AN INTELLIGENT INTEGRATED METHOD FOR RELIABILITY ESTIMATION OF OFFSHORE STRUCTURES WAVE LOADING APPLIED IN TIME DOMAIN

by

Sayyed Mohsen Vazirizade

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As members of the Dissertation Committee, we certify that we have read the dissertation prepared by Sayyed Mohsen Vazirizade titled An Intelligent Integrated Method for Reliability Estimation of Offshore Structures Wave Loading Applied in Time Domain and recommend that it be accepted as fulfilling the dissertation requirement for the Degree of Doctor of Philosophy.

Dr. Achintya Haldar
Date: (11/08/2019)

Dr. Jian Liu
Date: (11/08/2019)

Dr. Hassan Vafai
Date: (11/08/2019)

Dr. Hongki Jo
Date: (11/08/2019)

Dr. Jose Ramon Gaxiola-Camacho
Date: (11/08/2019)

Final approval and acceptance of this dissertation is contingent upon the candidate’s submission of the final copies of the dissertation to the Graduate College.

I hereby certify that I have read this dissertation prepared under my direction and recommend that it be accepted as fulfilling the dissertation requirement.

Dissertation Director: Dr. Achintya Haldar
Date: (11/08/2019)
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DEDICATIONS

Dedicated to
My parents
# TABLE OF CONTENTS

<table>
<thead>
<tr>
<th>LIST OF FIGURES</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>LIST OF TABLES</td>
<td>11</td>
</tr>
<tr>
<td>ABSTRACT</td>
<td>12</td>
</tr>
<tr>
<td>CHAPTER 1 INTRODUCTION</td>
<td>14</td>
</tr>
<tr>
<td>1.1 Statement of the Problem</td>
<td>14</td>
</tr>
<tr>
<td>1.2 Motivation of the Proposed Work</td>
<td>15</td>
</tr>
<tr>
<td>1.3 Objectives of the Research</td>
<td>16</td>
</tr>
<tr>
<td>1.3.1 Specific Objectives</td>
<td>16</td>
</tr>
<tr>
<td>1.4 Organization</td>
<td>16</td>
</tr>
<tr>
<td>CHAPTER 2 JACKET-TYPE OFFSHORE STRUCTURES – A BRIEF INTRODUCTION</td>
<td>19</td>
</tr>
<tr>
<td>2.1 Introduction</td>
<td>19</td>
</tr>
<tr>
<td>2.2 Morison Equation</td>
<td>22</td>
</tr>
<tr>
<td>2.3 Wave Modeling</td>
<td>23</td>
</tr>
<tr>
<td>2.3.1 Wave Modeling Using the New Wave Theory</td>
<td>24</td>
</tr>
<tr>
<td>2.3.2 Wheeler Stretching Effect</td>
<td>26</td>
</tr>
<tr>
<td>2.3.3 Directionality</td>
<td>27</td>
</tr>
<tr>
<td>2.4 Summary</td>
<td>29</td>
</tr>
<tr>
<td>CHAPTER 3 LITERATURE REVIEW OF RELIABILITY ESTIMATION OF OFFSHORE STRUCTURES</td>
<td>30</td>
</tr>
</tbody>
</table>
3.1 Introduction ................................................................................................................................................. 30
3.2 Research Done on Reliability Analysis at The University of Arizona .................................................... 32
3.3 Limitations in Current Reliability Methods for Loading in Time-Domain ................................................. 34
3.4 Summary ..................................................................................................................................................... 36

CHAPTER 4 UNCERTAINTY QUANTIFICATIONS OF OFFSHORE STRUCTURES .................................................. 38
4.1 Introduction .................................................................................................................................................. 38
4.2 Uncertainty in Structures .......................................................................................................................... 38
4.3 Uncertainty in Wave Loadings .................................................................................................................... 39
4.4 3D CNW Concept ....................................................................................................................................... 41
4.5 Uncertainty in the Wave Height Estimation .............................................................................................. 43
4.6 Uncertainty in Seismic Loadings ................................................................................................................ 45
4.7 Wave and Seismic Loadings – Comparisons ............................................................................................. 48

CHAPTER 5 RELIABILITY EVALUATION OF OFFSHORE STRUCTURES – A NOVEL SURROGATE MODEL ................. 49
5.1 Introduction .................................................................................................................................................. 49
5.2 Integration of RSM and FORM Producing AIRS ..................................................................................... 51
5.3 Generating AIRS ....................................................................................................................................... 53
5.3.1 The Form of the Polynomial ................................................................................................................ 53
5.3.2 The Center Point Around Which Samples Need to Be Generated ....................................................... 54
5.3.3 Sampling Scheme ................................................................................................................................... 55
8.1 Summary .................................................................96
8.2 Conclusions ............................................................98
8.3 Recommendations for Future Work ................................99
REFERENCES .....................................................................101
<table>
<thead>
<tr>
<th>Figure</th>
<th>Description</th>
<th>Page</th>
</tr>
</thead>
<tbody>
<tr>
<td>2.1</td>
<td>A typical jacket-type OFSs</td>
<td>20</td>
</tr>
<tr>
<td>2.2</td>
<td>A conceptual water surface</td>
<td>24</td>
</tr>
<tr>
<td>2.3</td>
<td>Particles velocity as a function of depth and time based on water surface</td>
<td>27</td>
</tr>
<tr>
<td>4.1</td>
<td>NW and CNW</td>
<td>40</td>
</tr>
<tr>
<td>4.2</td>
<td>CNW and its components generated by our script</td>
<td>41</td>
</tr>
<tr>
<td>4.3</td>
<td>surface elevation at the upwave and downwave legs (a) NW (b) CNW</td>
<td>43</td>
</tr>
<tr>
<td>4.4</td>
<td>The joint distribution of Hs significant wave height and zero-up-crossing</td>
<td>44</td>
</tr>
<tr>
<td>4.5</td>
<td>Time history of 1940 Imperial Valley earthquake at El Centro station</td>
<td>47</td>
</tr>
<tr>
<td>5.1</td>
<td>The schematic view of different factorial designs; (a) SD without cross</td>
<td>56</td>
</tr>
<tr>
<td></td>
<td>terms; (b) SD with cross terms; and (c) CCD</td>
<td></td>
</tr>
<tr>
<td>5.2</td>
<td>Advantages of Kriging over regression analysis</td>
<td>58</td>
</tr>
<tr>
<td>5.3</td>
<td>The schematic view of (a) variogram cloud (b) experiential variogram (c)</td>
<td>64</td>
</tr>
<tr>
<td></td>
<td>variogram (d) variogram</td>
<td></td>
</tr>
<tr>
<td>5.4</td>
<td>Flowchart diagram of AIRS-MUK-FORM</td>
<td>69</td>
</tr>
<tr>
<td>6.1</td>
<td>General sketch of the Platform</td>
<td>71</td>
</tr>
<tr>
<td>6.2</td>
<td>Comparing results using AIRS-MUK-FORM and MCS</td>
<td>76</td>
</tr>
<tr>
<td>6.3</td>
<td>Summarizing estimated values for $\beta$ using different LFs</td>
<td>77</td>
</tr>
<tr>
<td>7.1</td>
<td>Sample water surface level using CNW</td>
<td>80</td>
</tr>
<tr>
<td>7.2</td>
<td>Site-specific, ALE, and ELE spectra</td>
<td>83</td>
</tr>
</tbody>
</table>
Figure 7.3 Selected ground motions, their mean values, one standard deviation minus
and plus, and the target spectra ............................................................... 85

Figure 7.4 Comparing results using AIRS-MUK-FORM and MCS for wave loading .... 89

Figure 7.5 Comparing results using AIRS-MUK-FORM and MCS for earthquake
loading ........................................................................................................ 90

Figure 7.6 Summarizing estimated values for $\beta$ using different LFs for wave loading
.................................................................................................................. 92

Figure 7.7 Summarizing estimated values for $\beta$ using different LFs for earthquake
loading ........................................................................................................ 92
LIST OF TABLES

Table 5.1 The total number of coefficients for each polynomial type.......................... 54
Table 5.2 Comparison between different schemes in terms of TNDA ......................... 56
Table 6.1 Statistical information for RVs .................................................................... 73
Table 6.2 Structural reliability of OFS subjected to wave loading .............................. 74
Table 6.3 Summary of $P_f$ for the wave loading ...................................................... 77
Table 7.1 Selected time histories .............................................................................. 84
Table 7.2 Reliability estimation for the wave loading ................................................. 87
Table 7.3 Reliability estimation for the overall drift at the deck level for the
earthquake excitations .............................................................................................. 87
Table 7.4 Reliability estimation for strength for elements 146 and 95 for the
earthquake excitations ............................................................................................ 88
Table 7.5 Summary of $P_f$ for the wave loading ...................................................... 91
Table 7.6 Summary of $P_f$ for the earthquake loading ............................................. 91
ABSTRACT

Uncertainty management of complex dynamic structural systems vibrating in different media is addressed to design safer and more damage-tolerant structures. The dynamic properties (mass, damping, frequencies, etc.) of a structure vibrating in air (onshore) and in water (offshore) are expected to be different. Frequency contents of dynamic loadings (earthquakes and wave) are also expected to be different and unpredictable. The estimation of structural behavior with altered dynamic properties and uncertainty-filled loadings can be challenging. One of the main crucial loadings for offshore structures (OFS) is wave loading. The wave loading model is realistically developed to satisfy the underlying physics. The uncertainty management is carried out by estimating the underlying risk. A novel risk assessment procedure to estimate the underlying risk is proposed considering all major sources of uncertainty and nonlinearity. It is based on the multiple deterministic analyses-based concept to address uncertainty related issues in the formulation, currently a major research trend in the profession. To satisfy the underlying physics, the structures are represented by FEs. For wider acceptance, the dynamic loadings are applied in time domain. A novel risk evaluation concept using a surrogate metamodel Kriging technique is proposed. The implicit performance functions (PFs) are expressed explicitly using the significantly improved Kriging-based surrogate modeling technique. The proposed method consists of the response surface (RS) concept significantly modified for the structural reliability analyses and several advanced factorial schemes producing compounding beneficial effects. The risks corresponding to the serviceability-related global and strength related local PFs are evaluated. The method is clarified with the help of an informative example by estimating risks for serviceability and strength PFs of a large jacket-type OFS.
To compare the performance of structure in wave loading and seismic loading, a site-specific seismic safety assessment method for nonlinear structural systems is used to properly generate a suite of ground excitation time histories. The information on risk for both PFs and both loadings was extracted using about a couple of hundreds deterministic evaluations. The results were verified using thousands of Monte Carlo simulations (MCSs). The authors believe that they proposed an alternative to the basic MCS technique and a novel risk evaluation procedure for OFSs. The proposed uncertainty management concept appears to be exciting.
CHAPTER 1

INTRODUCTION

1.1 Statement of the Problem

Estimation of the underlying risk or probability of failure of structures has become an integral part of engineering analysis and design. The information is also essential to compare design alternatives. Furthermore, most design guidelines, at least in the U.S., have been modified to incorporate the risk-based design concept. The basic process enables the engineering community to incorporate major sources of uncertainty in the formulations resulting in optimal and safe design. However, any literature review will indicate that these design guidelines were essentially developed mostly for the static application of the loadings. Estimation of structural responses for the dynamic application of the loadings in the presence of uncertainty and major sources of nonlinearity, especially applied in time-domain, can be very challenging, and the state-of-the-art is evolving. Offshore structures (OFSs) excited by wave loadings will fall under this class of scenario. The most accurate wave for evaluating a structure is a nonlinear time history analysis. This is an important knowledge gap in implementing any risk-based design concepts for dynamic loadings applied in time-domain in the presence of nonlinearity. The research team at the University of Arizona identified this knowledge gap. The team proposed several sophisticated techniques for seismic loading applied in time-domain. These concepts need to be extended for OFSs excited by the wave loading. Jacket-type OFSs excited by the wave loading applied in time-domain in the presence of major sources of uncertainty and nonlinearity will be specifically addressed in this study.
1.2 Motivation of the Proposed Work

Engineered structures fail from time to time due to the inability of the profession to appropriately incorporate the presence of uncertainties in the problem, including statistical, model, data, and environmental conditions. Structures may fail also due to the inability of the engineers to predict the failure modes and the corresponding loading conditions, construction of structures not as designed, degradation of structures with time, the critical loading conditions and load combination. The problems get amplified if the dynamic loading needs to be applied in time domain, as required in most sophisticated deterministic analyses. As mentioned earlier, the risk of failure increases significantly under dynamic excitations caused by waves and earthquakes. Maximizing the safety and minimizing the cost are the major objective of engineering designs. OFSs can be considered as a special class of structures. Analysis and design of them are not very common. Structural shapes used to build them and the wave loading that acts on them are not widely known to the profession. It is reasonable to assume that the uncertainty in the structure will remain the same as in onshore structures (ONSs) the OFSs. However, the quantification of uncertainty in the wave loading is complicated and the state-of-the-art may not be well developed yet. The research team identified considerable knowledge gap in this area and plan to allocate a considerable amount of time and effort to quantify uncertainty in the wave loading. As will be discussed in more detail in the literature review section that the classical random vibration approach will be inapplicable for the time domain approach. Since the nature of uncertainty is not well understood, the basic Monte Carlo Simulation (MCS) is also not possible. The use of MCS for dynamic loadings applied in time domain may not be
practical, as will be discussed later. A new reliability estimation method is necessary for OFSs and this is the major objective of this study.

1.3 Objectives of the Research

This study aims to develop an efficient and accurate risk evaluation technique to study the stochastic behavior of complex nonlinear dynamical engineering systems, specifically for jacket-type OFSs excited by the wave loading.

1.3.1 Specific Objectives

The specific goals of this study are:

Objective 1: Identify major sources of uncertainty in OFSs.

Objective 2: Quantify uncertainty in the wave loading

Objective 3: Develop a risk evaluation technique for implicit limit state functions

Objective 4: Improve the efficiency of the method proposed in Objective 3

Objective 5: Increase the accuracy of the Proposed Method in Objective 3

Objective 6: Verifying the proposed models

Objective 7: Estimate the reliability of three-dimensional (3D) OFSs

Objective 8: Compare the reliability of OFSs against wave loading and seismic loading using the proposed method

1.4 Organization

This dissertation comprises eight chapters, including an introduction in chapter 1. In chapter 2, jacket-type OFSs are briefly reviewed, and the governing equation and the forces on OFSs are explained. Additionally, the conventional method for calculating the
hydrodynamic loading on an OFS is discussed. Afterwards, New Wave (NW) is introduced which is new technique for modeling the surface of the water. In summary, wave loading in detail and major parameters need to represent it are explained in this chapter.

Chapter 3 contains an introduction and a literature review on the available reliability evaluation approaches. This chapter contains the problem statement, the background of reliability evaluation methods of structural systems, the issues to be addressed and the knowledge gap. Also, it discusses the research done at The University of Arizona. This chapter explains the deterministic community expectations and the limitations of the currently available reliability evaluation methods for dynamic loadings applied in time domain for nonlinear complex structural systems.

Uncertainty quantifications of OFSs are addressed in chapter 4. This chapter represents a method which can incorporate the uncertainty in the frequency contents of the wave loading. It also formulates the procedure to generate the 3D water profile in time domain. Another important factor is the intensity of the waves. Uncertainty in the wave height is also addressed in this chapter. Furthermore, the major sources of uncertainty in OFSs are identified. In the last sections of this chapter, seismic loading, response spectrum-based analysis technique, and earthquake record selection criteria are explained. After defining wave and seismic loadings and the uncertainty in them, they are compared.

