INTERNATIONAL JOURNAL OF
COASTAL & OFFSHORE ENGINEERING

IJCOE Vol.7/No. 1/Winter 2022 (50-62)

Available online at: www.Ijcoe.org

Probabilistic Seismic Assessment and Fragility Curves for Fixed Pile-Founded Offshore Platforms

Samira Babaei[1*], Rouhollah Amirabadi[2], Mahdi Sharifi[3]

[1] PhD Candidate, Department of Civil Engineering, University of Qom, Qom, Iran, S.Babaei@stu.qom.ac.ir*
2 Assistant Professor, Department of Civil Engineering, University of Qom, Qom, Iran, R.Amirabadi@qom.ac.ir
3 Assistant Professor, Department of Civil Engineering, University of Qom, Qom, Iran, M.Sharifi@qom.ac.ir

ARTICLE INFO

Article History:
Received: 15 Apr. 2021
Accepted: 01 Dec. 2022

Keywords:
Probabilistic Seismic Demand Model
Probabilistic Seismic Demand Analysis
Incremental Dynamic Analysis
Fragility Curve
Fixed Pile-founded Offshore Platform

ABSTRACT

Fixed pile-founded offshore platforms installed in the seismic-prone areas are exposed to the risk of earthquake-induced disastrous failure and costly operation interruption. Accordingly, the development of applied seismic evaluation methodologies for these infrastructures is a matter of utmost importance. In the context of performance-based earthquake engineering (PBEE), probabilistic seismic assessments of fixed pile-founded offshore platforms have been investigated, here. A three-dimensional (3D) finite element model of a recently installed platform located in the South Pars Oil and Gas field of the Persian Gulf has been made. Soil-pile-structure interaction, as well as dynamic site response effects, has been considered. Probabilistic seismic demand modeling (PSDM) has been employed to manifest the efficient and sufficient ground motion intensity measures (IMs) which can rigorously predict the structural engineering demand parameters (EDPs). Derived from probabilistic seismic demand analysis (PSDA), the superb results have been also evaluated by means of the predominantly used method of incremental dynamic analysis (IDA). On the other hand, the drawn findings contributed to representing the fragility curves of the fixed pile-founded offshore platforms. The demonstrated results are highly recommended to be considered in related research.

1. Introduction

With the development of the world's economy, the exploitation of marine resources has been paid more and more attention to. For this to be accomplished the offshore platform is considered as the main facility and its safety and serviceability are pivotal. The earthquake-induced responses and vibrations are categorized as one of the inevitable factors which threaten the platforms crew casualty, environmental disasters, production shutdown, and equipment damage. On the other hand, the earthquake unpredictability has made it much more difficult for the engineers to exactly estimate the seismic behavior of the complex structural systems.

Accordingly, seismic evaluations of offshore platforms have been the subject of research in many conducted studies [1-9]. Moreover, Jafari et al. presented evaluation of dynamic effects in the response of offshore wind turbines using incremental wind-wave analysis [10-13].

For a more precise and reliable result, the fragility analysis should be considered to the seismic evaluation of these structures. The fragility analysis is an operational and comparatively mature method [14-16], which is commonly utilized in conventional structures, such as steel moment-resisting frames [17-19], bridges [20–24], storage tanks [25], pile-supported wharves [26], dams [27], transmission towers [28] and gantry cranes [29] to combine the seismic uncertainties and structural uncertainties into the seismic assessment.

Yasseri and Ossei [30], presented the limit states associated with the fixed pile-founded offshore platforms and fragility curves generating six synthetic earthquakes. In 2017, The seismic vulnerability of a fixed offshore platform through development of fragility curves capturing the effects of ageing and corrosion deterioration has been assessed by Jahanitabar and Bargi [31]. Ajamy et al., worked on an analytical approach to develop seismic fragility curves for an existing fixed offshore platform located in Persian Gulf [32]. The sensitivity of a fixed offshore platform considering the uncertainty of various structural parameters under near-fault ground

motions have been studied by Zarrin et al. [33]. Furthermore, the effects of ground motions sample size on seismic fragility analysis of offshore jacket platforms using Genetic Algorithm have been evaluated by Abyani et al. [34]. However, the latter ones [31-34] have used limit states based on FEMA [35-37] and ASCE [38] considerations which reflect the ordinary building structures requisitions. Besides, they conducted IDA [39] to generate fragility curves.
Surmounting the drawbacks, generate the motivation to investigate and develop the fragility curves for fixed pile-founded offshore platforms based on the limit states which reflect the specific considerations of these kind of structures. As the objective, this paper aims to propose a methodology based on an analytical method using numerical simulations to develop seismic fragility curves for this type of structures. To illustrate the methodology, a recently installed fixed pile-founded offshore platform located in the South Pars Oil and Gas Field has been selected. Probabilistic seismic assessments of the platform have been studied. As a regard, the ideal ground motion intensity measure which can well predict the structural seismic demand measures have been achieved. To obtain the damage measures, nonlinear dynamic analyses of the selected platform under seismic excitations have been performed. Furthermore, the simultaneous effects of model and ground motions uncertainties in both soil and structure parameters as well as the effects of SPSI have been thoroughly considered.

