. A similar curve with respect to wave height variation is shown in Figure 3.10(b). The exceedance probability using the SMLR model is observed to 0.42 at 2 m of wave height increased to 0.80 at 20 m of wave height, whereas the increment is from 0.64 to 0.78 for the RSM model. This high value of exceedance probability is because of median wind speed considered for the analysis.
Further, the peak exceedance probability observed from the surrogate models were compared. The peak exceedance probability was observed to have a high variation for deflection and flexure, however, for the shear, the peak exceedance was similar. Because of the linear behavior of the response and lack of interaction terms in the SMLR based model, the peak exceedance probability was conservative with values of 0.94, 0.98, 0.16, and 0.99 for tower top deflection, mudline flexure, mudline shear, and blade tip deflection, respectively. On the other hand, the second order RSM model includes the nonlinear behavior of the observed response for the failure estimation. This resulted in exceedance probability which was less conservative than the SMLR model with exceedance probability of 0.65, 0.85, 0.15, and 0.83 for tower top deflection, mudline flexure, mudline shear, and blade tip deflection, respectively. Both the surrogate models, in spite of the differences in the modeling approaches, estimated that the wind speed related failure was more critical than wave height related failure.

From the comparison of the fragility value obtained using SMLR and RSM model, it can be observed that the SMLR approach yielded more conservative wind and wave
fragility curve for all responses. Further, it can also be concluded that the RSM model predicts the performance of the OWT better than the SMLR model with lower statistical error and computational effort while maintaining a high level of accuracy. Also, the exceedance probability estimates using FAST simulation tend to lie closer to the fragility curve developed using RSM model, resembling actual prediction of the exceedance probability. Therefore, the RSM model was further used for multi-hazard vulnerability assessment.

3.8 Multi-Hazard Vulnerability Function

This section discusses in detail about the fragility surfaces obtained as a result of RSM-MCS simulation over different significant structural and loading parameters. This section also identifies the critical value of each structural parameter for the multi-hazard loading scenarios using both surrogate models.

3.8.1 RSM Fragility Surface for OWTs

As a result of RSM-MCS simulation, fragility curves were developed for the deflection, flexure, and shear failure of the monopile OWT structure. The following subsections describe the observed fragility behavior for deflection, flexure, and shear using the developed RSM models.

A representative OWT fragility surface is developed for the deflection at the tower top of the structure as shown in Figure 3.11. It can be observed in Figure 3.11(a) that wind speed plays a significant role in the deflection failure of the structure, reaching an exceedance probability of 0.64 at an extreme wind speed of 75 m/s and a low wave height of 1 m. On the other hand, the contribution of the wave height to the deflection
failure is relatively less, with an exceedance probability of 0.13 at an extreme wave height of 20 m and a low wind speed of 3 m/s. The deflection at tower top is directly affected by wind speed which increases the load at the tower top resulting in an increased tower top deflection. The wave load also increases the load on the turbine, but the effect is minimal in comparison to the wind load. The failure surface is almost negligible for wind speeds less than 20 m/s and wave height of less than 5 m. Beyond that, the exceedance probability increased rapidly to 0.66 at a wind speed of 75 m/s and wave height of 20 m. At an extreme wind speed of 75 m/s, increase in the wave height caused no significant increment in the exceedance probability while at an extreme wave height of 20 m, increase in wind speed increased the exceedance probability from 0.13 to 0.66. It can, therefore, be concluded that the wind speed is observed to be the critical loading parameter for the tower top deflection.

The tower top deflection fragility surface for different hub height and monopile thickness is shown in Figure 3.11(b). The increase in hub height is observed to increase the tower top deflection failure overall monopile thickness values. However, large monopile thickness is seen to reduce the peak exceedance probability with hub height increment. The increased hub height resulted in observed exceedance probabilities ranging from 0 to 0.47 at monopile thickness of 0.15 m, whereas at monopile thickness of 0.068 m the probabilities ranged from 0 to 0.85. The increase in hub height increased the deflection demand due to the cantilever action resulting in an increased deflection failure probability. Similarly, the increased monopile thickness led to an increase in the resistance of the structure reducing the exceedance probability.
A similar phenomenon is observed in Figure 3.11(c) when the hub height is plotted with respect to the tower base thickness. Increase in the hub height increased the deflection failure from 0 to 0.65 for all values of tower base thickness, but the increase in tower base thickness is observed to have minimal impact on the deflection failure with probability varying from 0.65 to 0.72 at 120 m hub height.