Chapter 5 develops a novel risk evaluation technique denoted hereafter as All Inclusive Response Surface - Modified Universal Kriging - First Order Reliability Method (AIRS-MUK-FORM). It is developed by combining several intelligent sampling schemes, surrogate modeling technics, significantly modified RS method, and FORM. This chapter explains all the aforementioned method in details, including Kriging and Variogram.
Furthermore, it explains why using UK is better than polynomial regression, and how choosing the sample points intelligently reduce the time consumption of the problem without compromising the accuracy.

The proposed methodology is required to be verified. Chapter 6 verifies the proposed concept for wave loading and generates benchmark values for comparison. A 3D jacket-type OFS already designed following conventional procedure is considered further for the reliability analysis. Considering major sources of uncertainty and using AIRS-MUK-FORM, the reliability of the structure against wave loading is evaluated using different criteria. The results are compared with MCS technique to show the accuracy and efficiency of the model.

The reliability of the same structure is estimated when excited by the seismic loading in Chapter 7. The results are compared with the wave loading. One of the main applications of the proposed methodology is to compare different alternatives in design or reliability of designs in different environments. This chapter compares the failure of a structure for both seismic and wave loadings in a region and discusses the corresponding implications. This chapter shows which loading is more critical and causes more uncertainty.

Chapter 8 summarizes the conclusions, contributions and findings achieved from the research. It explains the importance of using this method. The last section includes some recommendations for future work.
CHAPTER 2

JACKET-TYPE OFFSHORE STRUCTURES – A BRIEF INTRODUCTION

2.1 Introduction

OFSs are being built in increasing numbers to satisfy our needs for energy. It is expected that they need to be built according to the same standards used to build ONSs. At present, at least in the U.S., most ONSs are designed to satisfy some underlying risk. It is essential that OFSs are also designed according to risk-based criteria. However, they need to be designed for wave loadings. Since wave loading is not commonly used by practicing engineers, modeling them can be very challenging.

These types of platforms have been popular for extracting oil at shallow waters because of their simplicity of design and manufacturing. A jacket-type OFS is a tubular steel frame structure that extends from the mud line to above the mean water level (MWL). At the top, it supports a deck is usually modeled as rigid structure and the mass is considered to be lumped at the deck level. At the bottom, it is secured by tubular piles through its legs. A typical jacket-type OFSs is shown in Figure 2.1.

The equation of motion for OFSs is almost the same as that of ONSs. However, damping is expected to be different since the elements vibrate in water, and this movement of elements in water dissipates the energy in the form of hydro-dynamic damping. Furthermore, for wave loading, the forces are exerted to all elements based on their surface areas, wave height and the depth of the elements in water.
Figure 2.1 A typical jacket-type OFSs
Without losing any generality, the methodology used in this study is developed for commonly used jacket-type OFSs. They are structural frames mainly fabricated with tube sections. However, incorporating the wave loading, lack of familiarity of modeling it, and then formulating and incorporating the uncertainty in it are some of the major challenges. This is an important knowledge gap in implementing the dynamic wave loading applied in time domain in the presence of various sources of nonlinearity. It is believed that there are rooms for improvements, and they are presented in the following sections.

The governing equation for dynamic structural systems in the matrix form can be expressed as:

$$2\mathbf{M} \ddot{\mathbf{X}} + \mathbf{C} \dot{\mathbf{X}} + \mathbf{K} \mathbf{X} = \mathbf{F}$$  \hspace{1cm} (2.1)

where \(\mathbf{M}\), \(\mathbf{C}\), and \(\mathbf{K}\) are the mass, damping and stiffness matrices, respectively, \(\ddot{\mathbf{X}}, \dot{\mathbf{X}},\) and \(\mathbf{X}\) are the acceleration, velocity, and displacement vectors, respectively, and \(\mathbf{F}\) is the external force vector. The important differences in the governing equation between OFSs and ONSs are the mass matrix \(\mathbf{M}\), the damping matrix \(\mathbf{C}\), and the external force vector \(\mathbf{F}\). In OFSs, \(\mathbf{M}\) consists of the mass of the structure and the added mass caused by the motion of members in water. The added mass is generally estimated as \(\rho (C_m - 1)V\), where \(\rho\) is the mass density of water, \(C_m\) is the inertia coefficient, and \(V\) is the effective volume of the member in water. Damping also is more complicated. It contains the structural damping and the fluid damping. The fluid damping is caused by the attenuation of movements due to the difference between the velocity of water particles and the structure. The fluid damping is generally estimated as \(\rho A U C_d\), where \(A\) is the effective projected area of a structural element, \(U\) is the velocity of water particles, and \(C_d\) is the drag coefficient.
2.2 Morison Equation

A conceptual 3D wave loading is shown in Figure 2.2. As can be observed in this figure, the modeling of the wave loading can be challenging. If the information on acceleration and velocity of water is available, using the Morison equation, $F_i$, the hydrodynamic force per unit length acting normal to the axis of a cylindrical member $i$, can be calculated as the summation of the drag and inertia forces. As suggested by American Petroleum Institute (API 2007), the Morison equation can be defined as:

$$2F_i = F_{i,d} + F_{i,I} \tag{2.2}$$

where $F_{i,d}$ and $F_{i,I}$ are the drag and the inertia forces, respectively, per unit length of the member $i$. They are evaluated as:

$$F_{i,d} = C_d \rho / 2 A |U| \tag{2.3}$$

and

$$F_{i,I} = C_m \rho V \frac{\delta U}{\delta t} \tag{2.4}$$

where $C_d$ is the drag coefficient, $\rho$ is the density of water, $A$ is the projected area per unit length, $V$ is displaced volume of the cylinder per unit length ($\pi D^2/4$ for circular cylinders), $D$ is the effective diameter of circular cylindrical member including marine growth, $U$ and $|U|$ are the component of the velocity vector (due to wave and/or current) of the water normal to the axis of the member and absolute value of $U$, respectively, $C_m$ is the inertia coefficient, and $\frac{\delta U}{\delta t}$ is the component of the local acceleration vector of the water normal to the axis of the member.

The evaluation of the Morison equation requires the information of the fluctuation of the fluctuation of the sea surface about the mean sea level, as a function of time. There are
several methods available in the literature for this purpose. Some of the classical deterministic methods for estimating the sea surface fluctuations are Airy, Stocks, and Cnoidal (Dawson 1983). However, none of these methods model the water surface elevation precisely. To address this problem, the NW theory was proposed. It is deterministic in nature and accounts for the spectral content of the sea (Tromans et al. 1991). The following section discusses how to generate the water surface elevation using the NW theory.

2.3 Wave Modeling

Because of the importance of the wave loading in the governing equation and estimating the probability of failure, it should be accurately modeled the wave load. In this regard, two important factors should be addressed in wave loading: (1) the intensity of the wave which is generally expressed by wave height and (2) the real shape of the wave profile. These two features are explained by significant wave height and mean zero-up crossing period, which are two important metoceanic parameters.

Significant wave height, $H_s$, is the average of the upper third of the wave heights. Zero-up-crossing period, $T_z$, is defined as the average value of the time between successive up-crossings of the still water level. In other words, significant wave height and Zero-up-crossing periods represent the intensity and frequency content of wave, respectively.

In order to simulate the elevation of the sea surface, many methods exist. Calculating the sea surface in wave loading is important because calculation of the wave force on the structure highly depends on the shape of the surface. The classical deterministic methods for calculating the sea surface includes Airy, Stocks, and Cnoidal, (Dawson 1983) which do not consider the randomness of the sea waves.
Figure 2.2 A conceptual water surface

2.3.1 Wave Modeling Using the New Wave Theory

The elevation of the sea surface elevation will significantly influence the estimation of the wave force acting on structures. NW theoretically simulates the most probable shape at an extreme event by assuming the surface elevation as a Gaussian random process. For the nonlinear dynamic analysis, the load history is also very important in modeling the sea surface and the information on it needs to be incorporated in the formulation. Modeling the time domain sea surface over a long period of time can be cumbersome. To overcome this problem, the NW theory can be used. It simulates many hours of time domain simulation of wave loading in a more computationally efficient way (Cassidy 1999; Cassidy et al. 2001, 2002; Tromans et al. 1991). Using the NW theory, the surface elevation, $\eta$, as a function of time can be expressed as:
\[ \eta(x, t) = \frac{\alpha}{\sigma^2} \sum_{n=1}^{N} [S(\omega_n) d\omega] \cos (k_n x - \omega_n t) \] (2.5)

where \( x \) is the location and \( x = 0 \) represents the wave crest, \( t \) is the time relative to the initial position of the crest, \( k_n \) is the wave number of the \( n^{th} \) component, \( \omega_n \) is the angular frequency of the \( n^{th} \) component, \( \alpha \) is the NW crest elevation, \( S_{\eta\eta}(\omega_n) d\omega \) is the surface elevation spectrum, and \( \sigma^2 \) is the variance corresponding to that wave spectrum, i.e., the area under the spectrum. The most commonly used surface elevation spectrum is Jonswap (Hasselmann et al. 1973). It is mathematically expressed as:

\[ S(\omega) = \bar{\alpha} g^2 \omega^{-5} e^{-\beta_j (\omega / \omega_p)^{-4}} \gamma^a \] (2.6)

where, \( \gamma \) is the peakedness parameter (it is 3.3 for Jonswap and 1 for Pearson-Moskowitz), \( \omega \) is the angular frequency, \( \omega_p \) is the peak frequency—the frequency at which the spectrum peaks, \( \beta_j \) is the shape factor and equals 1.25, \( g \) is the gravitational acceleration, and \( a \) and \( \bar{\alpha} \) are modified Phillips constants. They are defined as:

\[ a = e^{-(\frac{\omega - \omega_p}{2\sigma^2 \omega_p^2})^2} = e^{-(\frac{\omega / \omega_p - 1}{\sigma \sqrt{2}})^2} \] (2.7)

and

\[ \bar{\alpha} = 5.058 \left( \frac{H_s}{T_p^3} \right)^2 (1 - 0.287 \ln \gamma) \] (2.8)

where, \( H_s \) and \( T_p \) are the significant wave height and peak frequency (the period associated with the peak of the spectrum), respectively. Denoting \( T_z \) as the mean zero-crossing period, \( T_p \) can be calculated as (Cassidy 1999):

\[ T_p = T_z (0.327 \ e^{-0.315 \gamma + 1.17}) \] (2.9)

where \( \gamma \) is defined earlier. \( H_s \) and \( T_z \) are site specific and depend on the location of the structure.
2.3.2 Wheeler Stretching Effect

The motion of the water particles and the forces they generate on the structure are a function of depth \( z \), as shown in Figure 2.3. It can be calculated as (Journée and Massie 2001):

\[
F_n(z) = \frac{\cosh(k_n(d+z))}{\sinh(k_n d)}
\]  

(2.10)

where \( F_n \) is called the attenuation factor with respect sea level (Y-axis) for \( n \)th component. Figure 2.3 shows the velocity of the water particles in the Z-axis as a function of time (X-axis) and depth. In order to make these values valid above the mean water-level, \( F_n(z) \) in Equation 2.10 need to be modified by any one of the following methods: Wheeler stretching, linear extrapolation, or Delta stretching interpolation (Wheeler 1970). Wheeler stretching is considered in the study. Mathematically, it can be represented as:

\[
F_n(z) = \frac{\cosh\left(\frac{k_n(d+z)}{1+\eta/d}\right)}{\sinh(k_n d)}
\]

(2.11)

where \( d \) is the depth under consideration, \( \eta \) is the free water surface, and \( z \) is the variable that can change from the sea bed to \( \eta \). By using Wheeler stretching, the velocity of water particles in different depths can be calculated as shown in Figure 2.3.
2.3.3 Directionality

As can be observed in Figure 2.2, the wave loadings are expected to be different in the two horizontal directions; essentially a function of $\theta$, the direction of the wave. In order to address the directionality-related issue, the directional spectrum concept can be used. It can be mathematically represented as:
\[ S(\omega, \theta) = S(\omega) D(\theta) \] (2.12)

where, \( S(\omega) \) is the surface elevation spectrum, and \( D(\theta) \) is the directional spreading function. Many different functions to represent it were used by researchers (Soares 1998). The aim of them is to generate higher weights for the directions closer to the main direction.

According to API (2007), \( D(\theta) \) is represented by a trigonometric function as:
\[ D(\theta) = C_s \cos^s(\theta - \bar{\theta}) \] (2.13)

where, \( \bar{\theta} \) is the direction of the wave and usually considered as zero, exponent \( s \) can be either 2 or 4 based on the observations. Obviously, exponent 4 causes less spreading than 2. Data from wind-driven sea condition indicate that exponent 2 is the appropriate spreading function. Exponent 4 is suitable for situations where limited fetch restricts the degree of spread. The above discussions indicate that for higher value of \( s \), energy is more concentrated around the main wave direction. \( C_s \) is a normalizing coefficient that makes
\[
\int_{\bar{\theta} - \pi/2}^{\bar{\theta} + \pi/2} C_s \cos^s(\theta - \bar{\theta}) d\theta = 1.
\]

It can be shown that when \( s \) values are 2 and 4, the corresponding \( C_s \) will be \( \frac{\pi}{2} \) and \( \frac{3\pi}{8} \), respectively.
2.4 Summary

This chapter provided a detailed summary and novel techniques to model wave loading and forces that act on the structure. A typical wave loading in time domain is extremely irregular similar to earthquake time histories. However, the frequency contents of the two loadings are very different. Just like the earthquake loading, capturing the irregular behavior of the wave loading mathematically can be very challenging. The most important factors in modeling wave loading are the profile of the water level surface and the wave height. Initially NW theory was proposed to model sea surface fluctuations as a function of time. It is deterministic in nature and accounts for the spectral content of the sea. To address randomness in it, the CNW theory can be used. Using the CNW theory, many hours of random wave loading in time domain can be simulated in a computationally efficient manner.
CHAPTER 3

LITERATURE REVIEW OF RELIABILITY ESTIMATION OF OFFSHORE STRUCTURES

3.1 Introduction

Even though, there are a lot of research topics and guidelines for ONS performance-based design, available literature will indicate that the risk-based design guidelines for OFSs are very limited. Det Norske Veritas (DNV) provides few initial guidelines for the reliability analysis of OFSs (Sigurdsson et al. 1996; Sigurdsson and Cramer 1996; Skjong et al. 1995). However, the scopes of their suggestions are very limited; they did not specifically address jacket-type OFSs. OFSs are needed to be designed using the reliability-based concept considering all major sources of uncertainty (Bargi et al. 2011). Uncertainty management of complex nonlinear dynamic engineering systems vibrating in different media (air or fluid) is expected to be very challenging. As mentioned in previous chapters, Dynamic engineering systems are generally assumed to be vibrating in air; i.e., they are ONSs. OFSs are increasingly used to address energy-related issues and to maintain our way of living. Reliability-based design of ONSs has become very common in all over the world to address major sources of uncertainty. OFSs are not very common and visible, and their engineering design has not kept up with similar improvements. It is essential that OFSs are also designed similar to ONSs. This is very important since the failure of OFSs will have disastrous consequences not only economically but also environmentally.
There are few studies where several reliability concepts including the First Order Reliability Method (FORM) (Leimeister and Kolios 2018), Inverse FORM (IFORM) (Vanem 2019), Second Moment Reliability Method (SORM) (De 1990), etc., were used for the design of Jacket-type OFSs and wind turbine structures (Kim and Lee 2015). There are several deficiencies in these studies. These studies failed to incorporate major sources of nonlinearities expected in the system just before failure. The dynamic wave loading was not realistically applied in time domain (2014). In addition, these methods failed to incorporate the information on the structural frequencies appropriately. There are few studies which attempted to use the Pushover analysis to estimate the design capacity, then, correlated it to the Reserve Strength Ratio (RSR) and probability of failure to define a form of reliability index without considering uncertainty in the frequency content of the seismic loading. (Asgarian and Agheshlui 2009; Fayazi and Aghakouchak 2015; Soom et al. 2018). De et al. (1991) estimated the probability of failure of jacket-type structures using the wave fragility analysis and the PDF of the extreme wave height. Nordal et al. (1987) used FORM and branch-and-bound analysis technique to estimate the probability of failure.