2. Fixed Pile-founded Offshore Platform Seismic Assessments Area of Study

2.1. Probabilistic Seismic Demand Model (PSDM)

Performance-Based Earthquake Engineering (PBEE) describes the quantitative means for achieving predetermined performance levels in specific earthquake intensities by developing a probabilistic framework. One of the basic components of this framework is PSDM [39]. PSDM is based upon a representative relation between IMs and EDPs and provided based on PSDA [40] or IDA [41].
The basic formulation for probabilistic assessment of structural demands, in which the conditional seismic demand is modeled, employing a lognormal distribution, has been addressed by Cornell et al. [38]:

$$P[EDP \geq edp|IM] = 1 - \Phi\left(\frac{\ln(edp) - \ln(\eta_{EDP|IM})}{\beta_{EDP|IM}}\right) \quad (1)$$

In Eq. (2), $\Phi(\cdot)$ is the standard normal cumulative distribution function, *edp* is the peak or residual demand, $\eta_{EDP|IM}$ is the median value of the demand in terms of an IM, and $\beta_{EDP|IM}$ is the logarithmic standard deviation, or dispersion, of the demand conditioned on the IM. The relationship between median structural demand, $\eta_{EDP|IM}$, and IM can be estimated by a power model expressed in Eq.(3):

$$\eta_{EDP|IM} = a\, IM^b \quad (2)$$

Where, constants a and b are regression parameters.
Data for the regression analysis are developed by performing non-linear time history analyses with analytical fixed pile-founded offshore platform models representative of a typical offshore platform class using a suite of N ground motions. Peak demands (edp_i) are then plotted against the ground motion intensity for estimating the regression parameters, as well as the dispersion term ($\beta_{EDP|IM}$). The conditional standard deviation of the regression used to estimate the dispersion, where edp_i is the ith realization of the demands from the non-linear time history analyses, can be shown as:

$$\beta_{EDP|IM} \approx \sqrt{\frac{\sum_1^N(\ln(edp_i) - (\ln(a\, IM^b))^2}{N-(m+1)}} \quad (5)$$

The smaller the $\beta_{EDP|IM}$ is, the more efficient the IM is. Besides, the selected IM should be sufficient. A sufficient IM, quantified by the p-value [43-45], is conditionally statistically independent of the ground motion characteristics, such as magnitude (M_w) and source distance (R). The null hypothesis rejecting probability in an analysis of variance is described by p-value, where the null hypothesis refers to that the slope coefficient of linear regression is zero. Among widely used levels of significance this study adopts the significance level of 5% (p-value = 0.05) for the threshold. For p-values< 0.05, evidence for rejecting the null hypothesis is strengthening, and consequently, the IM which leads to p-values< 0.05, is considered as an insufficient IM. Therefore, an IM is sufficient in which the conditional probability distribution of EDP, given IM, is independent of ground motion parameters such as M_w, R, and T_p [46]. Consequently, the optimal IM-EDP pair, is the one in which the IM can appropriately predict the structural behavior observed quantifiably based on the output of the corresponding non-linear dynamic analysis.
Ground motion records selection is considered as one of the most challenging issues in analytical models of seismic performance assessments. For the purposes of this paper, a suite of 80 ground motion records is taken from the PEER strong motion database to be used in PSDA [47]. The records are selected from the studies of Babaei et al. [48-51] and based on the following criteria: (1) The site is classified as site class D according to NEHRP seismic provisions [52], (2) The earthquake magnitudes are between 6 and 8,

and (3) The source-to-site distances are between 20 km and 80 km (which are not near fault records).
It has been tried to expand the variety of IMs and EDPs to cover the main considerations of fixed pile-founded offshore platforms seismic assessments.

2.2. Fragility Curves Development

According to the performance-based earthquake engineering advancement, the site-specific deterministic design criteria are transitioning towards fragility curves as a means of describing the performance at different levels of ground motion intensity measures [23]. Thus, seismic fragility curves can provide understanding of seismic performance of offshore platforms at different limit states. Fragility is a versatile term employed throughout earthquake engineering to describe the susceptibility of a structure (or part of a structure) to seismic damage. Its estimation can be based on a variety of empirical [53], numerical [54], or expert opinion data [55], hybrid [56] and their combinations, using various methodologies that range from pure statistical processing of existing data to computational static and dynamic procedures that generate new data from scratch [57].
The fragility or conditional probability can be expressed as [58]:

$$P(D > d \,\Big|\, IM = x) = 1 - \Phi(\frac{Lnx - \mu}{\beta}) \tag{3}$$

where *Lnx* is the median estimate of the demand as a function of IM, μ is median value of the structural capacity or the limit state, β is the standard deviation and $\varphi(.)$ is the standard normal cumulative distribution function(CDF). Since the lognormal appropriately matches a variety of structural [59,60], as well as nonstructural component failure data [61,62], it has been employed here having strong precedent in seismic risk analysis.
However, when the model includes lots of parameters -such that their probability density functions are highly nonlinear- achieving a closed form analytic solution is usually impossible. Therefore, in this paper optimal parameters are found using numerical non-linear optimization algorithms to maximize the log-likelihood functions.

3. Data and Method

3.1. Model Platform Characteristics

Recently installed in the 13th Phase, South Pars Oil and Gas Field of the Persian Gulf in (-57.2) meter water depth, the studied model is a X brace four leg battered fixed pile-founded platform with four main piles. The pile foundations penetrate 109 meters below the mudmat. To obtain the accurate results, three dimensional (3D) model of the platform has been taken into consideration and subjected to ground motion excitations in both X and Y directions. The key characteristics of the studied model (SPD 13 jacket) are presented in the Figure 1.