The failure surface for water depth and rotor diameter is plotted Figure 3.11(d). The increment in the rotor diameter at low water depths caused no change in the exceedance probability. However, at low rotor diameters, increase in the water depth is observed to cause a slight decrease in the exceedance probability from 0.24 at a water depth of 20 m to 0.18 at a water depth of 30 m. At higher water depths, the increment in the rotor diameter increased the exceedance probability, with the peak exceedance probability observed being when both the water depth and rotor diameter were at an extreme range of 30 m and 151 m, respectively causing the exceedance probability to reach 0.48.
Figure 3.11: 3-Dimensional fore-aft tower top deflection fragility surface observed for the OWT subjected to variation in input parameters: a) wind speed and wave height; b) monopile thickness and hub height; c) hub height and tower base thickness, and d) water depth and rotor diameter.

The overturning flexural failure profile at the mudline region of the OWT is represented in Figure 3.12. The failure surface increases gradually with the increase in both wind speed and wave height as shown in Figure 3.12(a). At an extreme wind speed of 75 m/s and an extreme wave height of 20 m, the exceedance probability reached 0.85. The increase in wave height, while maintaining a constant wind speed of 5 m/s increased the failure behavior of the structure. The exceedance probability due the wave load increased from 0 at a wave height of 1 m and reached 0.65 at a wave height of 20 m. The increase in wind speed caused the exceedance probability to rise from 8.4 × 10^-2 to 0.39 at a wave height of 1 m, while the exceedance probability increased from 0.65 to 0.85 at a wave height of 20 m. It is noteworthy that the flexural failure surface is a flat line when the wind speed and wave height are below 25 m/s and 5 m, respectively. However, the exceedance probability increased with increase in each of the loading parameters. This is because the overturning flexure at the mudline region of the monopile increased with
increase in the wind as well as wave loads due to the cantilever action. It can also be concluded that the wave height contributes to the exceedance probability to a greater extent when the wave height is at its extreme value.

The flexural exceedance probability surface for the variation of monopile thickness and hub height is shown in Figure 3.12(b). It can be observed that the increase in the hub height amplified the exceedance probability for all values of monopile thickness. The exceedance probability increased from 0.0182 at a hub height of 80 m to 0.82 at a hub height of 120 m at a constant monopile thickness of 0.068 m. Increase in the monopile thickness caused a slight decrement in the flexural exceedance probability.

Similar failure behavior is observed for the variation in hub height and tower base thickness as represented in Figure 3.12(c). Increase in the hub height caused the flexural exceedance probability to increase from 0.03 at a hub height of 80 m to 0.85 at a hub height of 120 m, observed for a constant tower base thickness of 0.027 m. However, the exceedance probability reduced on increasing the tower base thickness which resulted in a peak exceedance probability value of 0.76 at a tower base thickness of 0.068. Such phenomena can be observed because of the increase in the structural capacity of the structure with the increment in tower base thickness and the monopile thickness.

The mudline flexural fragility surface for varying rotor diameter and water depth is represented in Figure 3.12(d). The exceedance probability remained consistent with increment in the rotor diameter at a low water depth of 20 m. The exceedance probability reduced slightly with the increase in water depth at a rotor diameter of 115 m, whereas at a high rotor diameter of 151 m, the increase in the water depth from 20 m to 30m caused
the exceedance probability to increase from 0.57 to 0.84. The increase in rotor diameter along with water depth increased the wind and wave load on the structure resulting in a higher flexural exceedance probability.

![Exceedance Probability Surfaces](image)

Figure 3.12: 3-Dimensional fore-aft mudline flexural fragility surface observed for the OWT with variation in input parameters: a) wind speed and wave height; b) monopile thickness and hub height; c) hub height and tower base thickness, and d) rotor diameter and water depth.

Similarly, the fragility surface for the shear failure at the mudline region is illustrated in Figure 3.13. The shear failure for wind and wave variation in Figure 3.13(a) is observed to have a peak exceedance probability of 0.15 at an extreme wind speed of 75
m/s and a wave height of 20 m. The increase in the wind speed is observed to increase the exceedance probability, while the wave height only contributed to the failure at higher values. The wave load alone (while maintaining constant wind speed) causes the exceedance probability to reach 0.07 while the wind load alone caused the failure to reach 0.13. Wave heights of less than 5 m and wind speeds of less than 25 m/s do not seem to contribute to the failure, with failure surface observed as a flat line for these ranges with negligible exceedance probability.

