Seismic Performance Evaluation of Steel Jacket Platform with Float over Deck Systems

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SUMMARY
The different vertical bracing configuration at each direction of steel jacket type offshore platforms with float over Deck installation system affected to the seismic behavior of structure. This paper presents the result of seismic assessment of an existing platform with FOD system in Persian Gulf. For this purpose, the incremental nonlinear dynamic analysis of the platform in both directions is performed by considering structure-pile-soil interaction and nonlinearity in material and geometry of structural elements. Then, the mean annual frequency of exceeding a specified limit state is evaluated by estimating the demand and capacity from IDA results. The aleatory uncertainty and the epistemic uncertainty is also considered in the estimating seismic hazard, structural response and capacity. The results show that platforms with FOD system could not satisfy API-RP-2A design requirements in direction that braces are removed for float-over deck installation operation. So, the new bracing system was proposed for weak directions to improve the seismic performance of this type of platforms.

Keywords: Steel jacket offshore platform, Float-over deck, Mean annual frequency.

1. INTRODUCTION

Float Over Deck (FOD) installation is an alternative to lift-installing integrated decks using heavy lift crane vessels. In a float over the topside module typically is placed on a barge or heavy transport vessel positioned within or around the legs of a pre-installed substructure. The module then is settled onto the jacket legs by a combination of vessel ballasting and a mechanical lowering system. The bracing elements of structure must be removed at topside of jacket in one direction to entry the barge within the jacket legs. Evaluating earthquake performance of this type 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 configuration of bracing system in structure. The SSPSI (Seismic Soil-Pile- Structure Interaction) analysis is the main step in evaluation of seismic behavior of pile supported offshore platforms. The pile-soil interaction problem during earthquake loading as one of the most important sources of nonlinear dynamic response analysis of offshore platforms has received considerable attention in recent years.

An accurate evaluation of the structural dynamic behavior subjected to seismic loading is one of the critical issues in Performance-based earthquake engineering (PBEE) methodology. In particular, the recognition of weakness and the estimation of loss due to damage in structure depend on exact and realistic estimation about the performance of the structure. Recent advances in the field of earthquake engineering are quickly paving the evaluation of the seismic performance of structures. In order to evaluate the seismic risk and performance of structures, the SAC-FEMA (FEMA 2000 a, b) developed a method that permits probability assessment in a closed form. It represents a part of broader PEER (Pacific Earthquake Engineering Research Center) probabilistic framework. Within the framework of the SAC-FEMA method, incremental dynamic analysis (IDA), developed by Vamvatsikos and Cornell

(2002), usually determines the relationship between the seismic intensity and the engineering demand parameter. IDA is a powerful tool for the estimation of seismic demand and capacity for multiple levels of intensity. It involves subjecting a structural model to ground motion records, each scaled to multiple levels of intensity, and producing curves of response parameterized versus intensity level. The IDA curve contains the necessary information such as performance levels or limit-states that are important ingredients of PBEE methodology. The main power of the IDA method is that it can be used well in a probabilistic framework, where the probability of exceeding a specified limit state for a given intensity can be found. Using above parameters, the capacity, mean annual frequency (MAF) of exceeding a specified level of the structural demand and return period of structure can be calculated and compared with a current provision (such as American petroleum institute, API-RP-2A 2000) to evaluate the behavior of structures (Vamvatsikos, Jalayer, Cornell, 2003).

Evaluating earthquake performance of steel jacket type offshore platform in seismic active areas is one of the most important parts in design of this type of structure. Dynamic response of offshore platforms is a function of the loading characteristics, dynamic pile-soil interaction behavior and configuration of bracing system in the structure. In this paper, seismic performance of a newly designed steel jacket type offshore platform with the FOD installation system in Persian Gulf has been performed by considering soil-pile-structure interaction using incremental nonlinear dynamic analysis. The probabilistic performance objective for the assessment of this structure was performed taking into account soil-pile structure interaction, material and geometric nonlinearity of structure elements. The mean annual frequency of exceeding for each direction of structure (X and Y) was derived by taking into account the aleatory uncertainty (due to inherent randomness) and the epistemic uncertainty (due to limited knowledge) in three main elements including seismic hazard, structural response (as a function of ground motion intensity) and capacity. By comparing the mean annual frequency of the platform in two directions, it was distinguished that the jacket type offshore platform with the FOD system could not satisfy the criteria in the FOD direction.