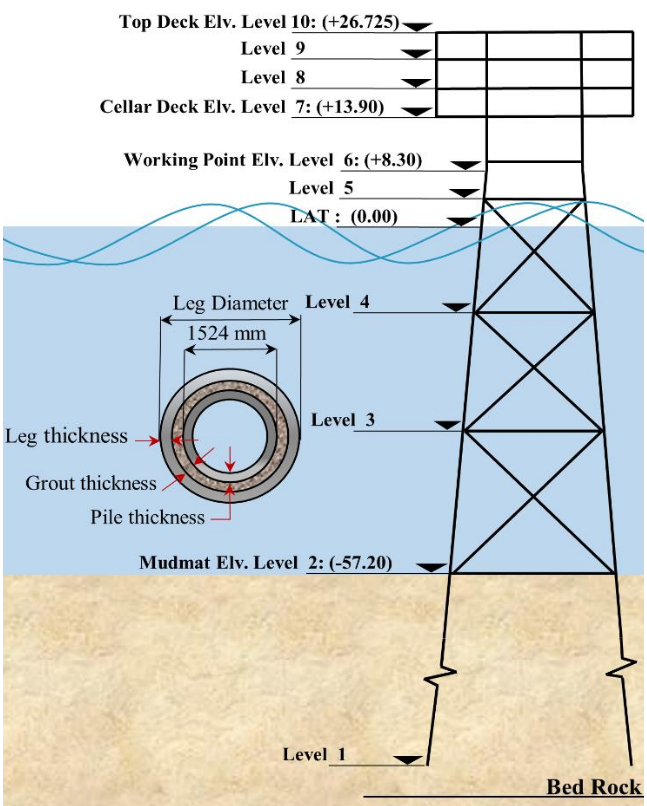


Figure 1. A schematic 2D view of SPD 13 platform illustrating the key characteristics

Aforementioned platforms generally consist of the following main parts: 1. A superstructure providing deck space for supporting operational appurtenances and other loads. 2. Completely braced welded tubular space frame, extends from an elevation at or near the sea bed to above the water surface, which is designed to serve as the main structural element of the platform, transmitting lateral and vertical forces to the foundation (jacket). 3. Foundation elements such as piles, that permanently anchor the platform to the ocean floor, and carry both lateral and vertical loads.
The Morison equation has been also used, estimating the hydrodynamic forces acting on the cylindrical platform members when the platform body moves relative to the flow in the inline direction as may occur confronting the platform to seismic-induced loadings based on equation (2):

$$F = \frac{1}{2}\rho C_D D(\dot{u} - \dot{u}_b)|\dot{u} - \dot{u}_b| + \rho C_M A(\ddot{u} - \ddot{u}_b) + \rho C_M \ddot{u} - M\ddot{u}_g \tag{4}$$

In which the first term reflects the drag force, the second and third ones refer to the hydrodynamic mass and Froude-Krylov forces, respectively [63], while the forth term is attributed to external (seismic) loading. Besides, $\dot{u}$, $\ddot{u}$, $\dot{u}_b$, and $\ddot{u}_b$denote the fluid velocity, acceleration and the body velocity and acceleration, respectively. M and $\ddot{u}_g$ indicate the structure's mass and the input ground motion acceleration. Not considering the effect of simultaneous seismic and wave loading based on API RP2A-WSD [64], the equation of motion for the platform experiencing seismic-induced vibrations in still fluid will be as represented below:

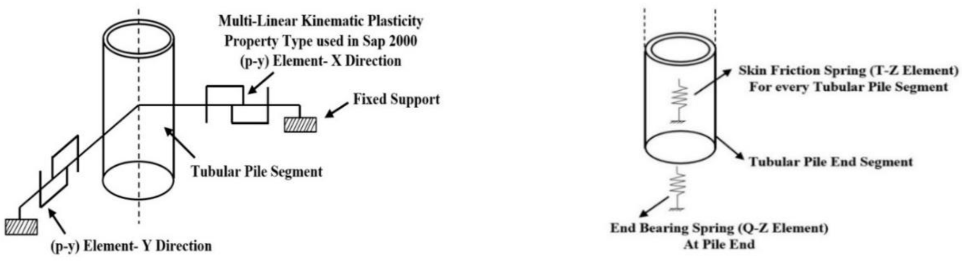
$$F = M\ddot{u}_b + C\,\dot{u}_b + Ku_b \tag{5}$$

In Equation (3), C and K indicate the structural damping and elastic stiffness, and u_b reflect the platform displacement. Combining Equation (2) and Equation (3) will result in the following equation, where $\acute{M}$ is the hydrodynamic mass per unit span that equals to $\rho.C_m.A$.

$$(M+\acute{M})\ddot{u}_b + \frac{1}{2}\rho C_D D(\dot{u}_b)|\dot{u}_b| + C\,\dot{u}_b + Ku_b = -M\ddot{u}_g \tag{6}$$

