A Technical Framework for Probabilistic Assessment of Existing Offshore Platforms in Persian Gulf under Extreme Environmental (Wave) Loading

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Abstract

In this paper, a new probabilistic method (reliability analysis) under extreme environmental wave loading is introduced. This method is a direct probabilistic performance assessment using numerical method, in which the expression for mean annual frequency (MAF) of exceedance is derived by taking into account the aleatory and epistemic uncertainty in environmental hazard, structural response and capacity. A more realistic approach is used to calculate the system demand and capacity called “Incremental Wave Analysis” (IWA) instead of current nonlinear pushover analyses, in which multiple performances from onset of damage through global collapse can be assessed. The uncertainty propagation is estimated by Monte Carlo simulation and the reliability analyses are performed by using a numerical method. As a case study, the MAF of exceeding a given level of response and also the probability of failure are evaluated by this new approach.

Keywords: Offshore platforms, Mean annual frequency (MAF), Incremental wave analysis (IWA), Probability of Failure.

1. INTRODUCTION

Offshore jacket structures have been used in petroleum industry for decades. In an economical point of view, performing reconstructions and repairs to an existing installation is much preferable than constructing a new one. So, structural safety control of platforms beyond their design life has always been a matter of consideration. Due to the large uncertainties associated with the assessment of offshore platforms, there has been increasing interest in establishing assessment methods which are based on explicit considerations of reliability. Recently, American Petroleum Institute has developed recommendations for reliability-based assessment of offshore structures [1-3]. Target reliability levels and consequences of failure were explicitly addressed in the API guidelines, and criteria for assessment of offshore platforms against wave and seismic loads were developed. In a reliability and performance context, seismic loads, especially, have been the subject of important studies recently, for example the introduction of the Load Resistance Factor Design Code [4] and the ISO developments [5,6]. A major proposed procedure employed alternatively for probabilistic assessment for extreme wave loading is to verify that the jacket structure has the desired reliability. One approach is to estimate the probability of failure in terms of the structure's capacity considering its uncertainty as determined from nonlinear static pushover analyses; and a probabilistic description of the external wave loads. A convenient analytical format for such risk computations may be found in [7-9]. In recent years, an effective foundation and a conceptual framework for the development of seismic performance-based guidelines is presented by Pacific Earthquake Engineering Research Center (PEER) [10,11]. In this new procedure, the evaluations of structural performance (or seismic risks) can be expressed in terms of the mean annual frequency (MAF) of exceeding a given level of response. The present paper addresses similar issues for offshore structures with a focus on extreme environmental wave loads. The objective of this research is introducing the new PEER novel approach (used in seismic performance evaluation) for estimating the probability of failure due to extreme wave loading and MAF of exceeding a given level of response.
It must be noted that most of the current reliability studies on fixed leg offshore platforms have not taken into account the influence of foundation modeling, joint failure and wave-in-deck effects. In this research, the pile-soil-structure interaction, wave-in-deck loading and joint strength effects are included in the nonlinear analyses. A new approach called Incremental Wave Analysis (IWA) is introduced for calculating the structural demands and capacity instead of the current nonlinear static (push-over) analysis.

2. Foundation Development and the Proposed Method

The proposed procedure’s objective here is quantifying the limit state probability. This expression is derived by taking into account the uncertainty in the estimation of wave hazard, structural response (base shear hazard), and structural capacity. The following steps are involved in this procedure:

1. At first the wave hazard (annual probability that the wave height exceeds a certain value) is calculated.
2. By performing multiple nonlinear static analyses, a relation between wave height and response base shear is derived, which is called Incremental Wave Analysis (IWA). The drag coefficient, $C_D$ and steel yield strength are chosen as the main source of uncertainty in response calculation of the jacket platform and thus, base shear will have a probability distribution for each wave height. The uncertainty propagation is evaluated by using the Monte Carlo simulation.
3. The probability distribution of system strength is also derived by taking into account the above uncertainties, using the IWA method.
4. Finally, from the above distributions, the probability of failure of the system and the MAF of exceeding a given performance level can be estimated.