The shear fragility surface with respect to varying hub height and monopile thickness is shown in Figure 3.13(b). The shear failure surface remained insignificant for hub heights below 110 m. The increment in hub height above 110 m, however, increased the exceedance probability from 0.03 at a hub height of 80 m to 0.6 at a hub height of 120 m. The increment in the monopile thickness caused a slight decrease in the shear exceedance probability. The observed probability decreased from 0.6 at 0.068 m monopile thickness to 0.55 at 0.15 m monopile thickness at a hub height of 120 m.

For the comparison of tower base thickness and hub height on the shear failure, a representative failure surface is plotted in Figure 3.13(c). The obtained failure surface was a smooth flat line with zero exceedance probability until the hub height of 100 m. Increase in the hub height above 100 m caused the exceedance probability to rise from 0.02 at 80 m to 0.68 at 120 m hub height. On the other hand, the increase in tower base thickness lowered the exceedance probability with a value of 0.68 at 0.027 m tower thickness which reduced to 0.34 at 0.068 m tower base thickness for a constant hub height of 120 m.
The shear exceedance probability surface is shown in Figure 3.13(d) with respect to water depth and rotor diameter. The failure behavior was observed to be directly proportional to rotor diameter and inversely proportional to the water depth. Increase in the rotor diameter at a constant water depth of 30 m caused the exceedance probability to rise from 0.07 to 0.23, whereas at a water depth of 20 m the exceedance probability increased from 0.0035 to 0.0259. It can thus be concluded that shear failure is maximum when the water depth and the rotor diameter are at extreme values of 30 m and 151 m, respectively.

Figure 3.13: 3-Dimensional fore-aft mudline shear fragility surface observed for the OWT with variation in input parameters: a) wind speed and wave height; b) monopile thickness and
hub height; c) hub height and tower base thickness, and d) rotor diameter and water depth.

Figure 3.14 represents the fragility surface developed for the blade tip deflection for the OWT with the variation in input parameters. Since most of the structural parameters considered have no effect on blade tip deflection, therefore, only significant parameters causing direct impact on the observed response have been studied here. Figure 3.14(a) illustrates the blade tip fragility surface for the variation in wind speed and wave height. It is observed that increase in wind speed caused the exceedance probability to increase significantly with peak exceedance probability of 0.88 at a wind speed of 75 m/s. The increase in wave height is observed to have minimal impact on exceedance probability. This phenomenon is observed because wind speed has a direct effect on the blade deflection but wave height causing higher wave load is insensitive to the deflection of the blade.

The fragility surface of the rotor diameter and hub height is shown in Figure 3.14(b). It can be observed that increase in rotor diameter caused the exceedance probability to increase for all the values of hub height. It is observed that the peak exceedance probability reaches to 0.80 at extreme rotor diameter of 151 m and an extreme hub height of 120 m. The hub height slightly increased the fragility surface, for a rotor diameter of 151 m, the exceedance probability with an increase in hub height increased from 0.76 at 80 m of hub height to 0.80 at 120 m of hub height. This is because the increase in rotor diameter increased the blade length causing higher deflection demand due to cantilever action.
Figure 3.14: 3-Dimensional blade tip deflection fragility surface observed for the OWT with variation in input parameters: a) wind speed and wave height; b) rotor diameter and hub height.

From the above discussion, it is observed that flexural failure is relatively higher than other modes of failure in monopile OWTs. Moreover, wind speed plays a vital role in estimating the structural component failure of the OWT, but the effect of wave height on exceedance probability cannot be ignored for estimating the mudline flexural response. On further comparison of the fragility surface with respect to the structural parameters, it was observed that hub height was a critical input parameter directly affecting the exceedance probability for tower top deflection, flexure, and shear response. Also, the increase in the rotor diameter also led to an increase in the exceedance probabilities, especially for the blade tip deflection. Increase in both the hub height and the rotor diameter increases the load on the OWT tower which explains this increase in the exceedance probability. The increase in monopile thickness, however, is observed to reduce the exceedance probabilities for tower top deflection, mudline flexure, and mudline shear modes of failure as expected. This is because the increase in the monopile
thickness increases the structural capacity of the structure at mudline thereby reducing the estimated exceedance probability of the structure.