3.2 Modeling of piles and the pile surrounded soil

According to the field and laboratory investigation, the stratum encountered at the borehole performed at the platform location was very soft calcareous becoming carbonate clay (CH) overlying medium dense becoming loose clayey siliceous carbonate sand (SC) at 10.60m followed by very stiff clay, and hard clay in deeper layers. All the structural elements and joints are checked in compliance with the technical considerations of API-RP-2A WSD [64]. In order to simulate the mass of the platform components in the dynamic analyses, lumped mass method has been employed. Entrapped water of flooded members and added mass for all the members below the sea level besides the mass of marine growth have been also assigned to the corresponding member joints. Moreover, the secondary and nonstructural items masses have been lumped to the nearest platform structural nodes. Since structural behavior of offshore platform in the nonlinear range depends primarily on the soil–pile–structure interaction (SPSI), the Beam on Nonlinear Winkler Foundation (BNWF) model, has been of a major concern and utilized in this study, often referred to as the *p-y* method [65]. The lateral soil stiffness is modelled using the *p–y* approach [66,67]. Sap 2000 [68] multi-linear plastic type link element is employed in the numerical model proposed in this paper reflecting the nonlinear lateral relation between the soil and the pile. In that link element, the nonlinear link stiffness for the axial degree of freedom is defined according to the *p–y* curve. Then the *p–y* curve is redefined as a force–deformation relationship in which *p* is the total force acting along the tributary length of a pile joint. In order to represent the lateral soil nonlinear behavior, a lateral link is defined for each joint along each unit pile segment. Chosen from the SAP2000 library, a multi-linear kinematic plasticity property type used for uniaxial deformation. The selected property models the hysteresis of the non-gapping soil behavior. Besides lateral loads, the pile foundation is exposed to the static and cyclic axial loads. Nonlinear axial load-deformation behavior along the shaft of driven tubular pile can be modelled using *t-z* data, recommended by API RP 2A-WSD [69]. Similarly, *q-z* curves model the elastic and plastic soil deformations around the pile tip reflecting the relationship between mobilized end bearing resistance and axial tip deflection. The schematic configuration of the proposed model in SAP2000 is illustrated in Figure 2.



(a). *p-y* Elements **(b).** *t-z* and *q-z* Elements

Figure 2. Schematic configuration of pile segments and (a) *p-y* elements, (b) *t-z* & *q-z* elements

It is worth noticing that, incorporating soil nonlinearity in any site response analysis, is essential. Each soil layer is characterized by its thickness, mass density, shear wave velocity, and nonlinear soil properties including nonlinear modulus reduction and damping curves which effect on the selected ground motion records. In fact, the results of site response display seismic performance assessment of nonlinear ground response analysis within the soil profile, while they can reflect in the input ground motions determination uncertainties, the site velocity profile characterization and the nonlinear properties specification as well as analysis technique selection [70]. The computer program DEEPSOIL is employed to perform site response simulations based on the soil layer characteristics and selected ground motion records [71] using outcropping motions in the time domain and the layered soil column as a multiple-degree-of-freedom lumped mass system. Figure 3 shows the multi-degree-of-freedom lumped parameter model for layered soil. The displacement time history from nonlinear site response analysis are reflected, as well.

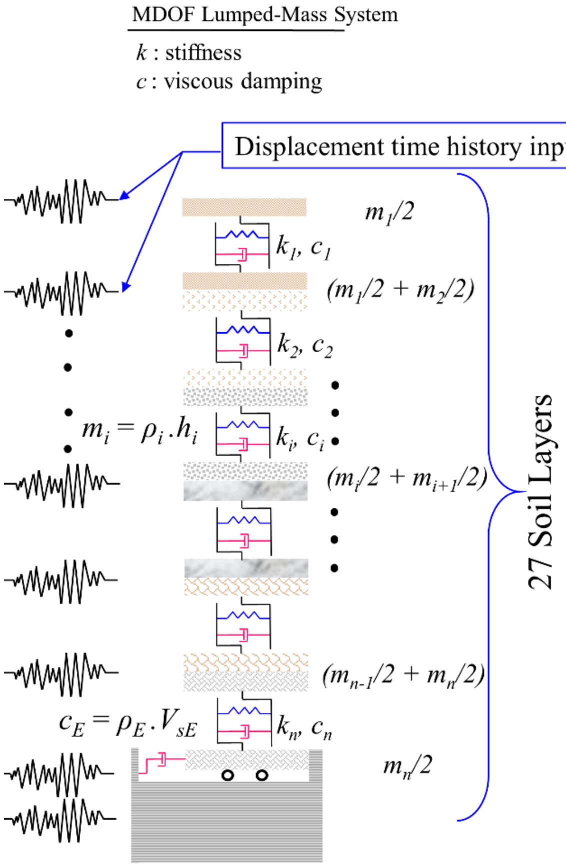


Figure 3: Seismic Site Response Considerations

3.4 Analysis Routine

The dynamic model of fixed pile-founded offshore platforms should reflect the key analytical parameters of mass, damping, and stiffness. The three dimensional (3D) model has been created employing Sap 2000, drag and inertia forces were exerted and Morison Equation has been employed calculating hydrodynamic loads. Each analysis routine includes a modal analysis to determine natural frequency as well as mode shape information, static pushover analysis (SPO) to represent yield values, and nonlinear dynamic time-history analysis (NTH) to determine demand measures. Numerous quantities were monitored in order to extract maximum and residual dynamic quantities, such as axial force, moment, horizontal and vertical displacements and so on. Access to response quantities extracted from the model is provided by post-processing. The periods of the first three vibration modes for the studied platform is listed in Table 1. Evident from the table, Sap 2000 results match well with the characteristics obtained from the initial model of the installed platform analyzed via Structural Analysis Computer System (SACS) software.

