Seismic Reliability Assessment of Jacket Offshore Platforms

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Abstract

The performance-based earthquake engineering has gained major attention in assessment of the structural dynamic behavior of Structures in the past decade. In this paper, Analytical models are employed including a comprehensive nonlinear model for offshore platforms that incorporates “Fiber Elements” which are capable of modeling post-buckling behavior of braces. Incremental dynamic analysis is then utilized to generate required data for performance based evaluation based on nonlinear dynamic analyses and reliability theory with regard to uncertainty. Moreover, case study on presently designated jacket offshore platforms in South Pars Gas Field (Phase 19 platform) Of the Persian Gulf region has been performed. Two-dimensional models of the mentioned platforms and the pile stubs with actual soil in-situ characteristics are simulated using “OpenSees” software. This research is intended to contribute to the progress in improvement of the methods on seismic design and evaluation of offshore structures.

Keywords: Performance-Based Earthquake Engineering (PBEE), Incremental Dynamic Analysis (IDA) Pile-Soil-Structure Interaction, Uncertainty, Confidence Level, Mean Annual Frequency (MAF).

Introduction

The Persian Gulf region is one of the important energy zones in the world. Various Oil and Gas fields are located in this region. To extract the valuable energy resources, offshore structures are designed, constructed and installed. These offshore structures are mainly fixed jacket type platforms which are cost effective since the maximum installation depths of jackets are about 70 m. The south pars gas field is located approximately 100 km off the Iranian coast in the Persian Gulf. Phase 19 of the south pars gas field is developed to allow a gas production of 2000 MMSCFD. The offshore portion of the project consists of three offshore platform complexes here an attempt has been made to determine the seismic reliability for the 19A complex i.e. SPD 19A through performance-based earthquake engineering (PBEE) framework.

One of the objectives in PBEE is to quantify the seismic reliability of an existent structure due to future random earthquakes. For that purpose, experimental and analytical investigations have been conducted to evaluate capacities of offshore structures, particularly on jacket types under seismic excitation [1].

Earthquake design of offshore platforms in seismic active areas is one of the most important parts in offshore platforms design. Dynamic response of offshore platforms is a function of the characteristics of the loading, dynamic pile-soil interaction behavior and dynamic characteristics of the piles structural system.

In order to evaluate the aforementioned dynamic behavior as accurate as possible, Incremental dynamic analysis (IDA) is utilized. IDA is a newly developed approach for performing structural analyses in a manner that reflects the earthquake-induced uncertainties systematically and completely. Implementation of the IDA involves selecting a set of ground motion records that best represent the characteristics typical to the ground motion likely to happen in the structure site.

Reliable Performance-based seismic evaluation necessitates a robust and detailed analytical model in which Jacket connections, braces, legs and Piles are all modeled to simulate the behavior of real steel jacket type offshore platforms as accurately as possible. Hence, in this research, comprehensive analytical model has been developed using OPEN System for Earthquake Engineering Simulation (OPENSEES[2]) which has the capability of accounting non-linear geometric and material properties. This program also has nonlinear elements, in particular, “Fiber Elements” which offers precise monitoring of each individual element’s behavior through its length in an adequate manner.

Performance-based seismic evaluation is a process to quantify the seismic reliability for a proposed new or existing structure due to future random earthquakes. Here this quantification is represented by confidence level or the probability to satisfy the desired performance at discrete hazard levels and is also presented in annualized basis, i.e. mean annual frequency (MAF) of exceeding a specified structural limit state of interest. In this paper, case studies are conducted on designated jacket in the region. Single (two dimensional) end-on or broadside frames of the jacket with in-plane apparent battered legs/piles are used for the analysis. Furthermore, overview of reliability assessment sample jacket type offshore platforms located in the Persian Gulf is outlined and the pertinent results are presented.

PBEE Methodology

Earthquake-induced damage in a structure will rise to consequential economic losses. From decision making stand point, the reduction in loss-induced costs due to a performance improvement plan, should be assessed with respect to different risk levels. On the other hand, the performance of a structure during a seismic event, as was the subject of traditional structure standards, was only regarded from a qualitative perspective and quantitative statements that could facilitate financial assessment of a design project was quite a missing aspect. Hence quantification of structures
performance against potential seismic events is the concern of newly developed performance-based guidelines, e.g. FEMA-273, FEMA 350, FEMA-356, ATC-63 and 50% draft of ATC-58 project report. These standards aim at quantifying structure performance in terms of a decision variable that is considered as the decision making tool and can be chosen, for instance, as economic losses associated with a design process. The PBEE methodology introduced by Pacific Earthquake Engineering Research (PEER) Center forms the basis of these guidelines methodology. The mathematical representation of this framework is presented in Equation 1[3]:

\[ \lambda(DV) = \int \int \int \int \int G(DV | DM) G(DM | EDP) G(EDP | IM) dG(IM) \]

where terms \( \lambda(X) \), \( G(X|Y) \), DV, DM, EDP, and IM denote respectively the Mean Annual Frequency (MAF) of exceeding the parameter X, the Cumulative Distribution Function (CDF) of X conditioned on Y, Decision Variable, Damage Measure, Engineering Demand Parameter, and Intensity Measure. For a good understanding of this equation it must be first noted that structure performance against ground shaking is dependent on four important factors. Prediction of these factors is associated with some levels of uncertainty, therefore, leading us to express them as a probability distribution function. These factors and the corresponding probabilistic terms, as used in PBEE (Eq. 1), are as follows:

- the intensity of ground shaking; \( \lambda(IM) \),
- the amount of demands induced in various structural and nonstructural components in a structure, i.e. response level, at a particular IM level; \( G(EDP|IM) \),
- the extent to which structure is vulnerable to damage at a specific response level; \( G(DM|EDP) \),
- and finally, the condition, such as number of people and facilities and their location in the structure, that affect the amount of economic and casualty losses occurred at a damage state; \( G(DV|DM) \).

According to these parameters, PBEE methodology entails four analysis steps: Probabilistic Seismic Hazard Analysis (PSHA), structural analysis, damage analysis, and loss analysis, each corresponding to assessing one of the above probabilities respectively. Eq. 1 combines these probabilities with respect to total probability theorem so that the risk of incurring different DV values is calculated in terms of MAF of exceedance. Following, a brief description is given of the four steps above.

PSHA involves considering all potential sources of earthquake (fault ruptures) at the vicinity of the structure site, and accordingly, assessing the likelihood of occurring earthquakes with intensity of IM or higher, where IM is a variable measure. The final output of PSHA procedure is the seismic hazard curve which expresses the MAF of exceedance for different IM values.

Structural analysis procedure must incorporate the newest and most reliable methods available for modeling the studied structure as in this research and also must account for two groups of uncertainties i.e. epistemic and aleatory that alter the predicted response of a structure, which are described in more detail in the following sections. The damage analysis and loss analysis are not covered in this study and still need to be addressed in future researches of jacket type platforms, to achieve this, the state of damage experienced by structural and nonstructural components must be calculated using the values of structure responses expressed in terms of determined EDP, i.e. Peak Floor Acceleration (PFA) and maximum inter-story drift ratio (MIDR), obtained from IDA results and losses consequent to occurrence of the aforementioned damage states. In other words this research covers only the first two terms from right in Eq.1.i.e. PSHA and structural analysis as will be illustrated in continue.

**ANALYTICAL MODELING**

The primary structural components of jacket type offshore structures include topside, jacket and pile foundations. Overall structural behavior of this type of platform subjected to lateral loads greatly depends on non-linear behavior of tubular members and pile–soil-structure interaction. In this paper, a non-linear fiber element termed as “Fiber Beam-Column Post Buckling Element” is used for the simulation of jacket member behavior in the non-linear range of deformation and also for the modeling of pile–soil-structure interaction. This element is a flexibility based element and its shape function varies as the state of the element changes without introducing additional nodes or elements [5]. This element type is an inelastic fiber beam-column element capable of accounting for buckling, post buckling and distributed plasticity, which has been set up by Asgarian, et al. [4].

### 3.1. Jacket member modeling

In this paper a non-linear fiber element for the simulation of buckling, post-buckling and hysteretic responses of tubular struts and non-linear behavior of portals is used. The element is an inelastic fiber beam-column element capable of accounting for buckling and distributed plasticity. In this element, stiffness and strength degradation of tubular members under seismic excitation are predicted accurately. A transition curve has been provided for this material in intersection of the branches (i.e. tangent moduli) to avoid any sudden change in local stiffness matrices and to ensure a smooth transition between the elastic, Pre-capping and post-capping plastic regions. Several assumptions have been made for the analyses which are as the following:

- The secondary deformation effects are considered in all of the analyses.
- Initial imperfection equal to 0.5% of the brace length is applied to all braces for buckling and post-buckling assessment.
• Strain Hardening for all structural steel of the jackets is assumed to be 3%.
• Number of segments for all members is taken as 10 and number of cross sectional fibers is set to 16 for accurate monitoring of all member behavior.
• Non-linear properties of the structural steel used in the jackets are introduced by the Stress–Strain curve.
• This steel conforms to (API 5L-Gr.X52 [5]) material grade, which is commonly used in steel jacket frames and piles.