2. FLOAT OVER DECK INSTALLATION SYSTEM

Topsides vary in weight, size, and configuration. Small decks have been traditionally installed as one unit using low capacity cranes vessels and jack-ups. Medium to large decks have been either modularized to facilitate installation with small crane, or built as integrated decks and installed by heavy-lift crane vessels (HLCVs) or by floating them over the substructure. (O'Neill and et al. 2000)
For lift installation, the crane availability, crane capacity, crane suitability and installation risk are usually considered. The majority of the medium-to-large capacity crane vessels operate generally in areas with extensive offshore oil and gas infrastructure such as the Gulf of Mexico and the North Sea. In areas remote from these locations, such as Persian Gulf, South China Sea, Australasia, Canada, etc. installation by HLCVs becomes a less attractive option. The float over deck installation was developed and the applicability was widened to heavier integrated deck and harsher environments. It is proving to be a competitive alternative for such an offshore installation condition. In this procedure, the deck is floated over the substructure legs (Salama, Suresh and Gutierrez 1999). Utilizing float over vessels to install heavy deck loads is not a new practice. It was first patented in 1862 by John Dubbios for installing truss bridges onto piers (Beal and Datta 1992). Float over was first utilized in the oil and gas industry as early late 1970s, but has gained prominence in recent years with major installations. This system has been employed in both sheltered water and open water. The largest ever float-over deck in open waters was 10000 tons, while decks in excess of 50000 tons have been floated onto structures in sheltered waters (O'Neill and et al. 2000).

At the FOD installation system, the vessel is moved into the jacket and transferred from the stand-off location to the correct location in jacket. Alignment of the vessel stern with jacket slot, limiting lateral impact loads on the jacket, no vertical impact loads between deck legs and jacket legs, and control over the movement of the vessel in the longitudinal and transverse direction are the most important considerations. Once the vessel is docked, the deck legs need to be aligned with the jacket legs pre-mating of the integrated deck to jacket. The tolerance for this alignment is mainly driven by diameter of the stabbing cones. By mating of the integrated deck on the jacket, the load of the deck will be transferred from 100% support on the installation vessel to 100% support on jacket legs. The transfer of the deck weight can be achieved by a variety of method such as ballasting of the installation vessel or active hydraulics in the deck supports (O'Neill and et al. 2000). The sequence of the float over deck installation is shown in figure 1.

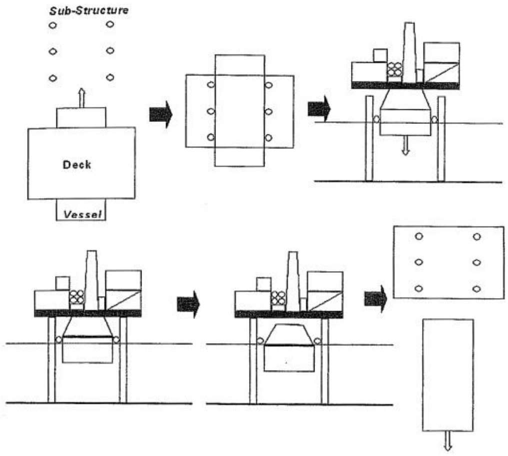

Figure 1. Float over deck installation sequence.

3. PROBABILISTIC SEISMIC ASSESSMENT

In the probabilistic framework discussed, the performance objective is stated in terms of a target or desired mean annual frequency of exceeding a performance level. SAC/FEMA (FEMA, 2000 a, b), Pacific Earthquake Engineering Research Center (Cornell and Krawinkler, 2000) and Vamvatsikos and Cornell (2002a) proposed a process for calculation the MAF of exceeding values. Jalayer and Cornell (2003) derived a closed-form analytical expression for mean annual frequency of exceedance by taking into account the aleatory uncertainty (due to inherent randomness) and the epistemic uncertainty (due to limited knowledge) in three main elements, seismic hazard, structural response (as a function of ground motion intensity) and capacity. The capacity variable defines the limiting value for the demand variable. Both demand and capacity in this framework are expressed as a displacement-based structural response such as a peak roof drift, the floor peak interstory drift angles of an n-storey structure, and the maximum peak interstory drift ratio. In this paper, the maximum peak interstorey drift is chosen as the structural response representing the structural demand and capacity variable (Jalayer and Cornell, 2002). The structure response that represents the nonlinear dynamic behavior of structure is usually expressed by two variables, intensity measure (IM) and damage measure (DM). These variables are normally obtained from nonlinear analysis result. Incremental dynamic analysis (IDA) is a parametric analysis method for estimating the structural performance under seismic loads that exhibit the structure behavior from the elastic response to final global dynamic instability (collapse) (Jalayer and Cornell, 2002). The IDA curves can generate from the result of IDA analyses, measured by a damage measure (DM, maximum peak roof drift θ_{max}), versus the ground motion intensity level, measured by an intensity measure (IM, the 5%-damped first-mode spectral acceleration Sa(T1;5%)). By the multi-record IDA curve, mean, median and 16%, 84% IDA curves can be defined (Vamvatsikos and Cornell, 2002). For a set of drift demand (DM) and spectral acceleration data point, the 2-parameter power-law model considered to express the relationship between spectral acceleration (IM) and (median) interstory drift (DM) (Jalayer and Cornell, 2003).