Table 1. Verification of the First Three Periods

	Period	Sap 2000(s)	SACS(s)
SPD 13, 3D Model, Modal Analysis	1st Mode	2.26	2.38
	2nd Mode	1.96	2.03
	3rd Mode	1.51	1.43

4. Result and Discussion

4.1 Optimal PSDM Determination

The structural behavior, natural period and mode shapes of the platforms have shown that these structures are flexible structures which are much more similar to high-rise buildings [48-51]. Due to this harmony in behavior, the considerations associated with these specific type of structures should be taken into account. While *PGA* exerts the greatest influence on the seismic response of the structures with higher frequencies (periods of less than 0.5 s), structures with lower frequencies (periods of more than o.5 s) are more sensitive to *PGV*. Tall long-period buildings studies indicated that due to their response frequency range which is much wider than low-rise or mid-rise buildings, IMs such as spectral values ($S_a(T_1)$) represent only specific points in frequency content of the response spectrum [72-74]. For that reason, the intensity measures comprising a wider range of frequency content of response spectra (e.g. Housner Intensity (*HI*) [75]) are more appropriate for the case of structures with periods more than 0.5 second (such as high-rise buildings and fixed pile-founded offshore platforms).

Accordingly, for the purposes of this study PSDM assessment for a large variety of IMs and EDPs has been conducted employing PSDA. Detailed evaluations have been presented in the study of Babaei et al. [48-51]. Their drawn findings indicated that PSDMs conditioned by *PGV* and the platform ductility (μ_{Global}) as well as the one provided by *HI* and the platform drift ration (θ_{Global}) are optimal based on the results presented in Table 2.

Table 2. PSDM assessments[48-51]

PSDM	Efficiency (dispersion)	Sufficiency(*p*-value)	
		w.r.t. M_w	w.r.t. *R*
PGV-μ_{Global}	0.28	0.249	0.164
HI-θ_{Global}	0.21	0.076	0.461

For the purposes of this study and based on the introduced damage measures, the appropriate PSDM will be used, the IM will be scaled and IDA will be performed. Accordingly, a suite of 15 ground motion records which lead to equivalent results have been chosen, as listed in Table 3.

Table 3. Selected Records

#	Ground Motion Records	M_w	R(km)
1	Kobe, Japan, Kakogawa-1995	6.9	22.5
2	Jiroft, Bam, Iran,2003	6.6	69.28
3	Landers, Barstow, 1992	7.28	34.86
4	Trinidad, Rio Dell Overpass-FF, 1995	7.2	76.06
5	Imperial Valley-06, Coachella Canal #4, 1979	6.53	49.1
6	Big Bear-01, Featherly Park – Maint, 1992	6.46	78.81
7	Hector Mine, Amboy, 1999	7.13	41.81
8	Landers, Fort Irwin, 1992	7.28	62.98
9	Morgan Hill, Capitola, 1984	6.19	39.08
10	Borrego, El Centro, 1942	6.5	55.88
11	Duzce, Turkey, 1999	7.14	34.3
12	Manjil, Iran, Abhar, 1990	7.37	75.58
13	Superstition Hills, Imperial Valley Wildlife, 1987	6.54	23.85
14	Hector Mine, NorthPalm, 1999	7.13	61.86
15	Denali, Alaska, R109, 2002	7.9	42.99

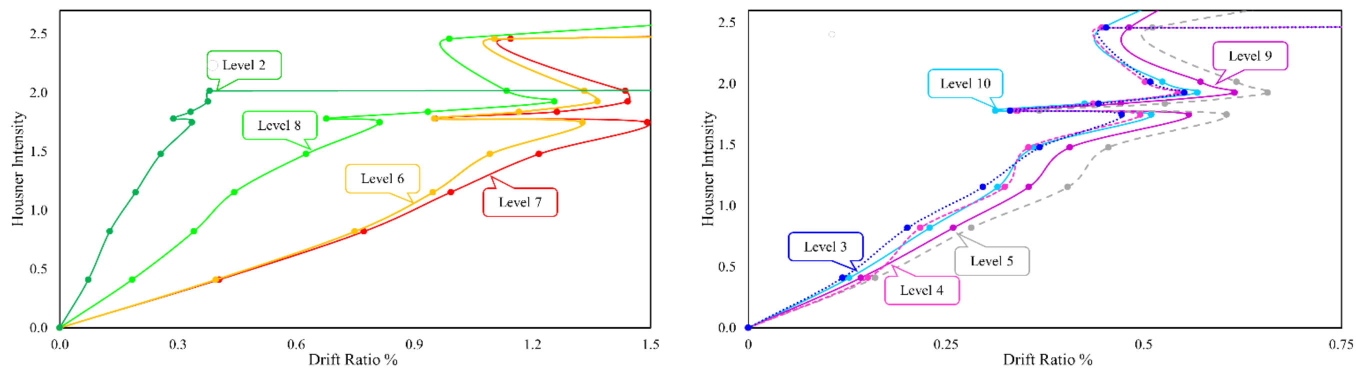
4.2 Performing Incremental Dynamic Analyses

Recently emerged to offer a comprehensive evaluation of the structures seismic performance, Incremental Dynamic Analysis (IDA) [39] is justifiably claimed as a powerful computer-intensive method that has been used within the framework of PBEE. Using numerous nonlinear dynamic analyses under a suite of multiply-scaled ground motion records, IDA allows the detailed assessment of the seismic performance of structures for a wide range of limit states, ranging from elasticity to dynamic instability and eventual collapse.

The hunt & fill algorithm has been chosen to perform the actual dynamic analysis required for IDA in a fast and automated way. This ensures that the record scaling levels are appropriately selected and minimize the computational efforts, accordingly. Analyses are performed at rapidly increasing levels of IMs until numerical non-convergence is encountered, while additional analyses are running at intermediate IM levels to sufficiently bracket the global collapse and increase the accuracy at lower IMs [31].