It is noteworthy to point out that even though several studies were conducted using RSR and Pushover analyses, Golafshani et al. (Golafshani et al. 2011b; a) demonstrated that neither Pushover nor RSR is appropriate for the reliability estimation of jacket-type platform. To consider the presence of uncertainty, simulation-based approaches including MCS, Latin Hypercube Sampling (LHS), importance sampling, etc. were used by several researchers (Ajamy et al. 2014; Chian et al. 2018; Kolios et al. 2015; Wisudawan et al. 2017; Yang et al. 2015). However, they failed to use the physics-based analytical
formulation. It is also known to the profession that simulation-based procedures can be very cumbersome, computationally expensive, and may not be conclusive to all the concern parties. Discussions made here clearly indicates that the risk-based design of OFSs is very limited. The authors believe that it is a necessity to design OFSs using the same risk-based concept similar to ONSs. Unfortunately, there is no method currently available to estimate the reliability of OFSs explicitly considering all major sources of nonlinearity and excited by the dynamic wave loading applied in time domain. Because of their locations, some of the OFSs also required to be designed for the seismic loading. Their risks need to be compared with the wave loading to ensure adequate and uniform levels of safety. Since the frequency contents of wave and seismic loadings are different and the dynamic properties of the same structure in air and submerged conditions are very different, the dynamic response behavior under both conditions are expected to be significantly different. Moreover, the selections of design wave and seismic loadings are still evolving; it can be very error-prone and full of uncertainty. Some of the major objectives of this study are to fill these knowledge gaps.

3.2 Research Done on Reliability Analysis at The University of Arizona

FORM proves useful and are relatively simple when the Limit State Function (LSF) is explicit. When the structures are represented by FEs, they can still be implemented using the stochastic finite element method (SFEM), as suggested by Haldar and Mahadevan (2000a). It cannot be used for the class of nonlinear dynamic problem.

The research team at the University of Arizona has been working on developing a risk evaluation procedure for dynamic loading applied in time domain (Gaxiola-Camacho et al. 2017). They concluded that any such risk or reliability analysis procedure should be finite
element based to consider all major sources of nonlinearity to satisfy the underlying physics as realistically as possible, as is commonly practiced by the deterministic community. Then, the seismic loading must be applied in time domain. The information on risk extracted in this way will be acceptable to the deterministic community. This is very important since they are the final decision makers in most cases (Huh and Haldar 2002; Lee and Haldar 2003).

Recently, the team attempted to address this issue in a more comprehensive way. In developing the Performance Based Seismic Design (PBSD) concept, they proposed a new concept by integrating the response surface method (RSM), and advanced factorial concept (AFC) (Gaxiola-Camacho et al. 2017, 2018b). The method was also used to study the performance of steel frames in the presence of flexibility of connections (Gaxiola-Camacho et al. 2018a; Villegas Mercado et al. 2017). Different factorial designs were compared with MCS to assess the underlying risk against seismic excitation (Azizsoltani and Haldar 2017b). The method has the potential to estimate the reliability of any complex systems under dynamic loading conditions. The team developed the method essentially for the seismic loading. However, they documented that it can also be used to study thermo-mechanical loading generated in solder balls in electronic packing (Azizsoltani and Haldar 2018).

However, there is no research on reliability of offshore structures in nonlinear zone and time domain. Also, the uncertainty in the wave loading has never been properly addressed. Also, having a robust reliability method, an offshore structure can be evaluated for various conditions. Apart from wave loading, seismic loading might be reason of
failure. An efficient and accurate reliability method can compare and contrast the probability of failure of failure against the aforementioned critical loadings. This research aims to fill this knowledge gap.

3.3 Limitations in Current Reliability Methods for Loading in Time-Domain

Generally, the engineering profession uses FE formulations to study the seismic behavior of structures as realistically as possible. The most rigorous FE analysis requires the proper application of dynamic loadings in time domain. The phrase “probability of failure” or “probability of not satisfying a performance requirement” implies that the risk needs to be evaluated just before failure in the presence of several sources of nonlinearities. Generally, three conditions must be taken into consideration for the proper calculation of probability of failure: (1) structures must be represented by FEps, (2) wave loading must be applied in time domain, and (3) major sources of nonlinearity and uncertainty must be considered.

The dynamic behavior of structures can be represented by LSFs, which are expressed in terms of Random Variables (RVs). The LSFs depend on prescribed performance requirements. As previously discussed, FORM can be used for seismic risk evaluation if LSFs are explicitly available. However, for nonlinear seismic loadings applied in time-domain, LSFs are expected to be implicit and a function of time. For implicit LSFs, some of the reliability evaluation methods that can be used are: MCS, RSM, and sensitivity-based methods (Haldar and Mahadevan 2000a). Using the sensitivity-based iterative perturbation method and FORM, the Stochastic Finite Element Method (SFEM) was developed by Haldar and Mahadevan (Haldar and Mahadevan 2000b).
At the first glance, the easier way to address this problem is MCS to estimate the probability of failure of systems. This method has no limitations in terms of mathematical definition of LSF as well as dynamic loadings in time-domain. Apart from this conventional approach, importance sampling, LHS (Beachkofski and Grandhi 2002), etc. can be used to estimate to reliability index. However, these approaches need a great number of deterministic simulations. In other words, although using a decent number of simulations brings up rather precise results, it is very time consuming for complex problems, in particular for dynamic loading. The probability of failure of many engineering systems is expected to be relatively small, perhaps between $10^{-3}$ to $10^{-7}$. To extract reliability information to estimate the probability of failure of $10^{-3}$, at least 10,000 simulations are necessary (Haldar and Mahadevan 2000a). Yet even 1,000 simulations may take over 10,000 hours or 1.14 years of uninterrupted running of a computer and the results may not be acceptable because of using such a small number of simulations. In other words, there is there is knowledge gap to calculate the probability of failure significantly fewer samples, tens or hundreds.

Another issue in implementing the proposed approach is how to define LSFs. As discussed earlier, defining an explicit LSF is virtually impossible for this class of problems. The most well-known method to define an LSF is RSM. RSM can be defined by various methods, and the most famous one is a polynomial most commonly polynomial is used expresses it. For the success of this approach, the polynomial needs to be defined in the failure region and around the design point. Depends on the number of the sample points, either linear or quadratic polynomial can be used. After representing an implicit LSF approximately by a RS function, FORM analysis can be applied to extract the reliability
index $\beta$. To achieve a better result, LSF can be updated at new design point during the iterations. Many various RSM-based techniques are available, including artificial neural networks (Hurtado and Alvarez 2001; Vazirizade et al. 2019a) (Hurtado and Alvarez 2001), High Dimensional Model Representation (HDMR) (Chowdhury et al. 2009) and Enhanced High Dimensional Model Representation (EHDMR) (Rao and Chowdhury 2009), several surrogate approaches including kriging (Bichon et al. 2008), Support Vector Machines-based explicit design space decomposition (Basudhar and Missoum 2008), univariate variate decomposition (Wei and Rahman 2007), etc. The other important topic is defining a decent algorithm for sampling points, which is called sampling scheme. This step is very crucial because LSF is actually based on the results of the sampling points, and the main goal is to form a representative LSF using the least number of sample points. In other words, a good selection of the sample results in generating a more accurate RS function in the failure region. In this regard, due to assorted types of limitations, including nonlinearity and loading in time-domain, sampling points should be chosen intelligently.

3.4 Summary

The above discussions clearly indicate that there is room for improvements in modeling uncertainty in the wave loading. This is an important knowledge gap in implementing dynamic wave loading applied in time-domain in the presence of nonlinearity. Although SFEM can be employed to evaluate the risk for both explicit and implicit LSF, it is not very efficient for complex nonlinear time-domain dynamic problems. In this regard, an alternative robust approach to SFEM is necessary for everyday use. The alternative should incorporate all the source of uncertainty. Additionally, it should be
efficient and accurate. In the following chapters, uncertainty quantification of important parameters and the proposed method will be discussed.
CHAPTER 4
UNCERTAINTY QUANTIFICATIONS OF OFFSHORE STRUCTURES

4.1 Introduction

The appropriate uncertainty quantification of design variable for OFSs and ONSs will be the first step will be the first step in the reliability estimation of them. Dynamic properties are expected to be very different if the systems are considered to be vibrating in air and water. Since the frequency contents of seismic and wave loadings are different and they significantly influence the dynamic behavior, the changes in the dynamic properties of ONSs and OFSs, are expected to compound the response estimation. Moreover, For OFSs, in addition to the seismic loading, the wave loading can also be very critical. The selection of design seismic and wave loadings can be very error-prone and full of uncertainty. The uncertainty in the intensity and frequency contents of the loadings are very important in nonlinear time history analysis.

4.2 Uncertainty in Structures

As discussed earlier, ONSs and OFSs are very similar in terms of general structural arrangements. The presence, types, and amounts of uncertainty in them are expected to be similar. The information on uncertainty can be collected by experiments or from the
available literature. Main RVs in jacket-type steel structures considered in this study are modulus of elasticity, yield stress of steel, strain-hardening ratio, areas and moment of inertias of the members, etc. The information on uncertainty in them is widely reported in the literature (Ellingwood 1980). Furthermore, because of nonlinearity in the problem, the response of system to the environmental forces in the presence of uncertainty is expected to be very complicated.

4.3 Uncertainty in Wave Loadings

The NW concept discussed in the previous chapter, does not consider the randomness of the sea waves. The CNW concept can be used to consider randomness in the wave loading force (Mirzadeh 2015). With this technique, the water surface is simulated in time domain. However, numerous different wave surfaces can be generated for the required wave state. This approach is an alternative to widely used regular wave theories, such as Airy and Stokes’ 5th-order wave (Cassidy 1999; Cassidy et al. 2001, 2002; Tromans et al. 1991). Using the CNW theory, many hours of random wave loading in time domain can be simulated in a more computationally efficient manner. Actually, Figure 2.2 is generated using this technique. Using the NW concept for a deterministic wave of predetermined height accounting for the spectral composition of the sea, CNW adds uncertainty in the formulation. Mathematically, the basic CNW concept can be expressed as:

$$\eta_c(x,t) = \eta_r(t) + r(x,t)[\alpha - \sum_{n=1}^{N} a_n] + \left(\frac{r(x,t)}{\lambda^2}\right)[\dot{\alpha} - \sum_{n=1}^{N} \omega_n b_n]$$

(4.1)

where $\eta_r(t)$ is the random surface elevation above the mean water level. $r(x,t)$ is the unit NW, which is the normalized water surface level, $\alpha$ is a predetermined constrained amplitude of the wave, $\sum_{n=1}^{N} a_n$ represents the random surface elevation at $t = 0$ [or $\eta_r(0)$],
\(-\dot{r}(x, t)\) is the slope of the unit NW, \(\lambda^2\) is obtained from the second spectral moment of the wave energy spectrum \((\lambda^2 \sigma^2)\), \(\dot{\alpha}\) is the predetermined constrained slope; for a crest, \(\dot{\alpha} = 0\), and \(\sum_{n=1}^{N} \omega_n b_n\) is the random surface slope at \(t = 0\) or \(\dot{\eta}_r(0)\).

The random surface elevation above the mean water-level for a uni-directional wave is (Mirzadeh 2015):

\[
\dot{\eta}_r(t) = \sum_{n=1}^{N}(a_n \cos(\omega_n t) + b_n \sin(\omega_n t))
\]  

(4.2)

where \(\omega_n\) is the angular frequency of \(n^{th}\) component as discussed earlier, \(a_n = \sqrt{S(\omega_n)}d\omega\) and \(b_n = \sqrt{S(\omega_n)}d\omega\); \(a_n\) and \(b_n\) are Fourier components, which are Gaussian random variables with zero mean and a standard deviation related to the wave energy spectrum at the corresponding discrete frequency which is \(\sigma_n = \sqrt{S(\omega_n)}d\omega\).

Figure 4.1 shows the water surfaces generated by the NW and CNW theories; \(H_s, T_z\), and \(\alpha\) are assumed to be 12 m, 10 s, and 15 m, respectively, and \(x = 0\), at the location of the maximum crest. It can be observed that CNW is adding uncertainty in NW. It is to be noted that the water surface elevations will be different according to the CNW for the same set of parameters indicating uncertainty in them.
To clarify the CNW theory further, the three components of CNW denoted hereafter as CNW1, CNW2, and CNW3, corresponding to the terms \( \eta_r(t) \), \( r(x, t) [\alpha - \sum_{n=1}^{N} a_n] \), and \( \frac{-r(x, t)}{\lambda^2}[\dot{\alpha} - \sum_{n=1}^{N} \omega_n b_n] \), respectively, are considered separately. They are plotted in Figure 4.2. CNW2 represents the scaled water surface elevation according to NW. The plots clearly indicate the uncertainty in NW.

![Figure 4.2 CNW and its components generated by our script](image)

### 4.4 3D CNW Concept

In order to convert CNW from 2D to 3D, the directionality effect should be considered. CNW generates water surface elevation considering uncertainty in the shape of the waves including the wave height and the frequency content. However, it does not include the directionality effects. To incorporate information on directionality in the formulation, 3D CNW is proposed (Mirzadeh et al. 2016). This is accomplished by adding another direction \( Y \). In other words, directionality consider the randomness in the two horizontal directions. The crest height changes in the perpendicular direction of the wave motion. To include the effects of wave directionality, \( \theta \) is introduced. This 3D CNW can be generated by a finite number \((M \times N)\) of sinusoidal wave components, where \( M \) is the number of
wave directions and $N$ is the number of frequency components. Thus, 3D CNW can be defined as:

$$
\eta_c(x, y, t) =

\eta_r(t) + r(x, y, t) \left[ \alpha - \sum_{m=1}^{M} \sum_{n=1}^{N} a_{m,n} \right] + \left( -\frac{\dot{r}(x, y, t)}{\lambda^2} \right) \left[ \dot{\alpha} - \sum_{m=1}^{M} \sum_{n=1}^{N} \omega_n b_{m,n} \right]
$$

(4.3)

where $\eta_r(t)$ is defined below, $r(x, y, t)$ is as defined in before, $\alpha$, $\dot{\alpha}$ and $\lambda^2$ are defined earlier, $\sum_{m=1}^{M} \sum_{n=1}^{N} a_{m,n}$ is the random surface elevation at $t = 0$ (or $\eta_r(0)$), $-\dot{r}(x, y, t)$ is the slope of the unit NW, is obtained from the second spectral moment of the wave energy spectrum ($m_2 = \lambda^2 \sigma^2$), $\dot{\alpha}$ represents the predetermined constrained slope; for a crest, $\dot{\alpha} = 0$, $\sum_{m=1}^{M} \sum_{n=1}^{N} \omega_n b_{m,n}$ is the random surface slope at $t = 0$ or $\eta_r(0)$. This is the most comprehensive equation, which includes directionality, randomness and frequency content. For $M = 1$, it will be 2D instead of 3D waves. The random surface elevation for the 3D waves as a function of $t$ can be defined as follows:

$$
\eta_r(t) = \sum_{m=1}^{M} \sum_{n=1}^{N} (a_{m,n} \cos(\omega_n t) + b_{m,n} \sin(\omega_n t))
$$

(4.4)

where $a_{m,n} = rn_{an} \sqrt{S(\omega_n, \theta_m)} d\omega d\theta$ and $b_{m,n} = rn_{bn} \sqrt{S(\omega_n, \theta_m)} d\omega d\theta$.