3. Structure Configuration

The proposed method is applied to a 2-D fixed leg jacket platform. A view of the platform is shown in Figure 1. Two piles with 45 m penetration depth transfer the structure's load to the underlying soil layers. Furthermore, the soil is divided into 1 m layers, for each two equivalent p-y and t-z springs are required for nonlinear soil-pile interaction. Also, a spring models the q-z curve at each pile’s tip. The standard USFOS program is used for the collapse analyses of the example jacket.

4. Wave Height Hazard

In this research, linear wave theory (airy wave theory) is used to model the wave phenomenon. In this theory wave height is assumed twice as wave crest height. The annual maximum wave height distribution is determined for the meteo-ocean criteria of Persian Gulf by the analytical method proposed in [8]. The wave height hazard curve, shown in Figure 2, represents the annual probability that the wave height exceeds a certain value:

$$\lambda_{\text{max}}(x) = P[H_{\text{max}} \geq x]$$

(1)
5. PROBABILISTIC DEMAND ANALYSIS UNDER EXTREME ENVIRONMENTAL (WAVE) LOADING

Probabilistic demand analysis assesses the performance of jacket type offshore platforms by probabilistically predicting the structural response under extreme environmental (wave) loading, and combining this information with wave hazard analysis results. In this method, the structural response is quantified via a demand parameter (base shear, RSR, or even displacement demand parameters). In the present study, the base shear is considered as the demand parameter and its variations are considered due to wave height, drag coefficient and material yield strength. The distribution of the wave height is evaluated from the wave height hazard curve. The drag coefficient is assumed to have lognormal distribution with median 0.75 and coefficient of variation 0.25. The steel yield strength is assumed to be normal random values with median 248.199 MN and coefficient of variation 0.06. By using the Monte Carlo method, 50 simulations for each random variable are generated. The IWA is obtained by performing multiple nonlinear static analyses for each pair of simulated random variables. This process can be interpreted in a different manner, e.g. for each specific wave height \( H_{\text{max}} = h \), a multiple strip containing 50 simulated point representing the variation in demand parameter (base shear) are provided. In the formulation of the present study, we should first calculate the demand hazard in order to be able to estimate the probability of failure. The demand hazard \( \lambda_{D,p} \) is defined as the annual probability that the demand parameter (DP) exceeds a certain value such as \( x \). \( \lambda_{D,p} \) is a direct measure of the performance of a jacket structure because it relates to the probability of experiencing the event \( DP > x \) within the next 25 years:

\[
\lambda_{D,p}(x) = P[DP > x]
\]

By applying the total probability theorem to Equation 2, we have:

\[
\lambda_{D,p}(x) = P[DP > x] = \sum_{h} P[DP > x | H_{\text{max}} = h] \times P[H_{\text{max}} = h]
\]

We should note that the above expression involves discrete variables. Since we are using analytic parameter estimations, we are going to base our derivations for the demand hazard derived for continuous variables:

\[
\lambda_{D,p}(x) = \int_{h} P[DP > x | H_{\text{max}} = h] \times \int_{h} d\lambda_{D,\text{NC}}(h) = \int_{h} G_{D,\text{NC}}(x|h) \times d\lambda_{D,\text{NC}}(h)
\]

Where \( G_{D,\text{NC}}(x|h) \) is the conditional complementary cumulative density function (CCDF) of demand parameter, \( DP \), for a given wave height \( H_{\text{max}} = h \). At each specific wave height, a fraction of waves will cause collapse in the structure (denoted by \( C \)), indicated by non-convergence of nonlinear static analysis or a large increment in structural deformations with a small increment in wave height intensity. If a relatively large number of simulated data are used, the probability of non-collapse (denoted by \( NC \)) at each \( H_{\text{max}} = h \) level \( (P_{NC|h}) \) can be empirically estimated as by the ratio of the non-collapse cases to the total number of simulated response points on the stripe. The demand hazard considering collapse and non-collapse conditions simultaneously can be obtained by [12]:

\[
G_{D,\text{NC}}(x|h) = G_{D,\text{NC},\text{NC}}(x|h) \times P_{NC|h} + 1 - P_{NC|h}
\]