Seismic assessments of SPD13-A have been conducted utilizing IDA. The results of probabilistic assessment which are represented through the following illustrations vary dramatically at different levels (indicated in Figure 5) of the studied platform.

In order to provide a better understanding of the illustrated result, the IDA curves for different levels based on their drastic differences have been split into two parts (a and b) in Figure 5. As can be seen, while the platform behavior for levels 3, 4, 5, 9 and 10 follows a similar trend subjecting to various scales of ground motion IMs, this is not complied in other levels. The inter-story drift ratios in these levels meet striking rises in higher IM values. Furthermore, the hardening and softening cases can be also seen.



a. Inter-story Drift Ratio for Levels 2, 6,7 and 8

b. Inter-story Drift Ratio for Levels 3, 4,5,9 and 10

Figure 5: IDA Curves for Different Platform Levels Subjected to Record No. 8

On the other hand, the structural responses of the incremental dynamic analysis for different IM scales through the platform various levels are reflected in Figure 6, a and b. Severe increase in the inter-story drift ratios of levels 2 and 7 experienced in higher IM values, has been drawn.

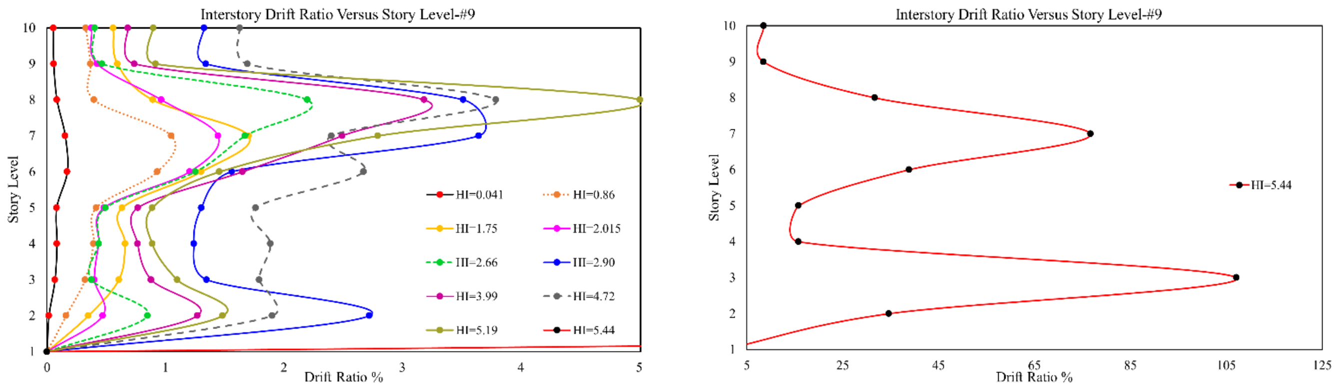
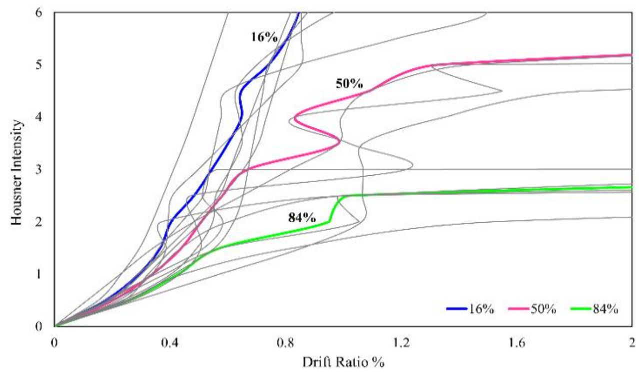
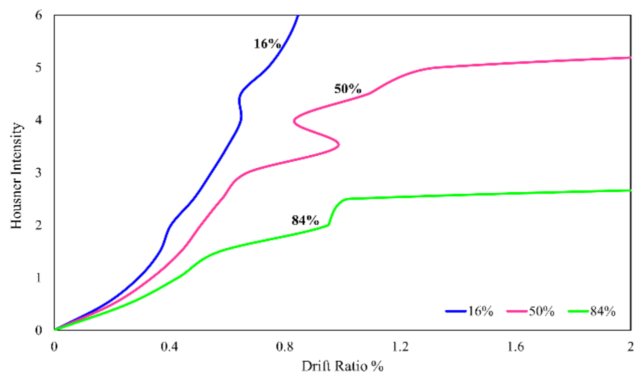


Figure 6: Inter-story Drift Ratios for Different Platform Levels according to Various IM Scales (Subjected to Record No.9)

Besides the single-record IDA curves, the multi-record IDA curves have been also prepared and represented in Figures 7 and 8, reflecting the platform global drift ratio as well as ductility, respectively. Consequently, the summary of the platform responses IDA results of 16%, 50% and 84% have been calculated which can be utilized in fragility investigations of fixed pile-founded offshore platforms.