The 3D effect of water surface level shown in Figure 2.2 can now be represented mathematically with the help of the proposed 3D CNW. Suppose, two legs of an OFS are separated by a distance. They are expected to be subjected to two different wave heights.

Figure 4.3 depicts the difference between water surface elevation at upwave and downwave legs of a jacket-type OFS. In this figure the parameters are are $H_s = 12$ and $T_z = 10$. Furthermore, it shows the difference between NW and CNW. In CNW, shape of the wave changes during the time; therefore, the wave produces different forces at upwave and downwave legs.
4.5 Uncertainty in the Wave Height Estimation

To estimate the uncertainty in intensity of the wave, a distribution for the wave height in the location of the structured is required. It is very common among the profession to estimate the uncertainty in intensity of the wave by the joint distribution of $H_s$ significant wave height and zero-up-crossing period $T_z$ (Eik and Nygaard 2003; Haver and Winterstein 2009). Figure 4.4 shows the joint distribution of $H_s$ significant wave height and zero-up-crossing period $T_z$ (Vazirizade et al. 2019b).

Lognormal distribution was used for conditional distribution of $T_z$ and Weibull for marginal distribution of marginal distribution of $H_s$ together (Bitner-Gregersen et al. 1989). Furthermore, the same approach was used and concluded that this is the prime candidate among Bivariate log-normal distribution, Fang and Hogben's distribution,
Bivariate Weibull distribution, and Marginal Weibull and conditional log-normal distribution (Mathisen and Bitner-Gregersen 1990). Bitner-Gregersen, Cramer, & Løseth (1995) asserted that this model not only works for the North Sea and Norwegian Sea but also for global wave conditions. This joint model, a combination of a conditional lognormal and a marginal Weibull distribution used as reliable modeling technique for modeling wind-sea and swell. Furthermore, they used data for different locations and climates to show uncertainties of data related to the proposed fits. Now, using Weibull distribution for the significant wave height is very common among the proficient, although calculating the parameters is a challenge.

Figure 4.4 The joint distribution of Hs significant wave height and zero-up-crossing period Tz
The uncertainty in the significant wave height, $H_s$, is considered to be described by the 2-Parameter Weibull distribution (DNV-GL 2014). The probability density function (PDF) and the cumulative distribution function (CDF) of it can be shown to be:

$$f_{H_s}(h_s) = \frac{\beta_w (h_s)^{\beta_w - 1}}{\alpha_w^{\beta_w}} \cdot \exp \left\{ - \left( \frac{h_s}{\alpha_w} \right)^{\beta_w} \right\} \quad (4.5)$$

and

$$F_{H_s}(h_s) = 1 - \exp \left\{ - \left( \frac{h_s}{\alpha_w} \right)^{\beta_w} \right\} \quad (4.6)$$

where $\alpha, \beta$ are calculated based on the observations at a location being considered (DNV-GL 2014). To estimate the uncertainty in the wave height $H$ based on the uncertainty in the significant wave height $H_s$ for a region, Forristall and Cooper (Forristall and Cooper 1997) suggested a conditional CDF of $H$ on $H_s$. It is generally referred in the literature as the Forristall wave height distribution. It can be mathematically expressed as:

$$F_{H|H_s}(h|h_s) = 1 - \exp \left( -2.26 \left( \frac{h}{h_s} \right)^{2.13} \right) \quad (4.7)$$

The unconditional CDF of $H$ can be derived:

$$F_H(h) = \int_{h_s} f_{H|H_s}(h|h_s) \times f_{H_s}(h_s) \cdot dh_s \quad (4.8)$$

### 4.6 Uncertainty in Seismic Loadings

For OFSs, excitation caused by the seismic loading can be very critical (Vazirizade et al. 2017). OFSs are needed to be designed considering all major sources of uncertainty not only considering the wave but also the seismic loading (Bargi et al. 2011). Seismic loading is generally considered as the most unpredictable natural loading. It depends on many features including the seismic activity of the region, local soil conditions (Haldar et al. 2019; Seyyed Alangi et al. 2018), types of structures to be built, etc. Some of them are
beyond the control of a designer. The seismic design guidelines changed, both conceptually and analytically, on a regular basis (American Society of Civil Engineers (ASCE) 2010; FEMA-222A 1995; FEMA-223A 1995; FEMA-302 1997; FEMA-303 1997; ISO-19901-2 2017; UBC 1997). At present, the most sophisticated deterministic seismic analysis requires that it must be applied in time domain. It is extremely irregular. A typical time history recorded at the Imperial Valley in 1940 El Centro station is shown in Figure 4.5 (Vazirizade and Bakhshi 2015). The figure clearly indicates that the acceleration level and the frequency contents are expected to be highly unpredictable. To address the related issues, the current design requirements require the spectrum concept and at least 11 earthquake time histories need to be considered fitting the design spectrum ASCE (2016).

Generation of multiple earthquake time histories satisfying a design acceleration spectrum is a multi-faceted challenging problem. The authors and their members addressed the related issues using a research grants from NSF (Azizsoltani et al. 2018; Azizsoltani and Haldar 2017a; b, 2018; Gaxiola-Camacho et al. 2017, 2018a; b, 2020; Gaxiola Camacho and Haldar 2017; Haldar et al. 2020, 2019; Vazirizade et al. 2020a; b; Villegas Mercado et al. 2017)

The basic concept is briefly summarized here due to lack of space. It is essentially based on generating a site-specific ground motion spectrum. The necessary steps required to implement the concept are conceptually discussed in ASCE/SEI 7-16 ASCE (2016). The design spectrum can be generated by taking the product of the risk coefficient, $C_R$, and the Uniform Hazard Response Spectrum (UHRS) (Loth and Baker 2015). UHRS is defined as the spectral response acceleration from a 5% damped acceleration response spectrum having a 2% probability of exceedance within 50 years. The authors decided to use the site-
specific hazard curves approach to generate it. Hazard curves are plots of the annual frequency of exceedance versus the ground motion acceleration for the different periods of a structure. The web application-based hazard curves developed by United States Geological Survey (USGS) for different locations in the U.S. are used in this study. The estimation of the appropriate $C_R$ values is suggested in ASCE/SEI ASCE (2016).

Figure 4.5 Time history of 1940 Imperial Valley earthquake at El Centro station

With the availability of the site-specific design spectrum, it is necessary to generate a suite of ground motion time histories fitting it as suggested by Beyer and Bommer (2007). Conceptually, a suite of earthquake time histories fitting the spectrum, can be generated in several ways. Scaling past recorded time histories available at Pacific Earthquake Engineering Research Center (PEER) database, a suite of time histories can be generated. They can also be generated numerically using Broad Band Platform (BBP) (Gaxiola
Camacho and Haldar 2017). In this study, the first alternative is used to generate the time histories.

4.7 Wave and Seismic Loadings – Comparisons

Even though both wave loading and earthquake loading fall into the category of dynamic lateral loading, they act very differently. The wave force is applied to all members which are below the water surface level. This force is usually higher closer to the water surface and attenuated along the depth. It also needs to be pointed out that in some areas, wave loading will be more critical than seismic loading and vice versa.

The quantifications of uncertainty in the seismic and wave loadings will be very different. The uncertainty and mathematical modeling of the waves were discussed before. The water sea level can be extremely irregular similar to earthquake time histories. However, the frequency contents of the two loadings are very different. Using the CNW theory, many hours of random wave loading in time domain can be simulated in a computationally efficient manner. As discussed before, in order to address the uncertainty in seismic loading, various time history records are scaled in a fashion that their average fit the designed spectrum. This approach can be applied to wave loading as well. By using CNW, unlimited number of water level surface in time domain can be generated, which have different profile and frequency content while sharing the same meteoceanic properties. Therefore, various wave loading can be applied on structure to evaluate the behavior of the structure in different conditions.
CHAPTER 5

RELIABILITY EVALUATION OF OFFSHORE STRUCTURES – A NOVEL SURROGATE MODEL

5.1 Introduction

For the appropriate uncertainty management of nonlinear dynamic engineering systems vibrating in different media, the understanding of appropriate physics-based analytical governing equations of motion for them are necessary, as discussed before. Also, at this stage, uncertainty associated with all the resistance and load related design variables will be available either based on the discussions made in the previous sections or based on the information available in the literature for a typical jacket-type OFS. It is now necessary to estimate the underlying risk or reliability. Risk is always estimated with respect to a specific performance function (PF). A typical PF is expected to be a function of all RVs in the formulation and the required performance objective or limit specified in the design guidelines. They can be strength and/or serviceability related.

When a mathematical representation of a PF is explicit in nature, the first/second order reliability method (FORM/SORM) (Haldar and Mahadevan 2000a) can be used to estimate the underlying reliability. FORM is used in this study. For easier implementation of FORM, first derivatives of a PF with respect to all design variables need to be available. If they are not, their implementation can be very cumbersome or tedious, and may not even be practical for a problem with large number of RVs (Haldar and Mahadevan 2000b). If a
PF is implicit in nature, the reliability estimation can be challenging. As expected, for nonlinear dynamic problems loading applied in time domain, PFs are expected to be implicit. For implicit PFs, Haldar and Mahadevan (Haldar and Mahadevan 2000b) suggested the sensitivity-based stochastic finite element method (SFEM), simulation, and RSM for the reliability estimation. One of the authors developed the SFEM concept and now believes that an alternative approach is necessary for several reasons. The MCS-based formulation is possible but considering its inefficiency, it may require continuous running of a computer for several years, particularly for large nonlinear dynamic problems excited in time domain (Azizsoltani and Haldar 2018). Sophisticated space reduction techniques, parallel processing, and other mathematical techniques can be used but they may require expertise not expected from a typical practicing engineer for everyday use. Considering many alternatives, the authors decided to use a meta or surrogate model of RSM to approximately generate a PF in the failure region considering all major sources of nonlinearity and uncertainty satisfying the reliability community.

The basic RSM concept was developed to study chemical reaction (Box and Wilson 1951). It was developed in the coded variable space using information on mean and variance, completely ignoring the distribution information of RVs. For structural reliability estimation, the distribution information of all RVs must be explicitly incorporated in the formulation. In addition, an RS must be generated in the failure region which will be unknown for most structural engineering problems of practical interest. The discussions clearly indicate that a RS generated using the basic concept needs significant improvements before it can be used for the structural reliability evaluation.
Since FORM-based reliability estimation procedure iteratively locates the most probable failure point (MPFP) in the failure region considering distributional information of all RVs present in the formulation, the authors decided to integrate the RSM concept with FORM. The integration process is discussed next. However, to differentiate between a RS generated in the old and the proposed integrated ways for the structural reliability evaluation, it will be denoted hereafter as all-inclusive RS or AIRS. After generating a required AIRS, it can be modified to represent a PF by including the performance requirement, as will be discussed in Section 5.4. A PF generated this way will be denoted hereafter as AIRS-PF. A typical AIRS-PF will be explicit in nature and the classical FORM can be used to extract the reliability information.

5.2 Integration of RSM and FORM Producing AIRS

As briefly mentioned earlier, using the old RSM concept, a typical RV is expressed in the coded variable space ignoring the distribution information as (Azizsoltani and Haldar 2018):

\[ X_i = X_i^C + hx_i \sigma_{X_i}, \quad i = 1, 2, ..., k \] (5.1)

where \( k \) is the number of RVs, \( X_i^C \) is the location of the center point of the \( i^{th} \) RV, \( h \) is an arbitrary factor controlling the distance from the center, \( \sigma_{X_i} \) is the standard deviation of the \( i^{th} \) RV, and \( x_i \) is the coded variable for RV \( X_i \) having values of 0, ±1 or \( \pm \sqrt{2^k} \), depending on the coordinates of the sampling point with respect to the center point and sampling schemes. In a typical structural reliability estimation problem, RVs are expected to have different distributions. The iterative process of FORM is generally initiated at the mean values of all RVs in the standard normal variable space. To satisfy this requirement,
all non-normal variables need to be transformed to the normal variable space at the checking point (mean values at the first iteration). This transformation can be completed by equating the PDF and cumulative CDF at the checking point of the non-normal and the equivalent normal distributions (Haldar and Mahadevan 2000b; a). Denoting $\mu_{X_i}^N$ and $\sigma_{X_i}^N$ as the equivalent normal mean and standard deviation, they can be estimated as:

$$\mu_{X_i}^N = x_i^* - \Phi^{-1}[F_{X_i}(x_i^*)]\sigma_{X_i}^N$$  \hspace{1cm} (5.2)

and

$$\sigma_{X_i}^N = \frac{\phi[\Phi^{-1}[F_{X_i}(x_i^*)]]}{f_{X_i}(x_i^*)}$$  \hspace{1cm} (5.3)

where $x_i^*$ is the design or checking point, $\phi(\cdot)$ and $\Phi(\cdot)$ are PDF and CDF of the standard normal variable, and $f_{X_i}(\cdot)$ and $F_{X_i}(\cdot)$ are PDF and CDF of the non-normal $i^{th}$ RV, respectively. No transformation is required if the variables are normally distributed (Haldar and Mahadevan 2000a). The integration of FORM and old RSM can be completed by replacing $X_i^C$ and $\sigma_{X_i}$ by $\mu_{X_i}^N$ and $\sigma_{X_i}^N$, respectively.

In FORM analysis we are looking for the closest distance between a surface and the center point in standard normal space, which is called reliability index $\beta$. The first part is formulizing the surface, which is done by a surrogate model. The second part is applying the FORM procedure, which is expatiated in (Haldar and Mahadevan 2000b; a). Denoting $x^*$ is the coordinates of the final checking or design point, the reliability index $\beta$ can be shown to be (Haldar and Mahadevan 2000b; a)

$$\beta = \sqrt{(x^*)^t(x^*)}$$  \hspace{1cm} (5.4)

The corresponding $P_f$, is can be evaluated as:
\[ P_f = \Phi(-\beta) = 1.0 - \Phi(\beta) \quad (5.5) \]

where \( \Phi(-\beta) \) is defined earlier.

5.3 Generating AIRS

In generating an AIRS, three items need to be addressed. They are: (1) the form of the polynomial, (2) the center point around which samples need to be generated, and (3) sampling schemes. They are discussed next.

5.3.1 The Form of the Polynomial

To maintain the mathematical efficiency, the form or the degree of the polynomial to represent an AIRS should be kept as low as possible. For nonlinear dynamic problems of interest in this study, the first-order polynomial may not be sufficient. Higher order polynomial may cause ill-conditioning (Gavin and Yau 2008; Huh and Haldar 2001, 2011; Sudret 2012; Villegas Mercado et al. 2017). Considering several alternatives, the authors decided to use the second-order polynomial without and with cross-terms. Mathematically, they can be represented as (Azizsoltani et al. 2018; Villegas Mercado et al. 2017):

\[ \hat{g}(\mathbf{X}) = b_0 + \sum_{i=1}^{k} b_i X_i + \sum_{i=1}^{k} b_{ii} X_i^2 \]

and

\[ \hat{g}(\mathbf{X}) = b_0 + \sum_{i=1}^{k} b_i X_i + \sum_{i=1}^{k} b_{ii} X_i^2 + \sum_{i=1}^{k-1} \sum_{j>i}^{k} b_{ij} X_i X_j \]

where \( b_0 \) is the intercept or bias, \( b_i, b_{ii}, \) and \( b_{ij} \) are the unknown coefficients, \( k \) is the total number of the variables, and \( X_i (i = 1, 2, \ldots, k) \) is the \( i^{th} \) RV. The total number of
unknown coefficients for the second-order polynomial without and with cross-terms can be shown to be $2k + 1$ and $(k + 1)(k + 2)/2$, respectively. When $k$ is large, say 100 representing a realistic problem, the total number of the coefficients necessary to develop second-order polynomial without and with cross-terms will be 201 and 5,151, respectively. The mentioned information is summarized in Table 5.1. Obviously, the use of cross-terms, as, needs further consideration to increase the efficiency of the algorithm, as will be discussed later in Section 6.