For non-collapse data, the conditional complementary cumulative density function of demand for a given wave height, \( G_{D,\text{NC},\text{NC}}(x|h) \), can be estimated by empirical distribution. Also, we can fit a probability distribution to the simulated data. Lognormal distribution seems to be a proper fit. The mean and standard deviation of \( \ln DP \) (denoted by \( \mu_{\ln DP,\text{NC}}(h) \) and \( \sigma_{\ln DP,\text{NC}}(h) \), respectively) can be estimated from the simulated sample of data using the method of moments [13]. So, using lognormal distribution properties we have:

\[
P[DP > x | H_{\text{max}} = h, NC] = 1 - \Phi\left(\frac{\ln x - \mu_{\ln DP,\text{NC}}(h)}{\sigma_{\ln DP,\text{NC}}(h)}\right)
\]

where \( \Phi(\cdot) \) is the standard Gaussian cumulative distribution function. From the above formulations, one can use the results of the multiple-stripe analysis in order to estimate the probability distribution of demand in the all regions especially in region of global instability. The results of the multiple-stripe analysis on the jacket structure are represented by a series of “stripes” that are plotted for multiple wave height levels. For each wave height level, \( h \), the non-collapse part of the stripe response is modeled by a lognormal (parametric) or
empirical (non-parametric) distribution. The results of the multiple-stripe analysis on the structures are plotted in Figure 3. The collapse and non-collapse cases are marked in this figure.

The CDF for the non-collapse part of the stripe response can also be estimated empirically (non-parametric) by the fraction of the non-collapse results that are less than or equal to a given demand value. The probability of non-collapse is estimated empirically by the ratio of the non-collapse cases observed in the stripe response to the total number response points on the stripe. Now, we can calculate the 16th, 50th and 84th percentiles of the marginal distribution in Equation 5 for each wave height level.

Figure 4-a illustrates the non-parametric (counted) percentiles, using empirical distribution, of the demand parameter as a function of wave height considering the collapse cases. The heavy lines stop when the probability of collapse becomes so large to leave them undefined. Figure 4-b is the same as Figure 4-a using lognormal distribution. The conventional method of moments can be used to determine the lognormal parameters. The curves, illustrated in Figure 4-a and Figure 4-b, are very close to each other and this implies the fact that the lognormal distribution for demand parameter given non-collapse is suitable.

From Equation 4 and 5, the demand hazard (MAF of exceeding a demand parameter level) can be evaluated from the following equation:

\[ \lambda_{DP}(x) = \int_{h_{min}}^{h_{max}} \left[ G_{\text{DP} \rightarrow \text{NC}} \left( x \big| h \right) \times P_{\text{NC} \rightarrow \text{NC}} \left( h \right) + 1 - P_{\text{NC} \rightarrow \text{NC}} \left( h \right) \right] \times dP_{\text{NC}} \left( h \right) \]

In calculation of demand hazard by numerical integration, two methods can be applied. In the first method, the empirical distribution (non-parametric) can be used to estimate the \( G_{\text{DP} \rightarrow \text{NC}} \left( x \big| h \right) \). The demand parameter hazard can also be calculated by applying the lognormal distribution. The demand parameter hazard curves by using the two methods are plotted in Figure 5, together. The two curves are so close that they may be used inter-changeably. This implies that the lognormal distribution is an adequate representation of the demand parameter given no collapse for given wave heights.
6. STRUCTURAL COLLAPSE AND BASE SHEAR CAPACITY

There are various probabilistic and deterministic aspects associated with the structural resistance analysis which require further studies in a system reliability assessment. There exist various sources of uncertainty in the resistance determination which have not been examined in sufficient detail so far. The majority of the studies in this field have concentrated on the substructure response without detailed examination of the foundations and deck response. Based on the results of such studies, component strength is known to be the most significant source of uncertainty in structural capacity determination [14]. In fact, modeling uncertainty has a major impact on structural reliability assessment [15, 16].