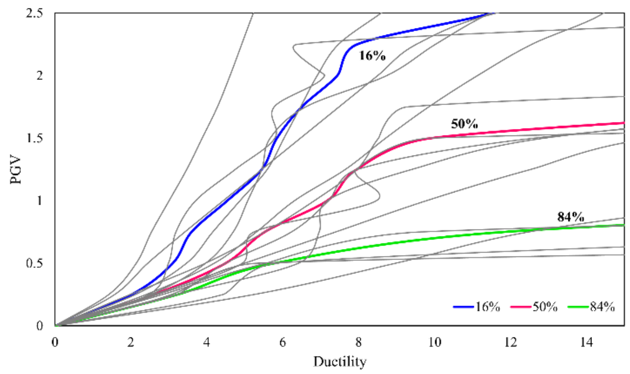


a. Multi-Record IDA Curves (Raw Curves)

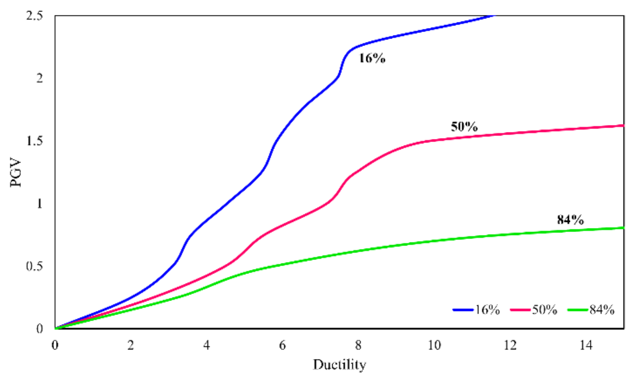


b. Summarized IDA Curves (16%, 50% & 84%)

Figure 7. Multi-record IDA Curves for $HI\text{-}\theta_{Global}$



a. Multi-Record IDA Curves (Raw Curves)



b. Summarized IDA Curves (16%, 50% & 84%)

Figure 8. Multi-Record IDA Curves for $PGV\text{-}\mu_{Global}$

4.3 Fixed Pile-Founded Offshore Platforms Fragility Analysis

As a graphical illustration of structural seismic vulnerability, fragility curves can be generated by the lognormal cumulative distribution function (CDF), which represents the probability of exceedance as a function of ground motion IMs. Since in the conventional design of fixed pile-founded offshore platforms, a linear elastic behavior is assumed, no yield or buckling is allowable. Furthermore, the platform collapse can be expressed due to the first member failure occurrence. It is worth noticing that, the fragility analysis of fixed pile-founded offshore platform is in its initial stage and presenting appropriate limit states (LSs) is a challenging subject area. Recently, different structural LSs have been suggested and employed in some limited studies referring to different guidelines to address structural behaviors. These LSs have been thoroughly studied and a summary of them has been presented as listed in Tables 4 and 5.

Table 4. Previously Employed LSs

IM-EDP	LS	Code	Reference
$S_a(T_1,5\%)$-θ_{max}	**IO**: θ_{max}=1%, **CP**: θ_{max}={6% or 20% of the elastic slope}	FEMA 350	Jahanitabar and Bargi[31]
	Collapse **L**imit **S**tate: θ_{max}=2%	ASCE 41-06, FEMA 350	Ajamy et al. [32]
	CP: θ_{max}={10% or 20% of the elastic slope}	FEMA 351	Zarrin et al. [33]
	Collapse **L**imit **S**tate: IMs related to points the IDA curves flatten		Abyani et al. [34]

Table 5. LSs Represented by Yasseri and Ossei [30]

IM-EDP	LS	Damage Description
PGA-μ	μ=1	**Negligible**: Minor repairs, no disrupting
	μ=2	**Slight**: Repairable damage, some replacement required
	μ=4	**Moderate**: serious disruption
	μ=6	**Extensive**: beyond repair
	μ=7.5	**Near collapse**

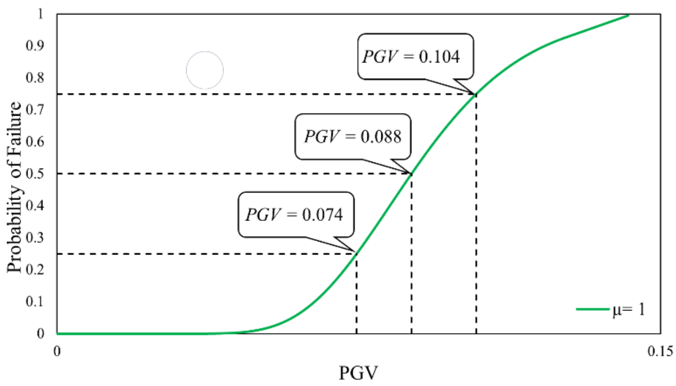
As drawn from Tables 4 and 5, the only appropriate LSs for fixed pile-founded offshore platforms has been assumed to be those reflected in the study of Yasseri and Ossei [30], while other studies referred their chosen LSs to the ordinary building ones derived from the related codes. Consequently, Table 6 represent the values of the summarized 16%, 50% and 84% of demand measures (DMs) for the ductility-based LSs based on Yasseri and Ossei [27].