Table 5.1 The total number of coefficients for each polynomial type

<table>
<thead>
<tr>
<th>total number of unknown coefficients</th>
<th>quadratic</th>
<th>quadratic with cross terms</th>
</tr>
</thead>
<tbody>
<tr>
<td>$k=100$</td>
<td>$2k + 1$</td>
<td>$(k + 1)(k + 2)/2$</td>
</tr>
<tr>
<td></td>
<td>201</td>
<td>5151</td>
</tr>
</tbody>
</table>

5.3.2 The Center Point Around Which Samples Need to Be Generated

To generate an AIRS, the coordinates of the initial center point will be the mean values of all RVs. Then, the appropriate AIRS-PF will be formulated and using FORM, the reliability index $\beta$ and the corresponding probability of failure ($P_f$) will be estimated. Since AIRS-PF will be nonlinear, it will take few iterations before the estimated $\hat{\beta}$ converges with a predetermined tolerance level. With the updated $\hat{\beta}$ value, the coordinates of the new center point will be estimated exactly following the FORM scheme. This iterative process will continue until estimated $\hat{\beta}$ converges to the final $\beta$ of interest for the reliability estimation.
5.3.3 Sampling Scheme

The final step in generating an AIRS is to estimate structural responses at sampling points around a center point. Considering efficiency and accuracy, three schemes are generally considered in the profession. They are Saturated Design (SD) without cross terms, SD with cross terms, and Central Composite Design (CCD). The SD scheme is very efficient; it requires only as many sample points as the number of unknown coefficients required to generate an AIRS. Furthermore, it can be used for RSs with second-order polynomial without and with cross terms. However, it may not be accurate enough for all applications. CCD sampling scheme is very accurate, has many desirable statistical characteristics, but can only be used for second-order polynomial with cross-terms. Furthermore, it requires a regression analysis to fit a polynomial through the data points or responses for this study. It can be shown that second-order polynomial without cross term can be generated using SD without cross term with $2k + 1$ samples or deterministic analyses. Also, the number of samples required for SD with cross term is $(k + 1)(k + 2)/2$. The required total number of samples using CCD for second-order polynomial with cross term will be $2^k + 2k + 1$. All schemes are shown in Figure 5.1 for 3 RVs. When $k = 100$, the absolute minimum number of samples according to SD without cross term will be 201, but for CCD, it will be about 1.26765e30. Obviously, CCD sampling scheme will be impractical for most problems with practical interest. Considering both accuracy and efficiency, the authors and their team members considered numerous sampling schemes. Since the proposed AIRS-PF is an iterative algorithm, the authors decided to use SD without cross term in the intermediate iterations, and CCD with cross terms in the final iteration. This combined sampling scheme will be denoted hereafter as the advanced
factorial design (AFD) scheme providing efficiency without compromising accuracy (Azizsoltani et al. 2018). It should be noted at this time that since CCD will be used in the final iteration, AFD is not yet implementable for large problems and needs further improvements, as will be discussed in Section 6. Table 5.2 compares between different schemes in terms of TNDA.

![Image of different factorial designs](image)

Figure 5.1 The schematic view of different factorial designs; (a) SD without cross terms; (b) SD with cross terms; and (c) CCD

Table 5.2 Comparison between different schemes in terms of TNDA

<table>
<thead>
<tr>
<th>k</th>
<th>Center</th>
<th>Axial</th>
<th>Edge</th>
<th>Factorial</th>
<th>Center</th>
<th>Axial</th>
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<td>1</td>
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<td>0</td>
<td>1</td>
<td>2k</td>
<td>0</td>
<td>(2^k)</td>
</tr>
<tr>
<td></td>
<td>2k + 1</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>k=3</td>
<td>6</td>
<td>0</td>
<td>0</td>
<td>7</td>
<td>6</td>
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<td>12</td>
<td>15</td>
<td>0</td>
<td>28</td>
<td>12</td>
<td>0</td>
<td>64</td>
<td>77</td>
</tr>
<tr>
<td>k=40</td>
<td>80</td>
<td>0</td>
<td>0</td>
<td>81</td>
<td>80</td>
<td>780</td>
<td>0</td>
<td>861</td>
<td>80</td>
<td>0</td>
<td>1.0955e12</td>
<td>1.0955e12</td>
</tr>
</tbody>
</table>
5.4 Generation of a Response Surface

As discussed in the previous section, since CCD is used in the final iteration of FOFM, a regression analysis is required to generate an AIRS. A regression analysis is essentially fit a polynomial to all the samples on an average sense. The authors believe that it compromises the inherited accuracy in using CCD. The authors decided to use the Kriging concept in generating an AIRS. Two basic desirable characteristics of Kriging are that an AIRS generated by it is uniformly unbiased and its prediction error is less than all other possible forms. These features make it the best linear unbiased surrogate for an AIRS (Wackernagel 2014). It can predict the response at any non-sampled point. They decided to use Kriging instead of regression to further improve the accuracy in CCD. The differences between the two approaches are shown in Figure 5.2. Suppose a curve needs to be fitted through sample points represented by dots in the figure. Curves obtained by regression analysis and Kriging are shown in the figure, clearly indicating the advantage of using Kriging. A brief mathematical discussion on Kriging is given below.
5.4.1 Kriging

Kriging is a weighted average of limited observations to estimate the value of any other points in space. To this end, the locations and values of the data points, the location of the point where an estimate is sought, a dissimilarity function, and a variogram is required. The weights are calculated based on minimizing this prediction variance subject to the conditions for uniform unbiasedness. There many different methods developed based on the idea of kriging. The main methods are Simple, Ordinary, and Universal kriging (UK) (Krige 1952; Wackernagel 2014).

5.4.1.1 Simple Kriging

Simple kriging assumes the mean value of random function $Z(X)$ is known and constant. In this regard, simple kriging is also known as kriging with known mean. Furthermore, $Z(X)$ is supposed to be second-order stationary with a constant mean value,
i.e. \( E[Z(X)] = \mu(X) = \mu \). The covariance function, also called covariogram, \( C(h) \) which only depends on the separating vector, \( h \), i.e. \( C(h) \equiv Cov(Z(X), Z(X + h)) \). A process is said second-order stationary (or weakly stationary) (1) if the mean of the process does not depend on \( x \): \( E[Z(x)] = \mu \), (2) if the variance of the process does not depend on \( x \): \( E[(Z(x) - \mu)^2] = \mu \), and (3) if the covariance \( C(h) \) only depends on the separation between two points of the process does not depend on \( x \): \( C(h) \equiv Cov[Z(Z(X), Z(x + h))] = E[Z(x)Z(x + h)] - \mu^2 \). The following formula represents the simple kriging predictor \( X_0 \):

\[
\hat{g}(X_0) \equiv \mu + \sum_{i=1}^{n} \omega_i(Z(X_i) - \mu) = \mu + \omega^T(Z - \mu 1) \tag{5.8}
\]

where \( \omega_i \in R, i = 1,2,\ldots,n \) denotes unknown weights corresponding to \( Z(X_i) - \mu \) and \( \omega \equiv (\omega_1, \ldots, \omega_n)^T \), which is the weight vector. By applying the two condition of minimizing the error and unbiasedness:

\[
\sum_{j=1}^{n} \omega_j C(X_i - X_j) = C(X_i - X_0) \quad i = 1,2,\ldots,n \tag{5.9}
\]

thus, weights can be calculated:

\[
\begin{bmatrix}
\omega_1 \\
\vdots \\
\omega_n
\end{bmatrix} = \begin{bmatrix}
C(X_1 - X) & \cdots & C(X_1 - X_n) \\
\vdots & \ddots & \vdots \\
C(X_n - X_1) & \cdots & C(X_n - X_n)
\end{bmatrix}^{-1} \begin{bmatrix}
C(X_1 - X_0) \\
\vdots \\
C(X_n - X_0)
\end{bmatrix} \quad \text{or} \quad \omega = \Sigma^{-1}C_0 \tag{5.10}
\]

with \( C_0 \equiv [C(X_1 - X_0), \ldots, C(X_n - X_0)]^T \in R^n \) denoting for the covariance between samples and location of interest. Also, \( \Sigma \in R^{n \times n} \) is the covariance matrix of \( Z \). Having \( \omega \), \( \hat{g}(X_0) \), simple kriging predictor, can be calculated.
5.4.1.2 Ordinary Kriging

Ordinary kriging considers the global mean constant; however, unlike simple kriging, does not consider mean as a known value, which makes ordinary kriging more practical than simple kriging. This method also assumes $Z(x)$ as an intrinsically stationary random function based on a predefined variogram function $\gamma(h)$. i.e.

$$\gamma(h) \equiv \frac{1}{2} \text{Var} [Z(X + h) - Z(x)] = \frac{1}{2} E[(Z(X + h) - Z(X))^2]$$  (5.11)

An intrinsically stationary process should satisfy the following conditions, (1) the mean of the process is constant: $E[Z(x) - Z(x + h)] = 0$, (2) the variance of the process does not depend on $x$: $E[(Z(x) - \mu)^2] = \mu$, (3) the variance of the process is finite and only depends on the separation between 2 points of the process: $\text{Var}[Z(x) - Z(x + h)] < \infty$. The ordinary kriging predictor at the required location $X_0$ is formulated as:

$$\hat{g}(X_0) \equiv \sum_{i=1}^{n} \omega_i Z(X_i) = \omega^T Z$$  (5.12)

In order to vanish the expected error, the sum of the weights should equal one:

$$\sum_{i=1}^{n} \omega_i = 1 \text{ or } \omega^T 1 = 1$$  (5.13)

Minimizing the error considering the above-mentioned constraint using the Lagrange multiplier $\lambda \equiv (\lambda_1, \ldots, \lambda_n)^T \in R^n$ results in:

$$\Gamma \omega + \lambda 1 = \gamma_0$$  (5.14)

which $\Gamma \in R^{n \times n}$ is symmetric variogram matrix, $\Gamma_{i,j} \equiv \gamma_Y(X_i - X_j)$, $i,j = 1,2,\ldots,n$, and $\gamma_0 \equiv (\gamma(X_1 - X_0), \ldots, \gamma(X_n - X_0))^T \in R^n$. Assuming $\Gamma$ is invertible, calculated weights are:
\[
\omega = \Gamma^{-1}[Y_0 - 1\left(\frac{1}{\Gamma^{-1}Y_0 - 1}\right)]
\]  

(5.15)

Having \(\omega\), \(\hat{g}(X_0)\) at the location of \(X_0\) can be calculated.

### 5.4.1.3 Universal Kriging

Considering the problem under consideration, UK concept is used in this study. UK is essentially a linear weighted sum of the required responses calculated by the analyses at sampling points using CCD. The generation of an AIRS, denoted hereafter as \([\hat{g}(X)]\), can be done using UK and addressing the two basic desirable characteristics of uniformly unbiased and the prediction error is less than all the possible forms (Azizsoltani and Haldar 2017b; Krige 1951; Wackernagel 2003). The UK predictor at the required location \(X_0\) is formulated as:

\[
\hat{g}(X_0) \equiv \sum_{i=1}^{n} \omega_i Z(X_i) = \omega^T Z
\]  

(5.16)

Where \(n\) is the total number of deterministic analyses, \(\omega_i\) is the unknown weights calculated based on the distance between the sample points and the unknown point, \(X_i\) is the coordinates of the \(i^{th}\) sample point, and \(Z(X_i)\) is estimated responses by the FE analyses for the \(i^{th}\) point in this study. It can be expressed as (Cressie 1993):

\[
Z(X) = u(X) + Y(X)
\]  

(5.17)

where \(Y(X)\) is an intrinsically stationary function with zero mean and underlying the variogram function \(\gamma_Y(h)\) and \(u(X)\) is a second-order polynomial with cross terms. To generate \(\gamma_Y(h)\), information on variogram cloud; i.e., a scatter diagram of the dissimilarities against distances, is required. To estimate weights, it is required do satisfy the universality condition as:
\[
\sum_{i=1}^{r} \omega_{i} f_{p}(X_i) = f_{p}(X_0) \text{ for } p = 0,1,\ldots,P
\]  

(5.18)

Where \( f_{p}(\cdot) \) is the ordinary regression equation of a second order polynomial with cross terms and \( X_0 \) is the coordinates of the unsampled point in which the response of the system needs to be predicted. Then, the weights required to define UK can be estimated as (Wackernagel 2003):

\[
\omega = \Gamma_Y^{-1}\left[ Y_{Y,0} - F\left(F^T\Gamma_Y^{-1}F\right)^{-1}\left(F^T\Gamma_Y^{-1}Y_{Y,0} - f_0\right)\right]
\]  

(5.19)

where \( Y_{Y,0} = [y_Y(X_1 - X_0),\ldots,y_Y(X_n - X_0)]^T \), \( \Gamma_{Y,i,j} = y_Y(X_i - X_j) \), \( F=f_p(X_0) \) is a matrix of size \( r \times (P + 1) \), and \( f_0 = f_p(X_0) \) is a vector of size \( (P + 1) \times 1 \). All the required steps are discussed in detail in (Azizsoltani and Haldar 2017b). With the availability of the weight factors, the mathematical expression for AIRS will now be available.

### 5.4.2 Variogram

The most important step in this process is formulating the variogram. Unfortunately, the true or underlying variogram is unknown and should be estimated. To this end, firstly, the dissimilarity function is defined as:

\[
\gamma^*(h_i) = \frac{1}{2} [Z(x_i + h_i) - Z(x_i)]^2
\]  

(5.20)

where \( Z(x_i + h_i) \) and \( Z(x_i) \) are the values of the predefined sample points, and \( h_i \) is the distance between them in direction of the \( i^{th} \) RV. The next step is creating the variogram cloud based on the experimental points, which is a plot of dissimilarity against distance. It provides a vague picture of the relationship between the points in space. Because, they might be more than one observation for a specific distance, experiential or empirical variogram is
defined, which is a function of the average dissimilarity between any two observations as a corresponding to their separation. The last but not least, a parametric variogram model should be used to fit the exertional variogram. Many assorted models are reported in the literature, including Nugget-effect, bounded linear, spherical, exponential, Gaussian, etc, and for the fitting method, least squares, maximum likelihood, minimum norm quadratic, etc. can be chosen. Definitely, the more input data for creating variogram cloud results in a more accurate function. It should be mentioned the covariance function (or covariogram) \( C(h) \) and variogram \( \gamma(h) \) are equivalent for a bounded variogram:

\[
C(h) = C(0) - \gamma(h) = \gamma(\infty) - \gamma(h)
\] (5.21)

Figure 5.3 summarizes the aforementioned steps schematically. Furthermore, part (c) and (d) explain some most important parameters in a variogram. Nugget is the difference between \( \gamma(0) \) and \( \lim_{|h| \to 0} \gamma(h) \), which is used to model discontinuity at the origin of the variogram. The value of \( \lim_{|h| \to \infty} \gamma(h) \) is defined as Sill. The Partial Sill is the difference between sill and Nugget. Range is the distance that variogram reaches its maximum value. In case of asymptotic functions, sill is usually considered as the 95% of the Sill value.