$$\eta_D(x) = a.x_a^b \tag{3.1}$$

Where η_D is the median of the demand, x is the spectral acceleration. By assuming the randomness in the demand due to a record-to-record variability and limited number of records and uncertainty in the estimation of median value, caused by using finite number of nonlinear analyses, the relationship between median drift demand variable and spectral acceleration can be presented as equation 2. A similar relationship can be derived for the capacity variable with considering the randomness and uncertainty in the estimation of median value (Jalayer and Cornell, 2003).

$$D = \eta_D(x).\varepsilon_{UD}.\varepsilon_{RD} = a.x_a^b.\varepsilon_{UD}.\varepsilon_{RD} \tag{3.2}$$

$$C = \eta_C(x).\varepsilon_{UC}.\varepsilon_{RC} \tag{3.3}$$

Where $\varepsilon_{UD}, \varepsilon_{RD}, \varepsilon_{UC}$ and ε_{RC} are uncertainty and randomness in drift demand and capacity evaluation. It is assumed that these variables are independent and have lognormal distributions with the following properties (Jalayer and Cornell, 2003).

$$\eta_{\varepsilon_{RD}} = \eta_{\varepsilon_{UD}} = e^{mean(\ln(\varepsilon))} = 1, \quad \sigma_{\ln(\varepsilon_{RD})} = \beta_{RD}, \quad \sigma_{\ln(\varepsilon_{UD})} = \beta_{UD} \tag{3.4}$$

$$\eta_{\varepsilon_{RC}} = \eta_{\varepsilon_{UC}} = e^{mean(\ln(\varepsilon))} = 1, \quad \sigma_{\ln(\varepsilon_{RC})} = \beta_{RC}, \quad \sigma_{\ln(\varepsilon_{UC})} = \beta_{UC} \tag{3.5}$$

The spectral acceleration hazard curves, normally provided by seismologists, provide the mean annual frequency of exceeding a particular spectral acceleration value for a given period and damping ratio. These curves are usually approximated by a power-law relationship (DOE 1994 and Luco and Cornell 1998).

$$H_{S_a}(S_a) = P[S_a \geq x] = k_0.x^{-k} \tag{3.6}$$

k_0 and k are the parameters that define the shape of the hazard curve. The hazard curve estimation involves many scientific assumptions, so there is uncertainty in estimation of a spectral acceleration hazard curve. The uncertainty in spectral acceleration hazard is introduced by a lognormal random variable ε_{UH} whose statistical parameters have the following characteristics.

$$median(\varepsilon_{UH}) = \eta_{\varepsilon_{UH}} = e^{mean(\ln(\varepsilon))} = 1, \quad \sigma_{\ln(\varepsilon_{UH})} = \beta_{UH} \tag{3.7}$$

So the mean spectral acceleration hazard can be stated as:

$$\overline{H}_{S_a}(x) = H_{S_a}(x).mean(\varepsilon_{UH}) = k_0.x^{-k}.e^{\frac{1}{2}\beta_{UH}^2} \tag{3.8}$$

The mean annual frequency of exceeding a specified limit state, denoted by H_{LS}, is defined as the product of the mean rate of occurrence of events with seismic intensity larger than a certain minimum level, ν, and the probability that demand, D, exceeds, C, when such event occurs. H_{LS} can be determined by decomposing the above expression into tractable pieces (equation 3.9), seismological part and structural engineering part, and using the total probability theorem (TPT) (Jalayer and Cornell, 2003). The equation 3.9 can be solved by separating it into three element, demand and capacity variable and spectral acceleration hazard. The closed-form analytical expression of H_{LS} after solving the above equation is stated as:

$$H_{LS} = \nu.P[D \geq C] = \int \nu.P[D \geq c] f_C(c).dc \tag{3.9}$$

$$H_{LS} = k_0.\left(S_a^{\eta_C}\right)^{-k}.e^{\frac{1}{2}\beta_{UH}^2}.e^{\frac{1}{2}\frac{k^2}{b^2}(\beta_{RD}^2+\beta_{UD}^2)}.e^{\frac{1}{2}\frac{k^2}{b^2}(\beta_{RC}^2+\beta_{UC}^2)} \tag{3.10}$$

where $S_a^{\eta_C}$ is equal to $(\eta_C / a)^{1/b}$.

4. PROBABILISTIC SEISMIC ASSESSMENT OF A PLATFORM WITH FOD SYSTEM

In this section, the mean annual frequency of a newly designed steel jacket offshore type platform with float-over deck installation system in Persian Gulf, ESPQ1, is evaluated. For this purpose, the full 3D-model of jacket, deck, piles and surrounding soil was studied. The structure (jacket and deck) is about 102m high and the water depth at the site is about 72 m. An illustration of the steel frame is shown in Figure 2. Eight skirt piles that are located in the corner of structure connect the piles to the jacket. The mean mechanical properties of the structural elements were used for the analysis. The total weight of the jacket and deck is about 8680 tons. The platform has a five-stories topside with weight of about 5440 tons and a four-story jacket with weight of about 3240 tons. To accommodate the platform heavy topside installation using the float-over system, there is no brace in the sea water level bay in the direction X and a portal action is performed in this direction. The main characteristics of the offshore are listed in Table 4.1.