Foundation and joint strength modeling are the two parameters which despite the high influence they have on structural uncertainty, are rarely included in the performed studies. The outstanding part of modeling in this research is the incorporation of the foundation and joint strength in the model. These two parameters definitely affect the near failure behavior of system and also the estimated structural capacity. For more information about the significance of joint failure reference is made to [17, 18]. The other significant part in the current model is the wave-in-deck loading which is applied as a concentrated load at the deck level. A good estimate of the slamming load on a plane area may be written as [19]:

\[ \rho = \rho \cdot u \cdot c \cdot A \]  

(8)

Where \( \rho \) is the density of water, \( u \) is the particle velocity in the wave, \( c \) is the wave velocity and \( A \) is the exposed area. The wave in deck has the primary influence in structural resistance. The load application pattern is another controversial issue. In the conventional pushover analysis, a load distribution associated with an extreme design event, such as the 100 year storm, is applied on the substructure and then it is increased in steps until structural collapse happens. But it seems that this approach cannot model the real wave phenomenon that happens in the site. So, the alternative approach called "Incremental wave analysis" is introduced previously in this paper in which increasing wave heights are applied to the structure. The advantage of using this technique is that we can take into account the wave-in-deck loading in structural capacity determination. In fact, this is the actual event in case of occurrence of extreme environmental loading which conventional pushover analysis was not able to model it. This method estimates lower ultimate capacities than pushover analysis method. As discussed in the previous section, the incremental wave analysis technique is applied to the jacket model in which joint strength, foundation modeling and wave-in-deck loading are accounted for. The wave height increased until the structural instability is occurred and this point associated to a specific wave height and base shear, is called the capacity of the jacket structure. These points are highlighted in Figure 3. The cumulative probability density function (CDF) of limit state capacity, \( F_{CS} \), can be obtained empirically. Also, the statistical properties of the so-called collapse points can be evaluated. It is observed that a lognormal distribution is an adequate representation for the global collapse capacity of the jacket platform. Figure 6 compares the CDF of limit state capacity by using empirical and lognormal distribution.
7. PROBABILITY OF FAILURE

The probability of failure or the limit state probability is composed of all possible combination of $D_P = d_p$, and $C < d_p$ in which $D_P$ denotes demand parameter and $C$ denotes capacity of global collapse limit state. The probability of failure, $P_L$ (limit state probability, $P_{LS}$) can be written as:

$$P_L = P_{LS} = P[DP > C] = \sum_{all \ dp} P(DP = dp \cap C < dp)$$  \hspace{1cm} (9)

Using the total probability theorem, an alternative solution strategy consists of decomposing the expression for the limit state probability in two steps and therefore employs two interface variables. The first step is to decompose the limit state probability with respect to the demand parameter (the first interface variable):

$$P_{LS} = P[C < DP] = \sum_{all \ dp} P[C < DP | DP = dp] \times P[DP = dp]$$ \hspace{1cm} (10)

The second step is to decompose the term $P[DP = dp]$ or the likelihood that the demand parameter is equal to a value $d_p$, with respect to the wave height (the second interface variable):

$$P_{LS} = P[C < DP] = \sum_{all \ dp} P[C < DP | DP = dp] \times \left( P[DP = dp | H_{\text{max}} = h_{\text{max}}] \times P[H_{\text{max}} = h_{\text{max}}] \right)$$ \hspace{1cm} (11)

Equation (11) can be re-written in the following expression:

$$\lambda_{LS} = \int \int F_{C_{LS}}(x) \times f_{DP_{\text{max}}}(x | h_{\text{max}}) \times \frac{d \lambda_{H_{\text{max}}}}{dh_{\text{max}}} \times dh_{\text{max}}$$ \hspace{1cm} (12)