3.8.2 Effect of Structural Parameters under Wind and Wave Loading

In this section, the critical values of the structural parameters which result in an exceedance probability of the structure are determined. In this study, 25% exceedance probability is considered. Such an analysis can help in identifying the necessary modifications in the critical design parameters which could improve the reliability of the OWT structure in multi-hazard loading scenarios. For this, one of the structural parameters was varied within the given range (as shown Table 3.1) while the remaining parameters were fixed at their median value.

Table 3.6 represents the value of each significant structural parameter which resulted in an exceedance probability of 25% for random values of loading parameters while using the RSM model. From the failure analysis with respect to hub height, it is observed that the increment in the hub height increased the exceedance probability in all failure modes with 25% exceedance probability being observed at a hub height of 109.83 m, 95.95 m, and 117.5 m for deflection, flexure, and shear response, respectively. Similarly, the increase in the monopile thickness decreased the exceedance probability with 25% probability being observed at a monopile thickness of 0.15 m, 0.068 m, and 0.133 m, respectively. Similarly, the critical values for rotor diameter, tower base thickness, and water depth are tabulated in Table 3.6. It should be noted that increase in structural parameters such as hub height, monopile thickness, tower base thickness, and water depth had no effect on exceedance probability for the blade tip deflection, therefore
no critical values are observed. The rotor diameter was observed to affect the exceedance probability for the blade tip deflection, with 25% probability observed at 118.97 m.

Table 3.6. Summary of the response observed for structural parameters at 25% exceedance probability using RSM model

<table>
<thead>
<tr>
<th>Observed Response</th>
<th>Structural Parameters</th>
<th>Loading Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hub Height (m)</td>
<td>Monopile Thickness (m)</td>
</tr>
<tr>
<td>Tower Top Deflection</td>
<td>109.83</td>
<td>0.15</td>
</tr>
<tr>
<td>Mudline Flexure</td>
<td>95.95</td>
<td>0.068</td>
</tr>
<tr>
<td>Mudline Shear</td>
<td>83.68</td>
<td>0.133</td>
</tr>
<tr>
<td>Blade Tip Deflection</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Note: * represents random variables and N/A represent no effect on the observed response.

A similar analysis is performed to determine the estimate of each significant structural parameter resulting in 25% exceedance probability while using SMLR model and is tabulated in Table 3.7. For the analysis, the randomness in loading parameters is considered. From the failure analysis, it is observed that the increment in the hub height cause increase in exceedance probability for all observed response with 25% probability observed at a hub height of 108.31 m, 85.77 m, and 86.91 m, for the deflection, flexure, and shear, respectively. Similarly, the increase in the monopile thickness decreased the exceedance probability with observed 25% exceedance probability observed at monopile thickness of 0.14 m and 0.11 m for deflection and shear response, respectively. The SMLR model for flexure did not consider the monopile thickness parameter, therefore, it is not included in the analysis. Also, the water depth was not considered in the SMLR model, thus not included in the analysis. Similarly, the critical values for the rotor diameter, tower base thickness are tabulated in Table 3.7. The blade tip deflection failure
was insensitive to the structural parameters such as hub height, monopile thickness, tower base thickness, and water depth, therefore the critical values are not included in the analysis. The only parameter affecting the exceedance probability is rotor diameter with 25% exceedance probability observed at 139.26 m.

Table 3.7. Summary of the response observed for structural parameters at 25% exceedance probability using RSM model

<table>
<thead>
<tr>
<th>Observed Response</th>
<th>Structural Parameters</th>
<th>Loading Parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hub Height (m)</td>
<td>Monopile Thickness (m)</td>
</tr>
<tr>
<td>Tower Top Deflection</td>
<td>108.31</td>
<td>0.14</td>
</tr>
<tr>
<td>Mudline Flexure</td>
<td>85.77</td>
<td>-</td>
</tr>
<tr>
<td>Mudline Shear</td>
<td>86.91</td>
<td>0.11</td>
</tr>
<tr>
<td>Blade Tip Deflection</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

Note: * represents random variables, - means parameters not included in the model, and N/A represents no effect on the observed response