Table 4.1. The main characteristics of the ESPQ1 platform

Water depth	72 m
Platform height	102 m
Jacket dimensions (horizontal plan)	31 m (X-Dir) × 34 m (Y-Dir)
Total No. of jacket legs	6
Total No. of jacket piles	8
Jacket piles	Skirt Piles
Pile Dimension & Penetration	(72 × 1.5in) – 62m

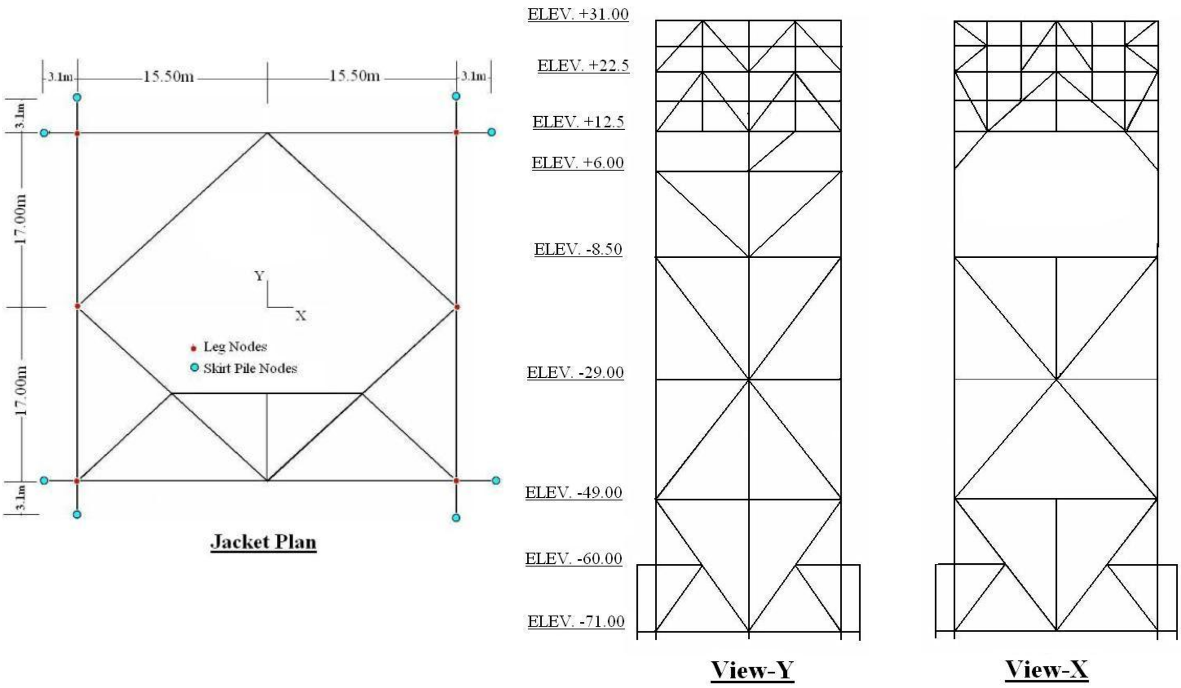

Figure 2. The plan and views of ESPQ1 steel jacket platform with FOD system.

4.1. Modeling of Structure

The computational model of the steel jacket offshore platform was developed using OPENSEES software (Mazzoni and et al. 2006). All members were modeled using beam-column element. The fiber section was considered in the section of all elements, and p-delta matrix stiffness was used for the geometric nonlinearity that is accurate enough for such an application. The mass used in the dynamic analysis consist of the mass of the platform associated with gravity loading, the mass of the entrapped fluids in main legs, and the hydrodynamic added mass. The added mass was estimated as the mass of the displaced water for motion transverse to the longitudinal axis of the individual structural members and appurtenances. In computing the dynamic response of the pile supported steel structures, viscous damping ratio of 2% was used for the analysis. The pile and soil and the soil-pile-structure interaction was also modeled by BNWF model, which is introduced in following section.

4.1.1. Pile-soil interaction analysis using BNWF model

The beam on nonlinear Winkler foundation models (BNWF), used for analyzing the dynamic response of the piles, should allow for the variation of soil properties with depth, nonlinear soil behavior, nonlinear behavior of pile-soil interfaces and energy dissipation through radiation and hysteretic damping (Boulanger and et al. 1999). Special attention must be given to the evaluation of the free-field excitation. The computed ground motion at different levels within the soil was applied to the nodal boundary supports representing the support motions (Asgarian and Aghakouchack, 2004).