![Variogram cloud](image1.png)  ![Experimental variogram](image2.png)
Variogram

figure 5.3 The schematic view of (a) variogram cloud (b) experiential variogram (c) variogram (d) variogram

In this study, due to maximum flexibility, the anisotropic Gaussian model with weighted least square regression is selected (Krige 1951):

\[ \gamma_Y(h) = b \left( 1 - \exp \left[ \sum_{i=1}^{n} - \left( \frac{h_i}{a_i} \right)^2 \right] \right) \] (5.22)

with unknown coefficients of \( a_i \), called the range parameter, \( b \) Sill parameter and \( n \) is the number of RVs. The explicit LSF can be generated using the RS estimated using UK and the FORM algorithm can be used to extract the reliability information as discussed earlier.

5.5 Performance Function

As discussed earlier, to extract the underlying risk, required limit state or PFs denoted earlier as AIRS-PFs, need to be generated at this stage. For this study, AIRS-PFs are
defined in two levels; global and local. Global limit state is considered to study global behavior of a structure such as the total drift at the top of it. It can be defined as:

\[
g(X) = \delta_{allow} - \hat{g}(X)
\]  

(5.23)

where \(\delta_{allow}\) the allowable drift at the deck level and \(\hat{g}(X)\) is the AIRS for the drift at the deck level generated by the Kriging method discussed in the previous section.

Local AIRS-PF can be used to study local behavior such as the failure of a member in strength. Since all the structural members in a jacket-type OFS is considered to be made of steel in this study, the interaction equations suggested by the American Institute of Steel Construction (AISC) (2017) to consider effects of both the axial load and bending moments, are considered for local PFs. For 3D OFSs, they are represented as(2017):

When \(\frac{P_u}{\phi P_n} \geq 0.2\)

\[
g(X) = 1.0 - \left( \frac{P_u}{P_n} + \frac{8 M_{ux}}{M_{nx}} + \frac{8 M_{uy}}{M_{ny}} \right) = 1.0 - \left[ \hat{g}_P(X) + \hat{g}_M(X) \right]
\]

(5.24)

When \(\frac{P_u}{\phi P_n} < 0.2\)

\[
g(X) = 1.0 - \left( \frac{P_u}{2P_n} + \frac{M_{ux}}{M_{nx}} + \frac{M_{uy}}{M_{ny}} \right) = 1.0 - \left[ \hat{g}_P(X) + \hat{g}_M(X) \right]
\]

(5.25)

where \(P_n\) and \(P_u\) are the nominal and required tensile or compressive strength, respectively. \(\phi\) is the resistance factor. \(M_{nx}\) and \(M_{ux}\) are the nominal and required flexural strength about X-axis, respectively, and \(M_{ny}\) and \(M_{uy}\) are the nominal and required flexural strength about Y-axis, respectively. \(\hat{g}_P(X)\) and \(\hat{g}_M(X)\) are the AIRSs corresponding to axial force and bending moment, respectively.
5.6 Improvements in the Efficiency of AIRS-PF Discussed in the Previous Section

Conceptually, AIRS-PF just presented is explicit in nature and the conventional FORM can be used to extract the reliability information. For further discussion on the estimation of the reliability index for explicit PFs, the readers are referred to (Haldar and Mahadevan 2000b).

Unfortunately, since in the last iteration CCD will be used to generate an AIRS-PF, it may not be implementable for problems consist of a large number of RVs. To implement the proposed procedure, $N$ deterministic FE analyses need to be carried out. As mentioned earlier, in the intermediate iterations, SD without cross terms will be used in generating an AIRS. In the final iteration, CCD with cross terms will be used. Suppose, the total number of RVs present in the formulation in $k$. If $k$ is 10, it can be shown that the total number of FE analyses required will be more than 1,045 and may not satisfy the efficiency requirement. In most problems of practical interest, $k$ is expected to be much greater than 10. To make the proposed procedure implementable, the authors propose the following additional steps.

It is known that not all RVs are equally sensitive to the estimation of the reliability index. Haldar and Mahadevan (Haldar and Mahadevan 2000b) discussed that the sensitivity index of a RV could be obtained from the information on the direction cosines of RVs; the information generated during the implementation of FORM. The authors propose that just after the first iteration, less sensitive RVs can be modified to deterministic variables at their mean values. This additional step is expected to reduce $k$ to $k_R$, the reduced number of RVs.
Obviously, if $k_R$ is reasonably small, say less than 10, AIRS-PF can be generated but it will still require close to 1,000 deterministic FE evaluations. For general applications, the efficiency needs to be improved further by altering the CCD scheme used in the last iteration. The authors proposed the following improvement. CCD requires one center point, $2k_R$ axial points, and $2^{k_R}$ factorial points. The basic terms $(1 + 2k_R)$ cannot be reduced any further. The authors arranged $k_R$ RVs in descending order of their sensitivity indexes. Then, the factorial points are added iteratively only for the most significant RVs. They added the factorial points for RVs in CCD one by one in order of their sensitivity indexes until $\beta$ converges to a predetermined tolerance level. Suppose, it takes $m$ number of most significant RVs; $m \leq k_R$. In order to avoid ill-conditioning, only the cross terms for $m$ most significant variables need to be considered. AIRS generated in this way will be denoted hereafter as Modified Universal Kriging (MUK). The total number of FE analyses required for MUK can be shown to be $2^m + 2k_R + 1$. In most cases, the authors observe that $m$ will be smaller than $k_R$. In summary, the total number of deterministic analyses ($TNDA$) required to implement the proposed method can be shown to be:

$$TNDA = (1 + 2k) + n(1 + 2) + (2^m + 2k_R + 1)$$

(5.26)

where $n$ is the number of intermediate iterations.

With the above two improvements, the proposed reliability evaluation method can be denoted as AIRS-MUK-FORM. It can now estimate the underlying risk of any OFS excited by 3D wave loadings in the presence major sources of uncertainty and nonlinearity by using any nonlinear computer FE program capable of conducting dynamic analysis in time domain. The authors wrote a computer program to implement AIRS-MUK-FORM.
5.7 Summary

In order to maximize the efficiency, the process initiates with SD and second-order polynomial without cross terms, which gives a rough estimation about the design point and sensitivity of each RV. Afterwards, the total number of RVs are reduced to the most sensitive ones, which increases the efficiency without compromising the precision. Consequently, because the number of the RVs are limited, the more advanced scheme designs can be applied. However, if \( TNDA \) is more than the unknown coefficients in the mathematical formulation of the required polynomial, \( RS \) does not exactly pass through the sample points but follow the trends like a regression analysis which compromises the accuracy. To address this problem, instead of using conventional methods, UK method is employed to generate the surrogate model. Furthermore, not all the factorial points are equally influential. Thus, by eliminating less effective ones, the efficiency of the process is improved. Finally, a method called MKM is provided using UK and modified scheme designs. Furthermore, by integrating FORM and surrogate modeling, the surrogate model is formed around the failure region thanks to the iterative process of FORM. This is crucial step towards a more accurate reliability index. Furthermore, FORM provides the most important RVs which is used to reduce the complexity of the problem. AIRS-MUK-FORM can estimate the probability of failure of a complex system by a limited number of deterministic analyses. It can be an efficient alternative to MCS. The following figure summarizes all the procedures as a flowchart diagram.
Figure 5.4 Flowchart diagram of AIRS-MUK-FORM
CHAPTER 6
RELIABILITY ESTIMATION FOR A JACKET-TYPE OFFSHORE STRUCTURE FOR WAVE LOADING

6.1 Introduction

As discussed earlier, the reliability of OFSs needs to be estimated by exciting them dynamically in time domain and in the presence of major sources of nonlinearity and uncertainty. The most sophisticated reliability evaluation requires to apply the wave loading in three dimensions and in time domain. As mentioned earlier, assuming that the uncertainty in the structures for both onshore and offshore will be similar, the reliability of OFSs depends on how accurately and realistically the hydrodynamic wave loading and damping are incorporated in the formulation. In this chapter, AIRS-MUK-FORM is employed to evaluate the probability of failure of a typical 3D jacket-type OFS. A typical OFS is shown in Figure 6.1.
6.2 Example and Results

The capabilities of the proposed reliability evaluation method need to be documented. To meet this objective, a 3D jacket-type OFS shown in Figure 6.1 is considered. It is assumed to be in the Persian Gulf area. The depth of the water is considered to be 65 m, and the structure was designed satisfying API standard (Dyanati and Huang 2014). It has four legs with hollow steel section and maximum and minimum diameters of 1.752 m and 1.651 m, respectively, and their areas are $0.3368 \text{ m}^2$ and $0.0669 \text{ m}^2$. The corresponding moment
of inertias are $0.1202 \text{ m}^4$ and $0.0224 \text{ m}^4$. The total number of members in the structure is 237. The frame is a 4-story structure with a total height of 76.25 m. The dimensions of the structure at the bottom are 29 m in the X direction and 36 m in the Y direction. The corresponding dimensions at the top are 14 m and 20 m, respectively. The 4 supports are modeled with 3 springs. The total mass acting at the top of the structure is considered to be $3.2 \times 10^6 \text{ kg}$.

Uncertainty associated with all the RVs required for the reliability analysis are summarized in Table 6.1. The information is collected from the available literature. They include the significant wave height $H_s$, hydrodynamic coefficients $C_M$ and $C_D$ (drag and inertia coefficients), modulus of elasticity $E$, yield stress of steel $f_y$, strain-hardening ratio $b$, marine growth $D_m$, current velocity $V$, and directionality parameter $S$. Areas and moment of inertias of the members are considered to be lognormally distributed with coefficient of variation (CoV) of 0.05. Also, the mass at the top of the structures is considered to be lognormally distributed with CoV of 0.1 (Ellingwood 1980).

To estimate the risk of overall lateral deflection at the top of the structure using, the information on the allowable deflection is necessary. After a comprehensive literature review, the allowable deflection for jacket-type OFSs (Golafshani et al. 2011a; Sharifian et al. 2015) and the requirement for the steel structures required by AISC (2017), are found to vary between $H/100$ to $H/600$, where $H$ is the height of interest. For this study, the maximum allowable deflection at the deck level is considered to be $H/200$ or 0.381 m. Risks for the strength PF for member 1 and member 2, shown in Figure 6.1, are estimated. At the initiation of the AIRS-MUK-FORM algorithm, the total number of RVs needed to be considered was $k = 72$. After the sensitivity analysis, the most 5 important RVs are
selected, $k_R = 5$, and after using MUK, $m = 2$ for (local) and 4 (global) were used for the two PFs. $H_s$ is represented by a Weibull distribution. The two parameters of the distribution for the Persian Gulf region are $\alpha_w = 1.23$, $\beta_w = 1.24$ (Veritas 2010).

Table 6.1 Statistical information for RVs

<table>
<thead>
<tr>
<th>RV Classification</th>
<th>Random Variable</th>
<th>Unit</th>
<th>Mean</th>
<th>CoV</th>
<th>Distribution</th>
<th>Ref.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Significant wave height</td>
<td>$H_s$</td>
<td>m</td>
<td>1.148</td>
<td>0.81</td>
<td>Weibull</td>
<td>(Veritas 2010)</td>
</tr>
<tr>
<td>Hydrodynamic coefficients</td>
<td>$C_M$</td>
<td></td>
<td>1.2</td>
<td>0.1</td>
<td>Lognormal</td>
<td>(API 2007)</td>
</tr>
<tr>
<td></td>
<td>$C_D$</td>
<td></td>
<td>1.05</td>
<td>0.2</td>
<td>Lognormal</td>
<td>(API 2007)</td>
</tr>
<tr>
<td>Marine growth</td>
<td>$D_m$</td>
<td>cm</td>
<td>7.5</td>
<td>0.5</td>
<td>Lognormal</td>
<td>(Skallerud and Amdahl 2002)</td>
</tr>
<tr>
<td>Current velocity</td>
<td>$V$</td>
<td>m/s</td>
<td>0.8</td>
<td>0.2</td>
<td>Lognormal</td>
<td>(Cassidy 1999)</td>
</tr>
<tr>
<td>Directionality</td>
<td>$S$</td>
<td></td>
<td>3</td>
<td>0.05</td>
<td>Uniform</td>
<td>(ISO-19902 2007)</td>
</tr>
<tr>
<td>Modulus of Elasticity</td>
<td>$E$</td>
<td>GPa</td>
<td>206</td>
<td>0.03</td>
<td>Lognormal</td>
<td>(JCSS 2006)</td>
</tr>
<tr>
<td>yield stress of legs</td>
<td>$f_y$</td>
<td>MPa</td>
<td>335</td>
<td>0.07</td>
<td>Lognormal</td>
<td>(JCSS 2006)</td>
</tr>
<tr>
<td>strain-hardening ratio</td>
<td>$b$</td>
<td></td>
<td>0.02</td>
<td>0.1</td>
<td>Lognormal</td>
<td>(Azizsoltani et al. 2018)</td>
</tr>
</tbody>
</table>

As mentioned earlier, the CNW concept is capable of generating information of random wave loadings. To demonstrate this capability, the information on risk for 5 different random sea waves at the aforementioned location are estimated and documented.

For 5 different sea waves, Table 6.2 summarizes the reliability indexes and the corresponding probability of failure, as well as $TNDA$ for 1 global PF and 2 local PFs. The results were verified using 50,000 MCS cycles. High-Performance Computing (HPC) facilities available at the University of Arizona and parallel processing techniques were used for the verification. Based on the results given in Table 6.2, it can be observed that the results obtained by AIRS-MUK-FORM are very close to MCS. The results clearly indicate the viability of the proposed concept. However, instead of using 50,000 simulation
cycles, only about 200 deterministic analyses were needed to extract the reliability information using the proposed method. The reliability indexes estimated for both local and global PFs are similar but different, as expected. The reliability indexes are different indicating the influence of frequency contents of the wave loading in different wave states. However, the differences are not as significant as observed by the authors for the seismic loading [16, 44].

Table 6.2 Structural reliability of OFS subjected to wave loading

<table>
<thead>
<tr>
<th>Wave States</th>
<th>PF Member</th>
<th>AIRS-MUK-FORM</th>
<th>MCS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>TNDA β P_f</td>
<td>TNDA β P_f</td>
</tr>
<tr>
<td>1 Local</td>
<td>1 182</td>
<td>2.75482 0.002936</td>
<td>50000 2.77033 0.00280</td>
</tr>
<tr>
<td></td>
<td>2 182</td>
<td>2.80820 0.002491</td>
<td>50000 2.79438 0.00260</td>
</tr>
<tr>
<td></td>
<td>* 194</td>
<td>3.17964 0.000737</td>
<td>50000 3.18651 0.00072</td>
</tr>
<tr>
<td>2 Local</td>
<td>1 182</td>
<td>2.38464 0.008548</td>
<td>50000 2.38498 0.00854</td>
</tr>
<tr>
<td></td>
<td>2 182</td>
<td>2.56014 0.005231</td>
<td>50000 2.52774 0.00574</td>
</tr>
<tr>
<td></td>
<td>* 194</td>
<td>2.95572 0.001560</td>
<td>50000 2.96366 0.00152</td>
</tr>
<tr>
<td>3 Local</td>
<td>1 182</td>
<td>2.70305 0.003435</td>
<td>50000 2.72245 0.00324</td>
</tr>
<tr>
<td></td>
<td>2 182</td>
<td>2.79930 0.002561</td>
<td>50000 2.78456 0.00268</td>
</tr>
<tr>
<td></td>
<td>* 194</td>
<td>2.89713 0.001883</td>
<td>50000 2.90098 0.00186</td>
</tr>
<tr>
<td>4 Local</td>
<td>1 182</td>
<td>2.38357 0.008573</td>
<td>50000 2.39900 0.00822</td>
</tr>
<tr>
<td></td>
<td>2 182</td>
<td>2.73753 0.003095</td>
<td>50000 2.71839 0.00328</td>
</tr>
<tr>
<td></td>
<td>* 194</td>
<td>2.89153 0.001434</td>
<td>50000 2.98455 0.00142</td>
</tr>
<tr>
<td>5 Local</td>
<td>1 182</td>
<td>2.48489 0.006480</td>
<td>50000 2.49714 0.00626</td>
</tr>
<tr>
<td></td>
<td>2 182</td>
<td>2.50684 0.006091</td>
<td>50000 2.47403 0.00668</td>
</tr>
<tr>
<td></td>
<td>* 194</td>
<td>2.84031 0.002253</td>
<td>50000 2.86577 0.00208</td>
</tr>
</tbody>
</table>

* For global PF; lateral deflection at the top of the deck level.