3.2. Equivalent pile stub
In this paper non-linear pile stubs instead of the actual pile–soil properties are implemented. The pile stub has the same properties as the pile, but the relative equivalent length is much smaller and is based on previous pile–soil-structure interaction experiences and could be computed based on pile head forces. In other words, it is assumed that the pile has sufficient fixity to resist lateral loads at the end of the pile stub depth. The lateral displacement, lateral forces and moments acting on the steel pile body dissipate at this point. This point is referred as the “Fixity Point” hereby. The fixity point depends on the pile head forces and soil properties but as per previous offshore experiences, assuming the fixity point within the 6–10 times of the pile diameter range is accurate enough for such analyses.

INCREMENTAL DYNAMIC ANALYSIS

4.1. Ground Motion Histories
For performing the IDAs 44 (22 pairs) ground motion time histories introduced and recommended by FEMA P695 [6] are used. The range of three seismological parameters of ground motions are given in table1. More details of corresponding events can be found in FEMAP695[6].

Table 1. Range of values for three seismological parameters of ground motion histories.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Minimum</th>
<th>Average</th>
<th>Maximum</th>
</tr>
</thead>
<tbody>
<tr>
<td>Magnitude</td>
<td>6.5</td>
<td>7</td>
<td>7.6</td>
</tr>
<tr>
<td>$PGA_{max}(g)$</td>
<td>0.21</td>
<td>0.43</td>
<td>0.82</td>
</tr>
<tr>
<td>Site-to-source epicentral distance</td>
<td>11.1</td>
<td>16.4</td>
<td>26.4</td>
</tr>
</tbody>
</table>

1 maximum between two orthogonal directions

4.2. IDA Curves
IDA is a series of nonlinear analyses for ground motions that is increased in amplitude until an IM level is reached at which dynamic instability occurs (the last point at which the solution converges). This process is based on a very inherent assumption that same ground motion can be used to evaluate the dynamic response of the structure- over a variety of intensity levels of ground motions- from elastic region and nonlinear regime up to global instability. This entails appropriately scaling each record to cover the entire range of structural response, from elasticity, to yielding, and finally global dynamic instability. In order to establish an automated, still time-efficient, procedure for performing IDAs, the “hunt and fill” algorithm [7] is utilized. By performing the IDA for each record, a large amount of data can be achieved and the IDA curves display a wide range of behavior, showing large record-to-record variability, thus this is essential to summarize such data and quantify the randomness introduced by the records IDA curves of the structure. For this purpose stripes of EDP values are calculated at several levels of the IM and the 16, 50, and 84% fractile EDP values given the IM level are estimated and shown in figures 1 in order to have more tangible insights of structures’ behaviors. under above mentioned records.
In order to be able to carry out the performance based assessment, limit-states on the IDA curves should be explicitly defined. Global dynamic instability (GI) which is known as sides way collapse of structure in recent researches and hereafter.

Sides way collapse of frame structures (GI) is a limit state that in which a specific story, or a series of them, displaces dramatically due to P-Delta or/and component deterioration effects and dynamic instability occurs and it will lead to complete loss of gravity load resistance in structural systems. Since sides way collapse happens when the flat line is reached and any increase in the IM results in practically infinite EDP response. Sides way collapse capacity is associated as the projection of the last point on the IM axis at which the structure has been stable (i.e., the slope of the IDA curve approached zero) for any individual record. Furthermore researchers concluded that correlating the structure’s collapse potential with the maximum ground motion intensity (here Sa) at which the structural system still maintains dynamic stability, does account for redistribution of damage and the ability of the structural system to sustain excessive component deformations.

**PERFORMANCE EVALUATIONS**

Performance evaluation procedures considered in this study involve the estimation of a level of confidence that a structure will be able to meet a desired performance objective and mean annual frequency of exceeding a specified structural capacity. Herein, seismic reliability assessment based on IM-based researches done by Krawinkler et al. [8] is provided in a probabilistic framework for considered structural limit states.

Furthermore, concepts described in these approaches can be used for a probabilistic-based design for the desired performance levels. Performance design targets could be expressed, using the proposed methodologies results (i.e. the confidence levels at given hazard levels and MAF of interest) for instance minimum design confidence levels of 90% for potential controlling behavior of global stability mode (as recommended by FEMA350) against 2/50 hazard level and/or performance target of a tolerable MAF of sideways collapse of 0.0002.