4.1.2. Free Field Excitations

Free field ground motion time histories are usually computed using common site response analysis techniques. In site response analysis, the ground motion of the soil layer is calculated due to earthquake excitations applied at bedrock. The results of such free field analysis (acceleration or displacement time history at different soil layer) are then used as the input excitation at support nodes of the BNWF-Fiber Element model (Asgarian and Aghakouchack, 2004). In present study, the nonlinear stress-strain response of soil layers was approximated by a nonlinear approach. In the analyses, Iwan (1976) and Morz (1967) model was used in which the nonlinear and hysteretic stress-strain behavior of soil is approximated by tangential shear modulus. Computer program NERA (nonlinear earthquake site response analysis) developed by Bardet and Tobita (2001) was used for free field ground motion analysis. The lowstrain shear modulus G_{max} was calculated from the dimensionless form of the equations by Seed and Idriss (1970):

$$\frac{G_{\max}}{P_{atm}} = 21.8K_{2,\max}\sqrt{\frac{\sigma'_m}{P_{atm}}} \quad for\ \ Sand \tag{4.1}$$

$$K_{2,\max} = 65 \qquad \sigma'_m = (1+2K_0)\sigma'_{vc}/3 \qquad K_0 = 0.6 \qquad P_{atm} = atmospheric\ \ \mathrm{Pr}essure$$

$$\frac{G_{\max}}{c_u} = 380 \quad for\ \ \ Clay \tag{4.2}$$

4.1.3 Pile and Soil Modeling

The pile and surrounding soil were subdivided into a number of discrete layers. Pile response was traced independently at nodal points of the pile segments within each layer. The dynamic characteristics of the pile segments (i.e. stiffness, damping and mass) were established at these nodes. The soil reaction to pile movement during transient seismic loading comprises stiffness and damping components. In present study, the soil stiffness was established using the p-y (lateral soil resistance versus lateral soil deflection) as well as t-z and Q-z spring elements. The procedures for generating p-y curves proposed by Matlock (1970), Reese and Welch (1975) and O'Neil and Murchison (1983) are recommended by the American petroleum institute and are widely used in both research and professional jobs. Therefore, the soil stiffness was modeled, employing the static p-y, t-z and Q-z curves recommended by API. Furthermore, the damping component of the soil resistance was represented by a dashpot whose coefficient was established based on the Berger, Mahin, and Pyke (1977) model, i.e.,

$$C_L = 4B\rho\nu_s \tag{4.3}$$

Where B is pile diameter, ν_s is the soil shear wave velocity and ρ is the soil unit density. For transferring acceleration from bedrock to soil layers, its characteristics, layers, and selected record were introduced in NERA software (Bardet and Tobita 2001). The time history of relative displacement at a selected sub layer was attained. After the formation of the model, the time history of relative displacement of soil column in pile nodes (obtained from NERA) was applied and the structure was analyzed from result of nonlinear dynamic analysis.

4.1.4. Record Selection

Previous studies (Shome and Cornell, 1999) have shown that for mid-rise buildings, ten to twenty records are usually enough to provide sufficient accuracy in the estimation of seismic demands. Consequently, a set of sixteen ground motion records was selected that belong to a bin of relatively large magnitudes of 6 to 7 and moderate distances.

4.2. Incremental dynamic analysis of the platform

Once the model was formed and the ground motion records were selected, a way to perform the actual nonlinear dynamic analyses, necessary for IDA, was required. This entails appropriately scaling each record to cover the entire range of structural response, from elasticity, to yielding, and finally global dynamic instability (Vamvatsikos and Cornell, 2002). The hunt and fill algorithm was chosen to trace the IDA curves. Analyses were performed at increasing levels of IM by increasing steps, until numerical non-convergence was encountered (Vamvatsikos and Cornell, 2002). An IDA Curve is a collection of IDA curves of the same structural model under different accelerograms that all are parameterized on the same IMs and DM. Figures 4 and 5 show all sixteen IDA curves in the X and Y directions. In these figures, vertical and horizontal axes are defined by first-mode spectral acceleration Sa(T1;5%) and maximum interstory drift ratio.

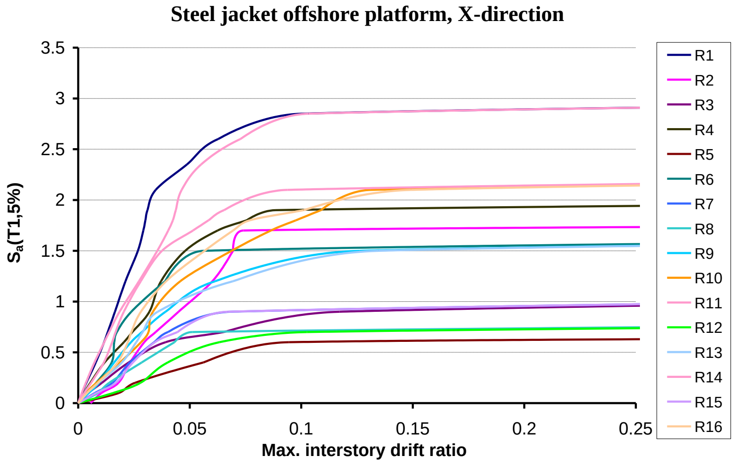

Figure 4. IDA curves (for T1 = 2.9 sec), steel jacket offshore platform in X-direction.