Figure 6.2 summarizes the accuracy of the AIRS-MUK-FORM compared to MCS. Figure 6.2 a, Figure 6.2 b, and Figure 6.2 c show the difference in β for each wave state using the
AIRS-MUK-FORM compared to MCS for Global and Local PFs. As it can be seen the error is not significant.
Figure 6.2 Comparing results using AIRS-MUK-FORM and MCS
Table 6.3 summarize the minimum, maximum, mean, standard deviation, and CoV of $P_f$ regarding the given values in Table 6.2.

<table>
<thead>
<tr>
<th></th>
<th>Global</th>
<th>Local 1</th>
<th>Local 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min.</td>
<td>7.37E-04</td>
<td>2.94E-03</td>
<td>2.49E-03</td>
</tr>
<tr>
<td>Max.</td>
<td>2.25E-03</td>
<td>8.57E-03</td>
<td>6.09E-03</td>
</tr>
<tr>
<td>$\mu$</td>
<td>1.57E-03</td>
<td>5.99E-03</td>
<td>3.89E-03</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>5.65E-04</td>
<td>2.71E-03</td>
<td>1.66E-03</td>
</tr>
<tr>
<td>CoV</td>
<td>0.36</td>
<td>0.45</td>
<td>0.43</td>
</tr>
</tbody>
</table>

Figure 6.3 explains the data in Table 6.2 using box and whisker plot. It shows the first quantile, third quantile, mean, median, minimum and maximum of the data for each $\beta$. It demonstrates that the probability of failure in global PF is lower than the local PF. Also the uncertainty in global failure is lower than the local failures.

Figure 6.3 Summarizing estimated values for $\beta$ using different LFs
6.3 Summary

A robust and efficient risk analysis procedure for OFSs excited by the uncertainty-filled 3D complex wave loadings was presented in earlier chapters. In this chapter, this method was verified using an existing example and MCS. The new procedure is denoted as AIRS-MUK-FORM, and it was employed to evaluate the probability of failure of jacket-type OFS for wave loading. Two different types of limit states were considered, local and global limit states, as explained in chapter 5. For the local limit states the failure of member 1 and member 2 were studied. For the global, the maximum drift of the deck was considered. The information on risk for both PFs was extracted using about two hundred deterministic evaluations. The results were verified using 50,000 basic MCS cycles. The results clearly indicate the viability of the proposed concept. The reliability indexes estimated for both local and global PFs. The risk of the platform under five wave loadings are different, indicating the implication of frequency contents of the wave loading in different wave states. The authors believe that they developed an alternative to the basic MCS technique and a novel reliability evaluation method for OFSs.
CHAPTER 7
COMPARISONS OF RISK OF JACKET-TYPE OFFSHORE STRUCTURES
FOR WAVE AND SEISMIC LOADINGS

7.1 Introduction

As mentioned earlier, one of the main applications of reliability analysis is to compare the reliability of systems in different environments or evaluate different alternative for design. AIRS-MUK-FORM algorithm is a useful tool to compare the probability of failure an offshore structure against wave loading and seismic loading, the main crucial loadings for OFSs, and compare the uncertainty in the results.

The same jacket-type of the previous chapter is used here. As discussed before, wave and earthquake forces can act very differently. Also, since it is virtually impossible that the critical wave and earthquake loadings act simultaneously, OFSs are designed separately for wave and earthquake loadings. Therefore, the authors believe that a comparative study between the two crucial loadings will be very beneficial.

Because the basic structure remain the same, all major sources of uncertainty routinely considered for the design of OFSs remain the same (Vazirizade et al. 2020b). For the sake of brevity, no additional discussion is necessary. It should be mentioned that the loading on the structure is modeled in 2D to be comparable with the unidirectional earthquake loading. Conceptually, a sample water surface level in time domain using CNW in 2D is shown in Figure 7.1.
The current seismic design in the U.S. requires to consider at least 11 time histories. Although it is not explicitly stated in a design guideline for OFSs, 5 different wave profiles in time domain are considered in this study to satisfy the intent of the seismic design guidelines.

### 7.2 Case Study

The underlying risks of a jacket-type OFS shown in Figure 6.1 are estimated by exciting it by the wave and seismic loadings. Since the OFS considered in this study is located the Persian Gulf area, the site-specific design earthquake acceleration spectrum according to ASCE (2016) will not be directly applicable. Also, it is very difficult to obtain design details of a realistic OFS, the procedure suggested by the International Standard Organization (ISO) is used to obtain multiple earthquake time histories for the area. ISO-19901-2 (2017) suggests two levels of earthquakes; Extreme Level Earthquake (ELE) and Abnormal Level Earthquake (ALE). ALE is of interest of this research. For this level of excitation, the structures are expected to develop nonlinear behavior and some damages are expected; however, it should not suffer a complete loss of integrity or collapse.
According to ISO-19901-2 (2017), the location of the structure is in Seismic Zone 2. \( S_a(T = 1) = 0.3g \) and \( S_a(T = 1) = 0.75g \), which are spectral acceleration at period of 1 and 0.2 seconds.

According to ISO-19900 (2013), manned OFSs should be designed for the life-safety category of S1 for the seismic loading. Also, they should be considered as the consequence category of C2 resulting in the corresponding exposure level L1. For Seismic Zone 2 and exposure level L1, Seismic Risk Category (SRC) is 4. Assuming that the piles at the supports will produce the horizontal ground motions near the surface soil (API 2007), site class is assumed to be D. Based on site class as well as \( S_a(T = 1) \) and \( S_a(T = 0.2) \), the site coefficients for the acceleration and velocity of the response spectrum, denoted as \( C_a \) and \( C_v \), respectively, are estimated to be 1 and 1.2. The ISO 1000-year site horizontal acceleration spectrum \( S_{a,\text{site}}(T) \) is expressed as:

\[
S_{a,\text{site}}(T) = \begin{cases} 
(3T + 0.4)C_aS_{a,\text{map}}(0.2); & T \leq 0.2\text{sec} \\
\min\{C_aS_{a,\text{map}}(0.2), C_vS_{a,\text{map}}(1.0)/T\}; & 0.2\text{sec} < T \leq 4\text{sec} \\
4C_vS_{a,\text{map}}(1.0)/T; & 4\text{sec} \leq T 
\end{cases}
\] (7.1)

where \( T \) is the fundamental period of the structure. The acceleration response spectrum for ALE earthquake, \( S_{a,\text{ALE}}(T) \), can be obtained by taking the product of a scale factor based on the structure exposure level, \( N_{\text{ALE}} \) and \( S_{a,\text{site}}(T) \), as:

\[
S_{a,\text{ALE}}(T) = N_{\text{ALE}} \times S_{a,\text{site}}(T)
\] (7.2)

Considering the exposure level L1, \( N_{\text{ALE}} \) is estimated to be 1.6. The ELE spectral acceleration can also be calculated as:
\[ S_{a,ELE}(T) = S_{a,ALE}(T)/C_r \]  \hfill (7.3)

where \( C_r \) is seismic reserved capacity factor. For steel jacket of fixed offshore platforms based on the characteristics of the structure considered in this example, \( C_r \) is estimated to be 2.8. Figure 7.2 summarizes the site-specific spectrum for the location of the structure. The corresponding ALE and ELE spectra based on the aforementioned assumptions are also shown in Figure 7.2.

Obviously, any number of time histories can be used to fit a spectrum. As mentioned earlier, at least 11 time histories are required to be considered according to the current design guidelines. The entire PEER database consisting of several thousands of earthquake time histories recorded from all over the world is used in this study to select a suite of ground motions. Initially, the ground motions in the database are scaled to match the design spectrum at the fundamental period of the structure. The most desirable scale factor (SF) is 1.0 (Watson-Lamprey and Abrahamson 2006) but it is very rarely obtained. In this study, ground motions with SF of more than 4 and less than 0.25 are ignored. Following the above ground motion selection process, the database can be reduced from several thousands to several hundreds.

Considering ALE, as the target spectrum, at least 11 scaled earthquake time history records are generated. The submerged fundamental period of the structure is estimated as 2.1 s. The intention is to scale the records so that their spectral values match the ALE spectrum at fundamental period of the structure of 2.1 s.

To select the most suitable site-specific 11 time histories, it is required to rank all the potential candidates. A response spectrum is generally developed for a wide frequency
range. The authors considered a suitability factor to rank them. To develop the suitability factor concept, the authors considered the range of the period to be 0.2 and 2 times the fundamental period of the structure. This range is then subdivided into 50 equally spaced intervals in the log scale. At each of these periods, the differences between the selected ground motion spectral acceleration spectrum and the target spectrum are estimated. The total error for all 50 intervals, denoted as Square Root of the Sum of the Squares (SRSS) are estimated for each of the scaled ground motions. Eleven earthquake time histories with lower SRSS values are considered for further study. They are expected to contain all major sources of uncertainty in the earthquake excitations and will also satisfy the current seismic design guidelines. Following the procedures discussed, 11 records summarized in Table 7.1 are selected that satisfies the basic intent of the ASCE/SEI (2016) guidelines.

![Figure 7.2 Site-specific, ALE, and ELE spectra](image-url)
Table 7.1 Selected time histories

<table>
<thead>
<tr>
<th>Record Number</th>
<th>Scale Factor</th>
<th>Earthquake Name</th>
<th>Year</th>
<th>Station Name</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.0696</td>
<td>Imperial Valley-06</td>
<td>1979</td>
<td>Delta</td>
</tr>
<tr>
<td>2</td>
<td>0.9763</td>
<td>Imperial Valley-02</td>
<td>1940</td>
<td>El Centro Array #9</td>
</tr>
<tr>
<td>3</td>
<td>1.3497</td>
<td>Managua_ Nicaragua-01</td>
<td>1972</td>
<td>Managua_ ESSO</td>
</tr>
<tr>
<td>4</td>
<td>1.6083</td>
<td>Tabas_ Iran</td>
<td>1978</td>
<td>Dayhook</td>
</tr>
<tr>
<td>5</td>
<td>1.0292</td>
<td>Northern Calif-03</td>
<td>1954</td>
<td>Ferndale City Hall</td>
</tr>
<tr>
<td>6</td>
<td>2.9505</td>
<td>Coyote Lake</td>
<td>1979</td>
<td>Gilroy Array #4</td>
</tr>
<tr>
<td>7</td>
<td>2.8942</td>
<td>Imperial Valley-06</td>
<td>1979</td>
<td>Calexico Fire Station</td>
</tr>
<tr>
<td>8</td>
<td>2.0342</td>
<td>San Fernando</td>
<td>1971</td>
<td>LA - Hollywood Stor FF</td>
</tr>
<tr>
<td>9</td>
<td>2.9332</td>
<td>Kern County</td>
<td>1952</td>
<td>Taft Lincoln School</td>
</tr>
<tr>
<td>10</td>
<td>1.3927</td>
<td>Borrego Mtn</td>
<td>1968</td>
<td>El Centro Array #9</td>
</tr>
<tr>
<td>11</td>
<td>3.4952</td>
<td>Friuli_ Italy-02</td>
<td>1976</td>
<td>Buia</td>
</tr>
</tbody>
</table>

Figure 7.3 shows the spectrum of each of the records in Table 7.1 in solid lines. They matched exactly at the fundamental period of 2.1 seconds of the structure. The value of these spectra and the ± one standard deviation range are also shown in the figure in dotted lines. Moreover, it shows the target response spectrum in solid black line.
As discussed earlier, the uncertainty associated with the resistance-related RVs are collected from the literature. The parameters regarding the structure are the same as the parameters in chapter 6. The environmental parameter are almost the same. However, since the loading is 2D, directionality is not considered. Also, make the comparison more clear between wave loading and seismic loading, the current velocity is considered as zero. Also, there is an extra parameter regarding the earthquake intensity, $g_e$, which is the only additional parameter here. It is considered as Type I distribution with mean of 1 and CoV of 0.1 (Azizsoltani et al. 2018).

The total number of RVs is found to be 70 or $k = 70$ for the structure. After the first iteration, it was reduced to $k_R = 5$, in this study. For the serviceability LSF, the allowable deflection at the top of the platform is estimated to be 0.38125 m.
The reliability index $\beta$ and the corresponding probability of failure, $p_f$, for the strength and serviceability LSFs are estimated using AIRS-MUK-FORM method. The information on $\beta$, $p_f$, and the necessary TNDA for 5 wave states is summarized in Table 7.2 for the drift at the deck level and the strength limits state for elements 146 and 95. The results are verified using 10,000 MCS. HPC at the University of Arizona and the parallel processing techniques are used in obtaining results for MCS.
Table 7.2 Reliability estimation for the wave loading

<table>
<thead>
<tr>
<th>Wave States</th>
<th>Limits State</th>
<th>element</th>
<th>AIRS-MUK-FORM</th>
<th>MCS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>( \beta )</td>
<td>( p_f )</td>
</tr>
<tr>
<td>1</td>
<td>Strength</td>
<td>146</td>
<td>180</td>
<td>2.425</td>
</tr>
<tr>
<td></td>
<td></td>
<td>95</td>
<td>180</td>
<td>2.712</td>
</tr>
<tr>
<td></td>
<td>Serviceability*</td>
<td>192</td>
<td>2.893</td>
<td>1.91E-03</td>
</tr>
<tr>
<td>2</td>
<td>Strength</td>
<td>146</td>
<td>180</td>
<td>2.694</td>
</tr>
<tr>
<td></td>
<td></td>
<td>95</td>
<td>180</td>
<td>2.705</td>
</tr>
<tr>
<td></td>
<td>Serviceability*</td>
<td>192</td>
<td>2.950</td>
<td>1.59E-03</td>
</tr>
<tr>
<td>3</td>
<td>Strength</td>
<td>146</td>
<td>180</td>
<td>2.748</td>
</tr>
<tr>
<td></td>
<td></td>
<td>95</td>
<td>180</td>
<td>2.745</td>
</tr>
<tr>
<td></td>
<td>Serviceability*</td>
<td>192</td>
<td>3.009</td>
<td>1.31E-03</td>
</tr>
<tr>
<td>4</td>
<td>Strength</td>
<td>146</td>
<td>180</td>
<td>2.644</td>
</tr>
<tr>
<td></td>
<td></td>
<td>95</td>
<td>180</td>
<td>2.643</td>
</tr>
<tr>
<td></td>
<td>Serviceability*</td>
<td>192</td>
<td>3.082</td>
<td>1.03E-03</td>
</tr>
<tr>
<td>5</td>
<td>Strength</td>
<td>146</td>
<td>180</td>
<td>2.491</td>
</tr>
<tr>
<td></td>
<td></td>
<td>95</td>
<td>180</td>
<td>2.567</td>
</tr>
<tr>
<td></td>
<td>Serviceability*</td>
<td>192</td>
<td>3.079</td>
<td>1.04E-03</td>
</tr>
</tbody>
</table>

The same information for the earthquake excitations is summarized in Table 7.3 and Table 7.4, for the serviceability and strength LSFs, respectively. For the strength LSF, two elements 146 and 95 are considered. Element 146 is in the splash zone.