On comparison of the critical parameters obtained from the SMLR and RSM model, the difference in critical hub height value for the deflection, flexure, and shear is 1.38%, 10.61%, and 3.8%, respectively. Similarly, for the monopile thickness, the difference observed is 6.7% and 17% for the deflection and shear, respectively. The differences in terms of critical rotor diameter are 17% and 19% for flexure and shear, respectively. The difference in tower base thickness for deflection is nearly 32%. The difference for the rotor diameter for the blade tip deflection is 14.56%. From the comparison, it can be observed that both methods concluded in the hub height and monopile thickness being the most critical parameters experiencing 25% exceedance probability at lower hub height and larger monopile thickness value, respectively.
Figure 3.15 shows the effect of the significant input parameters on different modes of failure of the OWT with respect to wind and wave loadings using the RSM model. This analysis will be useful in understanding the critical input parameter of OWT for each response observed in terms of wind speed and wave height. The blade tip deflection is not studied here as its effect on variation in structural parameter except for rotor diameter is insignificant. The exceedance probability here is measured in terms of quartile (25%) wind speed in Figure 3.15(a) and quartile wave height in Figure 3.15(b) for each of the parameters being considered. The green bar denotes the quartile exceedance probability observed for deflection failure with respect to wind and wave loading, while the blue and yellow represents the flexural and shear failure at mudline, respectively. For the tower top deflection failure, monopile thickness was observed to be the most critical parameter with 25% exceedance probability observed at a wind speed of 13.8 m/s and wave height of 8.85 m while varying the monopile thickness. The current speed was observed to be less critical with quartile exceedance probability observed at wind speed above 75 m/s and wave height above 20 m.
Figure 3.15: Effect of significant input parameters on 25% monopile OWT fragility behavior for deflection, flexure, and shear failure a) with respect to wind speed and b) with respect to wave height.

For the quartile flexural failure of OWT, the hub height parameter was observed to be the most critical input parameter with the considered failure observed at wind speed of 31.1 m/s and wave height of 5.87 m, while the water depth, rotor diameter, and current speed parameter were observed to be the least critical parameters with quartile failure reaching beyond wind speed of 75 m/s and wave height of 20 m. Similarly, for the quartile shear failure, water depth was observed to be the most critical input parameter reaching quarter exceedance probability at wind speed of 10.75 m/s and wave height of 9.44 m, while hub height, current speed, and monopile thickness were the least critical parameters with considered failure reaching at wind speed higher than 75 m/s and wave height above 20 m.
3.9 Conclusions and Future Work

The study compared two surrogate models developed using Response Surface Metamodels (RSMs) and Stepwise Multiple Linear Regression (SMLR) approach in terms of statistical performance, computational time, and observed fragility curve. The fragility curves were compared for increasing wind speed and wave height for tower top deflection, mudline flexure, mudline shear, and blade tip deflection failure. The results showed that SMLR model predicted the relatively more conservative response of the OWTs in comparison to the RSM model. All the results suggest that the RSM based model for the prediction of OWT performance yielded in higher accuracy with less computational effort. As a result of this analysis, the surrogate model developed using RSM could be used in the OWT design to predict the failure behavior of the OWT for multi-hazard risk assessment.

The RSM model was further used to determine the fragility behavior with respect to significant input parameters for tower top deflection, mudline flexure, mudline shear, and blade tip deflection. The RSM flexural failure surface was observed to be most critical among others. Under the variation of wind speed and wave height, the peak exceedance probability for deflection, flexure, and shear was 0.65, 0.85, 0.15, and 0.83, respectively. From the fragility surface, it was observed that the hub height and the rotor diameter increment caused the exceedance probability to increase for all responses, while the increase in the monopile thickness reduced the exceedance probability to some extent. Multi-hazard fragility analysis was done to determine the estimated value of the structural parameters that results in 25% exceedance probability for each of the responses. The results of the analysis suggest that the modification in those critical parameters could
improve the reliability of the OWT structure at multi-hazard loading scenarios. Further analysis of the significant input parameters was done to express the observed failure in terms of the wind speed and the wave height to determine the critical parameters for tower top deflection, mudline flexure, and mudline shear failure. It was observed that monopile thickness and hub height were the critical parameters. Such analysis is important during the design of OWTs to reduce exceedance probability thus reducing the maintenance cost during the hazardous loading conditions.

The proposed model only includes the interaction of wind and wave loading on the monopile structure. Additional analysis of the offshore environmental loadings involving ice loads, earthquake loads are needed. Further, the linear irregular wave was considered in the wave phenomena, but the offshore waves are nonlinear breaking waves which need further research. This procedure could also be extended to other types of foundations for the risk assessment.
REFERENCES


Echavarria, E. (2009). RAMS for offshore wind farms. WE@ SEA R&D.