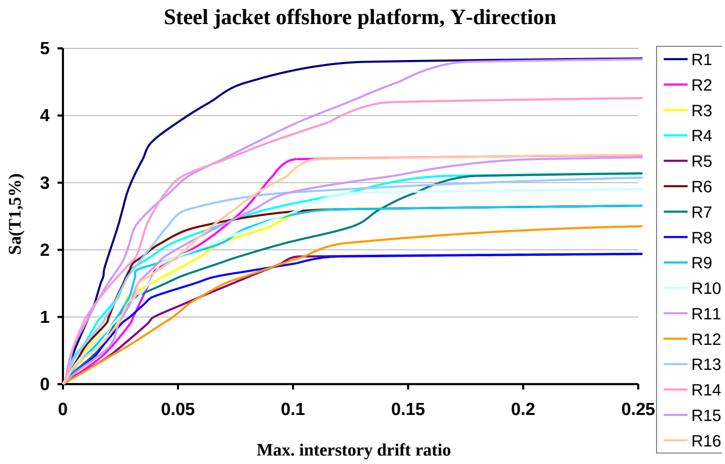

Figure 5. IDA curves (for T1 = 2.67 sec), steel jacket offshore platform in Y-direction.

4.2.1 Defining limit-states

In order to do the probabilistic seismic assessment, it is required to define limit-states. In this study, two limit states were used: immediate occupancy (IO) and collapse prevention (CP), both defined in FEMA (2000 a, b c). According to this provision, it is assumed to set the IO limit-state to appear at θ_{max} = 1%, elastic limit state whichever occurs first or for ground motions with a 50% chance of exceedance in 50 years and the CP limit-state was not exceeded on the IDA curve until the final point where the local tangent reaches 20% of the elastic slope or a drift ratio of θ_{max} = 10%, whichever occurs first in IM terms. The main idea is to place the CP limit-state at a point where the IDA curve is softening towards the flat line but at low enough values of θ_{max} (less than 10%) (for ground motions with a 2% chance of exceedance in 50 years).

4.2.2 Evaluation the MAF

In this section, all parameters that are essential in the probabilistic seismic assessment of structure were obtained from the result of incremental dynamic analysis. For this purpose, it was necessary to summarize IDA curves. They can be easily summarized into some central value (the mean or the median) and a measure of dispersion (the standard deviation, or the difference between two fractiles). Consequently, the 16%, 50% and 84% fractile values of DM and IM capacity were chosen. Figure 6 shows the summarized IDA curve in the X and Y direction. Based on Iranian earthquake code (Standard No. 2800, 2005), the site-specific hazard curve was selected for a site located in Persian Gulf at T = 2.8 sec, T being (close to) the first natural period of the structure. By using the power-law relationship, which was introduced in previous section, the line with slope k and intercept k_0 was fitted to the hazard curve around the region of interest (e.g., MAFs between 1/72 or 50% frequency of exceedance in 50 years, and 1/2475 or 2% frequency of exceedance). Here, k, k_0 and β_{UH} (uncertainty in the spectral hazard acceleration) are 2.73, 0.000196 and 0.65, respectively. The fitted curves to median IDA curves in figure 7 (Curve 50% in the X and Y direction) were calculated as power-law equation. These equations were obtained as follow.

$$\eta_D(x) = ax_a^b = 0.051x_a^{1.47} \;, r = 0.987 \;\; X \;\; Dir \qquad (4.4)$$