**5.1. Uncertainty and it's treatment**

Sources of uncertainty in performance-based seismic evaluation of structural systems are differentiated into aleatory and epistemic. The aleatory uncertainty reflects the small frequency and the large uncertainty associated with earthquakes or variability in structure response due to random nature of ground motions (record-to-record (RTR) variability).The epistemic uncertainty is mainly due to inadequacy or lack of knowledge, including inability to incorporate all elements that may affect lateral strength and stiffness as well as deterioration properties, etc, in the structural model and in the real element properties. For further notational simplicity throughout when the distinction is critical, the relevant parameters will be subscripted by a R or U for epistemic and aleatory uncertainties respectively.

One of the most common method used for incorporating the effect of epistemic uncertainty in the estimation, is inflating the dispersion due to record to record variability $\beta_R$, to square-root-of-the-sum-of-the-squares of $\beta_R$ and $\beta_U$ as shown in Equation (2) [9]. Since this method provides a mean estimate of the performance-based seismic evaluation, it is denoted as the "mean method":

$$\beta_{\text{eq}} = \sqrt{\beta_R^2 + \beta_U^2}$$  \hspace{1cm} (2)
Another method for incorporating the effect of epistemic uncertainty is by changing the median of median value \( \hat{\eta} \) to \( \eta^X \) which has the confidence \( X \) in any estimate of \( \hat{\eta} \) or the probability \( X \) that the actual median value is greater than \( \eta^X \). Hence this method is referred to as ‘confidence method’:

\[
\eta^X = \hat{\eta} e^{-\beta_X K_X}
\]

\( K_X \) is the standard Gaussian variant associated with probability \( X \) of not being exceeded found in conventional probability tables, e.g., if \( K_X = 1.28 \) then \( X = 90\% \).

5.2. IM-based approach:

In this alternative approach the capacity for desired performance level is defined as the ground motion intensity that is associated with defining limit state point rather than engineering demand parameter. This approach may be used to develop confidence curves for specific performance level that can be employed directly for estimation of level of confidence associated with any particular hazard level or determination of the mean annual frequency (MAF) of desired performance level by integrating it over the hazard curve.

Given the structural system and representative ground motion records for the location of the structure, one can develop confidence curves by setting elements’ properties in mathematical model to their median value and then performing IDAs as discussed in the previous section.

Using the assumption discussed in sec.4.1, it became possible to obtain the confidence level or the probability of structure to satisfy the desired performance at given hazard level with \( Q \) confidence by following these steps:

1. Assigning a certain distribution (normally a lognormal one) to desired capacity points on IDA curves to find the median estimate of median spectral acceleration \( \hat{\eta}_\text{C} \) and the dispersion due to record to record variability \( \beta_{\text{RC}} \) and estimating the dispersion due to epistemic uncertainty in the structural model \( \beta_{\text{UC}} \);

2. Using the cumulative lognormal distribution function \( \phi() \), the ‘confidence curve’ for desired performance levels with regard to both aleatory and epistemic uncertainties can be obtained. Mean estimate of such a confidence curve is represented by Equation (3) Using the ‘mean method’, with \( Q \) confidence at a given value of \( Sa_C \) calculated by Equation (4).

\[
\mathcal{C}_{\text{L}} = 1 - \Phi \left( \frac{\text{Ln}(Sa_C) - \text{Ln}(\hat{\eta}_\text{C})}{\beta_{\text{RC}}} \right)
\]

\[
Q = 1 - \Phi \left( \frac{\text{Ln}(Sa_C) - \text{Ln}(\hat{\eta}_C)}{\sigma} \right)
\]

In which \( \hat{\eta}_\text{C} \) is the median estimate of median spectral accelerations (\( Sa_C \) points) and \( \sigma = \beta_{\text{RC}} \beta_{\text{UC}} / (\beta_{\text{RC}} + \beta_{\text{UC}}) \). Another type of the ‘confidence curve’ for desired performance levels can be achieved utilizing "confidence method" for accounting epistemic uncertainty, by Equation (6)

\[
\mathcal{C}_{\text{L}} = 1 - \Phi \left( \frac{\text{Ln}(Sa_C) - \text{Ln}(\eta^X)}{\beta_{\text{UC}}} \right)
\]

In order to disregard the effect of epistemic uncertainty, one could simply substitute \( K_X = 0.0 \) in Equations (3) and (6). In such case, the associated confidence will be 50%.

3. Using the mean hazard curve obtained for the structure, the spectral acceleration value that corresponds to the considered hazard level can be correlated to the confidence level through related confidence curve.