Where $\lambda_{LS}$ is the mean annual frequency of exceeding a limit state, $F_{C_{LS}}$ is the cumulative probability density function (CDF) of capacity for the limit state, $f_{DP_{\text{max}}}$ is the probability distribution function of the demand for a given level of wave height and finally $\lambda_{H_{\text{max}}}$ denotes wave height hazard in terms of the mean annual frequency of exceedance. Equation (12) can be re-written in the following manner:

$$\lambda_{LS} = \int F_{C_{LS}}(x) \times \left| \frac{d \lambda_{DP}}{dx} \right| \times dx$$ \hspace{1cm} (13)

$$\frac{d \lambda_{DP}}{dx} = \int \int G_{DP_{\text{max}}}(x | h_{\text{max}}) \times \frac{d \lambda_{H_{\text{max}}}}{dh_{\text{max}}} \times dh_{\text{max}} = \int f_{DP_{\text{max}}}(x | h_{\text{max}}) \times \left| \frac{d \lambda_{H_{\text{max}}}}{dh_{\text{max}}} \right| \times dh_{\text{max}}$$ \hspace{1cm} (14)

By combination of Equations (13) and (14), the expression in Equation (12) for mean annual frequency of exceeding a limit state can be obtained.
The mean annual frequency of exceeding a limit state, $\lambda_{LS}$, can be obtained by numerical integration by implementing two approaches: empirical distribution and lognormal distribution. The mean annual frequency of exceeding a limit state, $\lambda_{LS}$, obtained by numerical integration is

$\lambda_{LS} = 2.855 \times 10^{-6}$

using empirical distribution in calculation of CDF of limit state capacity, $F_{CLs}$ and demand parameter hazard, $\lambda_{DP}(x)$, and

$\lambda_{LS} = 3.022 \times 10^{-6}$

using lognormal distribution for limit state capacity and demand parameter. The mean annual frequency of exceeding a limit state, $\lambda_{LS}$, is plotted in figure 8 with the demand parameter hazard, $\lambda_{DP}(x)$, which are both evaluated by numerical integration method. It is illustrated that for large demand parameter values, the demand hazard is asymptotically equal to the annual frequency of collapse limit state. It can be concluded that the lower limiting value of the demand hazard curve is the annual frequency of collapse, $\lambda_{LS}$, as illustrated in Figure 8. A certain design criteria is to design a jacket structure so that the mean annual frequency of exceeding a certain criteria limit state (limit state frequency) is less than or equal to the allowable annual probability $P_o$:

$$\lambda_{LS} < P_o$$  \hspace{1cm} (15)

8. CONCLUSION

In this paper, a new approach for determining the probability of failure of a jacket platform is introduced. The IWA is introduced as a substitute to the conventional pushover analysis. In IWA, the structural demand (base shear in this paper) is evaluated by increasing the wave heights. The advantage of using this technique is that we can take into account the wave-in-deck loading in structural capacity determination. In fact, this is the actual event in case of occurrence of extreme environmental loading which conventional pushover analysis was not able to consider. The demand hazard curve is obtained by combining the wave height hazard obtained from the long term sea state analysis with the demand results from the IWA by means wave height as an intermediate variable. In this research, the uncertainty due to drag coefficient and material yield strength are taking into account, in which the former is a random variable associated with the wave load on the structure and the later is a random variable due to member capacity.

By using Monte Carlo method, 50 different simulations are obtained for each random variable. The IWA can be achieved for each pair of simulations. So, the distribution of structural demand for each wave height can be attained by using multiple strip analysis, for which the lognormal or empirical distribution can be used. Both of these distributions will results identical demand hazard curve.

The probability of failure and the annual frequency of collapse can be evaluated by combining the demand hazard curve with the capacity distribution of the jacket, which is also evaluated by IWA. It is illustrated that for large demand parameter values, the demand hazard is asymptotically equal to the annual frequency of collapse limit state. It can be concluded that the lower limiting value of the demand hazard curve is the annual frequency of collapse. This is a matter that needs to be further examined. Moreover, some more simplifications are required to be applied to this procedure, in order that it can be used as a practical probabilistic strategy for assessment of existing offshore platforms.
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10. REFERENCES


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