$$\eta_D(x) = ax_a^b = 0.0188x_a^{1.59}, r = 0.981 \; Y \;\; Dir \qquad (4.5)$$

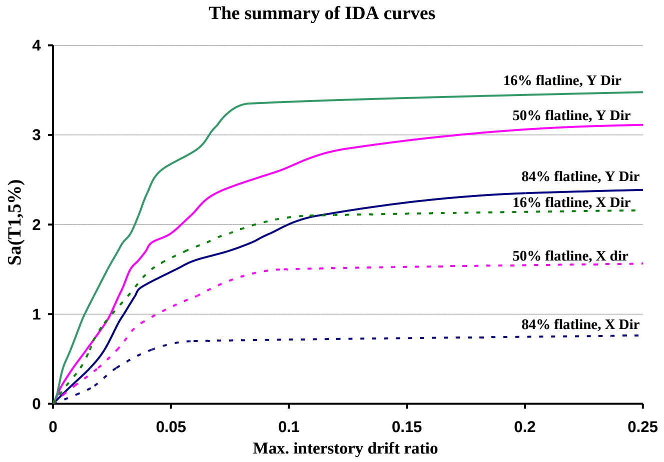

Figure 6. The summary of IDA curves in the X and Y direction of jacket platform

The limit states capacities, C, the fractional standard deviation of demand and capacity, β_{RD} and β_{RC} at each direction were estimated from the result of incremental dynamic analysis and are shown in Table 4.2. There are several practical ways to estimate the fractional standard deviation. In this paper, the fractional standard deviation of demand, β_{RD}, was obtained by conducting a regression analysis of lnD and lnSa for 16%, 50% and 84% fractile values. The fractional standard deviation of capacity, β_{RC}, was estimated here by the average $\ln(S_a^{84^{th}} / S_a^{50^{th}})$ and $\ln(S_a^{50^{th}} / S_a^{16^{th}})$, where the symbols

$S_a^{84^{th}}$, $S_a^{50^{th}}$ and $S_a^{16^{th}}$ denote the spectral acceleration values corresponding to 84%, 50% and 16% percentiles of the ordered data, respectively (Jalayer and Cornell 2002). The modeling errors and other approximations involved in current analysis procedure was limited to the statistical uncertainty in the median due to the finite sample size (n_{sample}) of ground motions. So β_{UD} and β_{UC} can be calculated as:

$$\beta_{UD} = \frac{\beta_{RD}}{\sqrt{n_s}}, \qquad \beta_{UC} = \frac{\beta_{RC}}{\sqrt{n_s}} \tag{4.6}$$

Table 4.2. Summarized capacities, randomness and uncertainty

		Capacity parameters				Randomness				Uncertainty			
	Percentile	θ_{max}		$S_a(T_1;5_{\%})$		β_{RD}		β_{RC}		β_{UD}		β_{UC}	
		IO	CP	IO	CP	IO	CP	IO	CP	IO	CP	IO	CP
X-Dir.	16	0.01	0.055	0.12	0.7								
	50	0.01	0.075	0.22	1.35	0.42	0.44	0.52	0.65	0.105	0.11	0.13	0.163
	84	0.01	0.097	0.34	2.1								
Y-Dir.	16	0.01	0.089	0.25	1.97								
	50	0.01	0.10	0.42	2.58	0.4	0.36	0.58	0.3	0.1	0.09	0.145	0.075
	84	0.01	0.10	0.80	3.57								

The spectral acceleration corresponding to given capacity, $S_a^{\eta_C}$ in equation 3.10 can be found both analytically and graphically. The graphically estimation was used in this paper to increase the accuracy of the MAF estimation. The mean annual exceeded frequencies from each limit state were calculated from equation 3.10 and are shown in Table 4.3. The probabilistic seismic assessment of steel jacket type offshore platform with FOD system reveals that the structure in Y direction has an acceptable mean annual frequency of exceeding the above limit states (Immediate occupancy and collapse prevention). However, the MAF of exceeding the limit states in X direction are not satisfied the recommended performance levels, and are less than the specified value 0.0139 and 0.0004 per annum.

Table 4.3 MAFs of exceedance and corresponding return periods.

	X Direction		Y Direction	
	IO	CP	IO	CP
H_{LS} (MAF of Exceedance)	0.0343	0.00042	0.00643	0.000025
Return Period (years)	29	2380	155	39726

5. CONCLUSIONS

In this paper probabilistic performance based evaluation method is applied for the assessment of a newly designed jacket type offshore platform with the float-over deck installation method. A complete 3D model of platform including jacket, deck, pile and surrounding soil of the pile is considered using capability of the OPENSEES software. For the modeling of material nonlinearity force-based fiber element of OPENSEES and the for geometric nonlinearity, P-delta stiffness matrix of the software is used. In order to evaluate the seismic risk and estimate the demand and capacity of the structure at each performance level (IO & CP), the incremental dynamic analysis is performed using sixteen ground motion records. The mean annual frequency of exceeding a limit state for investigated offshore platform are obtained by taking into account the aleatory uncertainty (randomness) and epistemic uncertainty in seismic hazard, demand and capacity that was derived in closed-form analytical expression by Jalayer and Cornell (2003). The result of probabilistic assessment are shown that the MAF of these type of platforms are not satisfied the both immediate occupancy and collapse prevention performance levels at direction that the braces are removed due to deck installation operation. The result are shown that the steel jacket type offshore platform with FOD system in FOD direction is not satisfied the MAF of exceeding a limit states that recommended in API-RP-2A and FEMA 350 codes.