Mean spectral accelerations at the 63/50, 10/50 and 2/50 hazard levels for periods of studied Structure (2.18 sec) are obtained from site specific hazard analysis [10] and is used to find the confidence levels. For estimation of Mean Annual Frequency (MAF). These data are supplemented by hazard curves regressed through three IM values by fitting a ‘power function’. Choice of ‘power function’ is based on assuming a linear form for the hazard curve in the log–log domain. Figure 2 shows these hazard curves along with spectral accelerations at the 63/50, 10/50, and 2/50 hazard levels. Utilizing the regressed hazard curves will not affect significantly more frequent events [8] But As seen in Figure 2, may result in unrealistic larger mean recurrence interval values of earthquakes, which is due to the type and unity of the function used for this regression in all domains. This may have an effect on the MAF of sides way collapse considering large tolerable median spectral acceleration of this limit state.
As discussed previously, in order to estimate the MAF of desired performance level, one needs to integrate the cumulative lognormal distribution of spectral acceleration capacity points, over the mean hazard function. By using some simplifier assumptions this integral would turn into a close form solution; equation (7) and (8) give MAF with confidence \( X \) and mean estimate of the MAF of exceeding desired performance level, respectively [11].

\[
\lambda_c^X = \left[ \overline{\lambda}_{\alpha}(\hat{\eta}_C) \right] \left[ \exp(1/2k^2\beta_{\alpha}^2) \right] \exp[K_c(k\beta_{\alpha})] \\
\lambda_c^{-} = \left[ \overline{\lambda}_{\alpha}(\hat{\eta}_C) \right] \left[ \exp(1/2k^2\beta_{\alpha}^2) \right]
\]

This approach has been just implemented for sides way collapse assessment and by using the FOSM method, It is suggested that \( \beta_{UC} \) for sides way collapse capacity of moment-resisting frame is in order of 0.4 [8]. More accurate estimation of \( \beta_{UC} \) - the dispersion due to *epistemic* uncertainty in the structural model- still have to be addressed in future and necessitates comprehensive research.

<table>
<thead>
<tr>
<th>Table 2. Calculation of confidence level.</th>
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<tr>
<td>Structure</td>
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<tr>
<td>------------</td>
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<tr>
<td>SPD19A</td>
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Figure 3 shows confidence curves with different confidence levels for the structure. The median estimate of the median of sides way collapse capacity, \( \hat{\eta}_C \) and dispersion due to *aleatory* uncertainty, \( \beta_{UC} \), are obtained from IDA curves. The dispersion due to *epistemic* uncertainty, \( \beta_{UC} \), is set to 0.4 [8]. The collapse confidence curve denoted with ‘(Mean)’ is obtained using the ‘mean method’. In addition using the alternative confidence method, collapse confidence curves with the associated confidence of 50% (excluding *epistemic* uncertainty) and 84% are shown for each structure. In table 2, the values associated with the 2/50 hazard level, for the structures is used to calculate the confidence level correlated with each curve. we also obtained the MAF of collapse associated with 50% (excluding *epistemic* uncertainty) and 84% confidence and mean of MAF of collapse from Equation (7and 8). see table 3.

<table>
<thead>
<tr>
<th>Table 3. The annual probability of the sides way collapse</th>
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<td>Structure</td>
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Results and Discussion

Minimum confidence levels of sides way collapse of 83% at the 2/50 hazard level in the mean sense is satisfied. Moreover, in all cases shows the tolerable mean estimate of MAF of collapse is less than 0.0002. These performance targets imply that Jackat are configured and detailed to provide a high level of system ductility and acceptable potential controlling behavior of global stability mode. Furthermore, it should be noted that the main reasons that a relatively lower confidence level is calculated for Jackat structure is that it is flexible to some extent which increases the fundamental period and decreases the corresponding spectral acceleration of the same hazard level comparing to other offshore structures and also we see a significant difference in the calculated confidence levels and MAF of collapse by using different incorporation of epistemic uncertainty in obtaining the confidence levels which could be due to the relatively large value of $\beta_{UC}$.

Conclusions

This paper provides a summary of presently-used procedures for seismic reliability assessments and performance evaluations of structural systems. And two proposed measures of level of confidence that a structure will be able to meet a desired performance objective and mean annual frequency of exceeding a specified structural capacity, are illustrated and estimated for typical offshore structure, considering response of different height structures that are located in a high seismic zone.

Furthermore, accurate analytical modeling for seismic loads is included with introduced model for structural components which incorporates pile–soil-structure interaction under cyclic loading, considering a bilinear hysteretic model.

Seismic reliability assessments were carried out for the commonly-used limit state of global dynamic instability (GI) or sides way collapse of structure. Results show that special consideration and provisions imposed for the offshore platforms systems dedicate superior deformability which can be implied by IDA and its post-processing or more obviously can be pronounced by calculated aforementioned measures.

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