Table 7.3 Reliability estimation for the overall drift at the deck level for the earthquake excitations

<table>
<thead>
<tr>
<th>Record Number</th>
<th>AIRS-MUK-FORM</th>
<th>MCS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \beta )</td>
<td>( p_f )</td>
</tr>
<tr>
<td>1</td>
<td>178</td>
<td>2.567</td>
</tr>
<tr>
<td>2</td>
<td>178</td>
<td>2.394</td>
</tr>
<tr>
<td>3</td>
<td>178</td>
<td>2.503</td>
</tr>
<tr>
<td>4</td>
<td>178</td>
<td>2.699</td>
</tr>
<tr>
<td>5</td>
<td>178</td>
<td>2.695</td>
</tr>
<tr>
<td>6</td>
<td>178</td>
<td>2.642</td>
</tr>
<tr>
<td>7</td>
<td>178</td>
<td>2.709</td>
</tr>
<tr>
<td>8</td>
<td>178</td>
<td>2.452</td>
</tr>
</tbody>
</table>
Table 7.4 Reliability estimation for strength for elements 146 and 95 for the earthquake excitations

<table>
<thead>
<tr>
<th>Record Number</th>
<th>AIRS-MUK-FORM</th>
<th>MCS</th>
<th>AIRS-MUK-FORM</th>
<th>MCS</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\beta$</td>
<td>$p_f$</td>
<td>$\beta$</td>
<td>$p_f$</td>
</tr>
<tr>
<td>1</td>
<td>189</td>
<td>2.503</td>
<td>6.17E-03</td>
<td>2.536</td>
</tr>
<tr>
<td>2</td>
<td>189</td>
<td>2.164</td>
<td>1.52E-02</td>
<td>2.165</td>
</tr>
<tr>
<td>3</td>
<td>189</td>
<td>2.306</td>
<td>1.05E-02</td>
<td>2.319</td>
</tr>
<tr>
<td>4</td>
<td>189</td>
<td>2.443</td>
<td>7.28E-03</td>
<td>2.442</td>
</tr>
<tr>
<td>5</td>
<td>178</td>
<td>2.854</td>
<td>2.16E-03</td>
<td>2.848</td>
</tr>
<tr>
<td>6</td>
<td>189</td>
<td>2.338</td>
<td>9.70E-03</td>
<td>2.378</td>
</tr>
<tr>
<td>7</td>
<td>189</td>
<td>2.860</td>
<td>2.12E-03</td>
<td>2.834</td>
</tr>
<tr>
<td>8</td>
<td>189</td>
<td>2.472</td>
<td>6.73E-03</td>
<td>2.442</td>
</tr>
<tr>
<td>9</td>
<td>189</td>
<td>2.299</td>
<td>1.08E-02</td>
<td>2.297</td>
</tr>
<tr>
<td>10</td>
<td>189</td>
<td>2.311</td>
<td>1.04E-02</td>
<td>2.330</td>
</tr>
<tr>
<td>11</td>
<td>189</td>
<td>2.272</td>
<td>1.16E-02</td>
<td>2.280</td>
</tr>
</tbody>
</table>

Figure 7.4 and Figure 7.5 summarize the accuracy of the AIRS-MUK-FORM compared to MCS. Figure 7.4 a, Figure 7.4 b, and Figure 7.4 c show the difference in $\beta$ for each wave state using the AIRS-MUK-FORM compared to MCS for Global and Local PFs. Figure 7.5 a, Figure 7.5 b, and Figure 7.5 c express the same idea but for earthquake loading. The lines representing MCS and AIRS-MUK-FORM are very close to each other, which corroborates the accuracy of the method.
Figure 7.4 Comparing results using AIRS-MUK-FORM and MCS for wave loading
Figure 7.5 Comparing results using AIRS-MUK-FORM and MCS for earthquake loading
Table 7.5 and Table 7.6 summarize the minimum, maximum, mean, standard deviation, and CoV of $P_f$ regarding the given values of Table 7.2 through Table 7.4.

### Table 7.5 Summary of $P_f$ for the wave loading

<table>
<thead>
<tr>
<th></th>
<th>Deck Drift</th>
<th>Element 146</th>
<th>Element 95</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min.</td>
<td>1.03E-03</td>
<td>2.99E-03</td>
<td>3.04E-03</td>
</tr>
<tr>
<td>Max.</td>
<td>1.91E-03</td>
<td>7.64E-03</td>
<td>5.13E-03</td>
</tr>
<tr>
<td>$\mu$</td>
<td>1.38E-03</td>
<td>4.93E-03</td>
<td>3.81E-03</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>3.77E-04</td>
<td>1.99E-03</td>
<td>8.34E-04</td>
</tr>
<tr>
<td>CoV</td>
<td>0.27</td>
<td>0.40</td>
<td>0.22</td>
</tr>
</tbody>
</table>

### Table 7.6 Summary of $P_f$ for the earthquake loading

<table>
<thead>
<tr>
<th></th>
<th>Deck Drift</th>
<th>Element 146</th>
<th>Element 95</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min.</td>
<td>3.38E-03</td>
<td>2.12E-03</td>
<td>5.49E-03</td>
</tr>
<tr>
<td>Max.</td>
<td>1.38E-02</td>
<td>1.52E-02</td>
<td>2.06E-02</td>
</tr>
<tr>
<td>$\mu$</td>
<td>6.73E-03</td>
<td>8.42E-03</td>
<td>1.06E-02</td>
</tr>
<tr>
<td>$\sigma$</td>
<td>3.33E-03</td>
<td>4.00E-03</td>
<td>4.32E-03</td>
</tr>
<tr>
<td>CoV</td>
<td>0.49</td>
<td>0.47</td>
<td>0.41</td>
</tr>
</tbody>
</table>

Figure 7.6 explains the data in Table 7.2, and Figure 7.7 depicts the data in Table 7.3 and

Table 7.4, using box and whisker plot. It shows there is more uncertainty in earthquake loading compared to wave loading. Also as it was observed before, for wave loading, the local failures are more crucial than the global failure. This is to some extent correct for seismic loading as well.
Figure 7.6 Summarizing estimated values for $\beta$ using different LFs for wave loading

Figure 7.7 Summarizing estimated values for $\beta$ using different LFs for earthquake loading
7.3 Observations

The first conclusion from the results is the accuracy of the proposed method. For both the wave and earthquake excitations, $p_f$ values obtained by the proposed AIRS-MUK-FORM method match very well with the results obtained by 10,000 MCS. However, the proposed reliability method only requires only about 200 deterministic FE analyses instead of 10,000 MCS for both the serviceability and strength limit states. The proposed method appears to be very efficient without compromising the accuracy of the underlying estimated risk for both the seismic and wave loadings, which corroborates the conclusion of the previous chapter.

Apart from the verification of accuracy and efficiency of the proposed method, several important observations can be made from the case study. For this structure, the mean values of $p_f$ for both loadings are of the same order of magnitude indicating it is designed reasonably well. In the case study, the mean $p_f$ values for serviceability and strength LSFs due to earthquake loading appear slightly higher than that of the wave loading, indicating the earthquake loading will control the design. For this example, the seismic loading appears to be more critical. In terms of risk and consequences of failure, earthquake is more crucial than wave loading since its occurrence is not predictable while it is possible to evacuate the platform in adverse weather conditions. Therefore, the structure should have higher safety factors against seismic loading. However, the results from are reasonable since this structure is located in a gulf region close to some major faults which makes the seismic force as the dominant lateral load.

The uncertainty in predicting the probability of failure in strength for element 146, a member in the splash zone, indicates that it may be difficult to predict loading in the area.
In other words, the element in the splash zone compared to the other element, which is not in the splash zone, experience more uncertainty in wave loading.

Generally, the CoVs of $p_f$ are observed to be higher for the seismic loading than that of the wave loading for both the strength and serviceability limit states. This may indicate that there is more uncertainty in predicting the seismic loading than the wave loading. The period of the wave loading is expected to be higher than the earthquake loading. However, the submerged state of the OFSs is expected to increase its period compared to when they are not submerged. This tendency of approaching the wave period may cause the wave loading more critical than the earthquake loading. However, it is not the case for structure considered in the case study.

The example clearly indicates that the accuracy and efficiency of estimating the risk of the proposed AIRS-MUK-FORM for OFSs are very encouraging. Since no such method is currently available, the proposed method can be used for the uncertainty management of them and other complex structural dynamic systems.

### 7.4 Summary

The uncertainty management of complex nonlinear dynamic engineering systems can be achieved with the help of reliability analyses because uncertainty associated with some of the variables may not be important. A novel reliability analysis procedure, denoted as AIRS-MUK-FORM, was proposed in Chapter 5 to estimate the risk of such systems by representing them with FEs and by applying the dynamic excitation in time domain. Underlying risk of OFSs was not estimated in this way in the past. An OFS is then exited by the seismic loading in line with the recent guidelines. The underlying risks for both
wave and seismic loadings are estimated with the help of AIRS-MUK-FORM using only about 200 deterministic analyses. The accuracy of the proposed method was verified using 10,000 MCS for the serviceability and strength LSFs. The study confirms that the proposed method can be used to estimate risk of OFSs by applying wave loading in time domain. For the case study, the mean $p_f$ values for serviceability and strength LSFs due to earthquake loading appear slightly higher than that of the wave loading, indicating the earthquake loading will control the design. The CoVs of $p_f$ are observed to be higher for the seismic loading than that of the wave loading for both the strength and serviceability limit states. This may indicate that there is more uncertainty in predicting the seismic loading than the wave loading. The uncertainty in predicting $p_f$ for the strength LSF for an element in the splash zone (element 146), indicates that it may be difficult to predict loading in this zone. The period of the wave loading is expected to be higher than the earthquake loading. However, the submerged state of the OFSs is expected to increase its period compared to when they are not submerged. This tendency of approaching the wave period may cause the wave loading more critical than the earthquake loading. However, it is not the case for structure considered in the case study. It is very encouraging to note that AIRS-MUK-FORM can be used to estimate risk of OFSs accurately and efficiently for different LSFs. Since no such method is currently available, the proposed method can be used for the uncertainty management of OFSs and other complex structural dynamic systems.
8.1 Summary

Engineering profession’s main goal is designing the most reliable systems using limited resources. This goal cannot be achieved without considering uncertainty in problems and rare events. Structures are not built to last forever, but in order to make structures economically reasonable, a certain range of risk is acceptable. Thanks to the development of powerful processors, and new methods of computational analysis, more complex algorithms can be created to solve very advanced problems. Not surprisingly, there might be many different designs for a system, and they should be compared in terms of underlying risk and cost. Reliability analysis is an approach to calculate the underlying risk for a designed system, which paves the way of comparing different alternatives. The research team at the University of Arizona under the supervision of Professor Haldar has been developing a robust method for reliability analysis of complex nonlinear dynamical engineering systems over the last decades. The method established on the concept of SFEM and developed in time to be able to assess more advanced structures. The very last updated version of this method is the current study which is able to evaluate reliability of complex nonlinear 3D structures in time-domain. One of the most valuable structures which deserve attention is jacket-type OFSs. These structures are very complex and usually failure happens in nonlinear zone. Additionally, apart from the complexity of the structure, the environmental loading on the structure and the interaction between them is very important. Two major environmental loadings for these types of structures are wave loading and earthquake loading. In other to satisfy the deterministic community and also the
nonlinearity of the problem, the forces should be applied on the structure in time domain. The most accurate method in time domain is time history dynamic analysis. In this regard, considering all the sources of uncertainty in the structure and environment and their interaction in time domain make the problem more complicated. It also requires an accurate modeling of both seismic loading and wave loading including all of their uncertainties, in particular their intensity and frequency content. Even though there are decent research topics and papers for uncertainty quantification of seismic loading, there is few of them for wave loading. Considering wave loading in time domain and also incorporating the nonlinearity has never been addressed. The main goal of this study is reliability estimation nonlinear complex structures in time domain satisfying both deterministic and probabilistic community using the minimum number of deterministic analyses. In particular, this research focused on jacket type OFSs and uncertainty quantification of wave loading.

The developed novel risk evaluation technique is called AIRS-MUK-FORM which is a combination of intelligent sampling, UK, RS, and FORM. By employing UK, a surrogate model is generated. However, it is important to generate this surrogate model for the failure region, which is identified by FORM. In other words, in an iterative process, FORM defines the failure region, and the surrogate model is updated according to the new failure region.

To verify the proposed method, an OFS is evaluated for failure against wave loading. Also, the wave loading is modeled in 3D and time history to simulate the actual loading as accurately as possible. Various PFs are considered. The reliability of the structure is also estimated using MCS to verify the values by the proposed method. In another case, an OFS is exposed to both seismic loading and wave loading. The results show that not only can
this method be used for different types of loading, but this method can be used to compare a structure against different loading scenarios and optimize the design.

Based on the promising results of Chapter 6 and Chapter 7, the authors believe that the proposed method can be used for the reliability analyses of OFSs by applying loading in time domain. This method is applicable to various engineering systems, including 3D structures, nonlinear systems, and time-domain loading. The successful completion of the study paves the way to develop an accurate and efficient risk evaluation technique, which obviates the need for huge processors for reliability engineering. This method is expected to push the envelope of state-of-the-art of risk estimation.

8.2 Conclusions

Upon completion of this study, many key conclusions can be made as follows:

1. The proposed method can be used to evaluate the reliability of nonlinear complex engineering dynamic systems excited in time domain, in particular jacket type OFSs.

2. This method estimates the reliability of OFSs using a limited number of finite element analyses. This method can be a bridge between deterministic community and probabilistic community. It incorporates all major sources of uncertainty, and also considers dynamic excitations in time domain to satisfy deterministic community.

3. One of the most important sources of uncertainty in OFSs is the wave loading. In order to accurately take into account the uncertainty in the wave loading, the fluctuation of water surface level was modeled in time domain. The directionality of the wave loading was also considered. Three-directional wave loading is used in developing the proposed risk estimation procedure.
4. In order to take into account the uncertainty in earthquake-excitations, a site-specific seismic safety assessment procedure for nonlinear structural systems is presented.

5. Risks due to wave loading and seismic loading for an OFS are compared. This is one of the major advantages of this method which allows the user to compare different design alternatives or different loadings.

6. The proposed intelligent method is very efficient. Finite element analyses of complex nonlinear systems in time domain can be very cumbersome which makes MCS-based risk estimation virtually impossible. However, the proposed method provides similar results as that of obtained by MCS using few hundreds deterministic simulations instead of hundreds of thousands.

7. By combining FORM and UK, the reliability index can be calculated in the failure region using the iterative advantage of FORM.

8. Using UK, as an alternative to regression for surrogate modeling is obtained. It will increase the accuracy and efficiency of the risk estimation procedure.

9. The proposed method, AIRS-MUK-FORM, is an alternative for nonlinear random vibration concept and simulation.

10. This study pushes the envelope of state-of-the-art of risk estimation procedures.

8.3 Recommendations for Future Work

Even though the main goals of this research to study stochastic behavior of complex nonlinear dynamical engineering systems is achieved, there are rooms for future research based on the proposed method. The list of potential future research topic is as follows:

1. Capabilities of the proposed method can be verified using various complex structures such as Dam, fuselage of airplane, ship, thermal loading, etc.
2. This study does not consider the correlation between RVs.

3. There is a possibility for further dimension reduction in the problem using more advanced dimension reduction or machine learning techniques.
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