REFERENCES

American Petroleum Institute, (2000). Recommended practice for planning, designing and constructing fixed offshore platforms. *API Recommended Practice 2A (RP-2A) 21st ed. American Petroleum Institute, Washington, D.C.*

Asgarian, B., Aghakouchack, AA., (2004). Nonlinear Dynamic Analysis of Jacket Type Offshore Structures Subjected to Earthquake Using Fiber Elements. *13th World Conference on Earthquake Engineering*. **Paper No. 1726**.

Bardet, JP, Tobita T., (2001). NERA- a computer program for Nonlinear Earthquake site Response Analysis of Layered Soil Deposits. *Department of Civil Engineering, University of Southern California*.

Beal, D. D. and Datta, B. N., (1992). Design and Installation of North Dauphin Island Platform. *Offshore Technology conference, Houston, Texas*.

Berger, E., Mahin, S.A., and Pyke R. (1977). Simplified method for evaluating soil-pile-structure interaction effects. *Proceedings of the 9th offshore Technology Conference*, **OTC paper 2954,** Huston, Texas. 589-598.

BHRC (2005). Iranian Code of Practice for Seismic Resistance Design of Buildings: Standard No. 2800 (3rd edition). *Building and Housing Research Center.*

Boulanger, RW., Curras, CJ., Kutter, BL., Wilson DW., Abghari, A., (1999). Seismic soil pile structure interaction experiments and analysis. *Journal of Geotechnical and Geoenvironmental Engineering*, **ASCE, 125(9), 750-759.**

Cornell, C.A., and, Krawinkler, H., (2000), Progress and Challenges in Seismic Performance Assessment. *PEER newsletter.*

DOE (1994), Natural phenomena hazards design and evaluation criteria for Department of Energy Facilities. U. *S. Dept. of Energy, Washington, D. C.*, **DOE-STD-1020-94.**

FEMA 350 (2000), Recommended seismic design criteria for new steel moment-frame buildings. *SAC Joint Venture, Federal Emergency Management Agency, Washington, DC*.

FEMA 351 (2000), Recommended seismic evaluation and upgrade criteria for Existing Welded Steel Moment-Frame Buildings. *SAC Joint Venture, Federal Emergency Management Agency, Washington, DC*.

FEMA 352 (2000), Recommended post-earthquake evaluation and repair criteria for welded steel moment-frame buildings. *SAC Joint Venture, Federal Emergency Management Agency, Washington, DC.*

Iwan, W.D. (1976). On a class of models for the yielding behavior of continuous and composite systems. *Journal of Applied Mechanics*, **ASME, 34: 612-617**.

Jalayer F. and Cornell C.A., (2003). A technical framework for probability-based demand and capacity factor design (DCFD) seismic formats. *Department of Civil and Environmental Engineering, Stanford University, Stanford, CA,* **Report No.08 to the PEER Center**.

Jalayer, F. and Cornell, C. A., (2002). Alternative nonlinear demand estimation methods for probability-based seismic assessments. *Earthquake Engng Struct. Dyn.* **00:1-6**

Luco, N., and, Cornell, C. A., (1998). Seismic drift demands for two SMRF structures with brittle connections. *Structural Engineering World Wide*, **Elsevier Science Ltd., Oxford.**

Matlok, H., (1970). Correlations for design of laterally loaded piles in soft clay . *Proceeding of the 2nd Offshore Technology Conference, Houston, Texas*. **Vol. 1: 577-588**.

Mazzoni, S., McKenna, F., Fenves, GL., (2006). OpenSees Command Language Manual.

O'Neill, M. and Murchison, J. (1983). An evaluation of p-y relationships in sands. *Department of Civil Engineering, University of Houston*, **Report GTDF02-83**.

O'Neill, L. A., Fakas, E., Ronalds, B. F., and Christiansen, P. E., (2000). History Trends and Evolution of Floatover Deck Installation in Open Waters. *SPE Annual Technical Conference and Exhibition, Dallas,* **SPE Paper 63037.**

Reese, L.C., and Welch, R.C. (1975). Lateral loading of deep foundations in stiff clay. *Journal of the Geotechnical Engineering Division*. **ASCE. 101(GT7), 633-649.**

Salama, K. S., Suresh, P. K., and Gutierrez, E. C., (1999). Deck Installation by Float over Method in Arabian. *Offshore Technology Conference*, **Gulf, paper OTC 11206**.

Seed, H. B., and Idriss, I. M., (1970). Soil moduli and damping factors for dynamic response analysis. *Earthquake Engineering Res.Ctr., University of California, Berkeley, Calif*, **Report No. UCB/EERC-70/10**.

Shome N, Cornell CA. (1999). Probabilistic seismic demand analysis of nonlinear structures. *RMS Program, Stanford University, Stanford*. **Report No. RMS-35.**

Vamvatsikos, D., Cornell, CA., (2002). Incremental dynamic analysis. *Earthquake Engineering and Structural Dynamics,* **Vol. 31(3), 491-514.**