Table 6. Summary of the Results

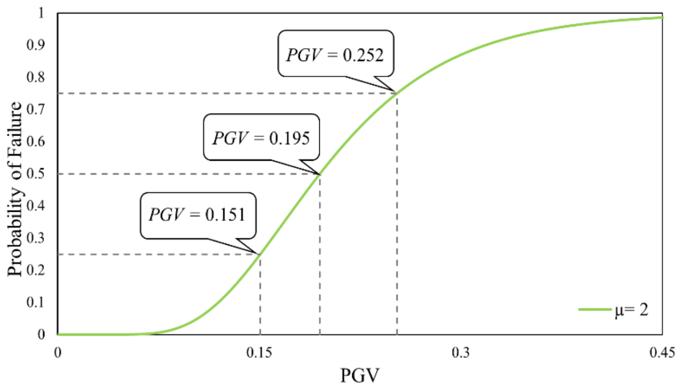
	IM: PGV(m/s)			**EDP:** μ
	$IM_{16\%}$	$IM_{50\%}$	$IM_{84\%}$	
Negligable	0.124	0.096	0.078	1
Slight	0.248	0.193	0.155	2
Moderate	0.857	0.435	0.326	4
Extensive	1.558	0.823	0.509	6
Near Collapse	2.030	1.448	0.572	7.5

In fragility analysis of fixed pile-founded platforms investigated here, the platform responses, obtained from the IDA has been utilized, and the lognormal fragility function in each LSs has been defined via two parameters, the IM median value (at which the platform reaches the LS threshold) as well as the logarithmic standard deviation or dispersion. These parameters represent the IM corresponding to 50% probability of exceeding a certain LS and the dispersion in the results due to record-to-record variability, respectively. The results for five various LSs have been obtained and illustrated in Figure 9. For the purposes of this study the widely used method of Porter has been employed [74]. For each LSs, the IM values which have led to 25%, 50% and 75% probability of exceeding are obtained and shown in the illustrations.

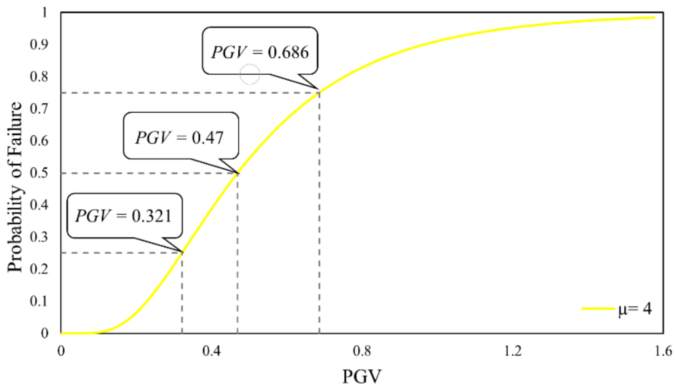
Moreover, as shown in Table 7, the medians and the logarithmic standard deviations have been also computed and represented.



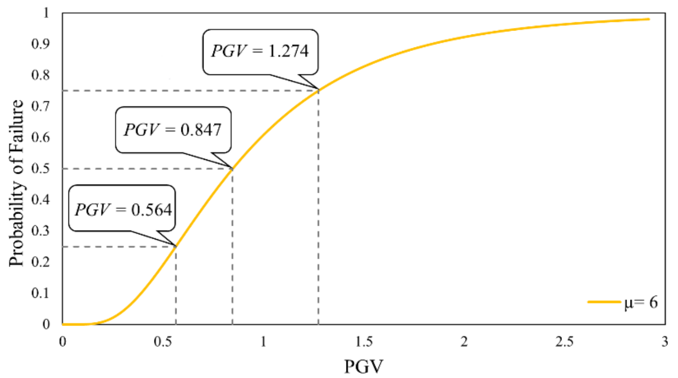
a. Fragility Curve for Ductility=1



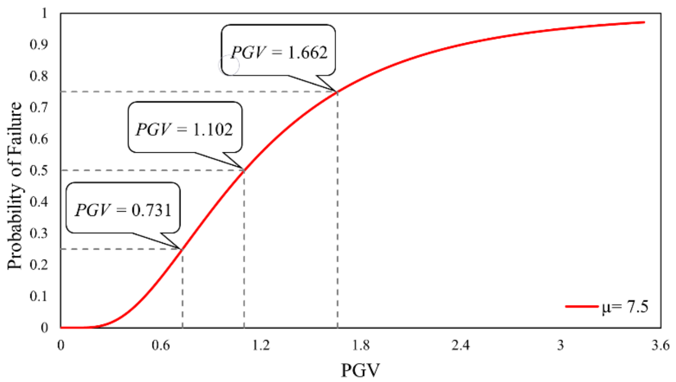
b. Fragility Curve for Ductility=2



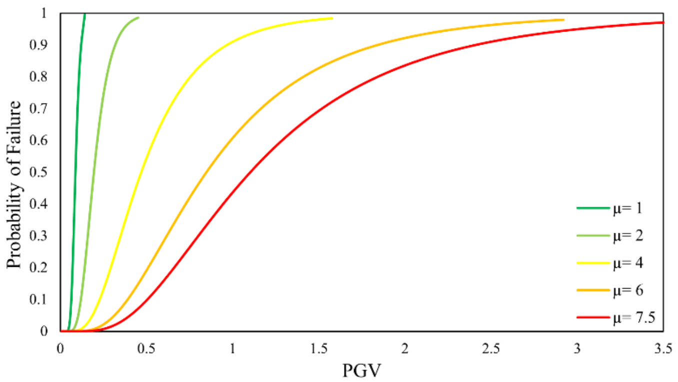
c. Fragility Curve for Ductility=4



d. Fragility Curve for Ductility=6



e. Fragility Curve for Ductility=7.5



f. Fragility Curve for Ductility

Figure 9. Fragility Curves for Fixed Pile-Founded Offshore Platforms

Table 7. Summary of the Results, median and standard deviation

performance Levels	Negligible	Slight	Moderate	Extensive	Near Collapse
median	0.09	0.2	0.47	0.85	1.10
standard deviation	0.258	0.396	0.583	0.626	0.631

5. Conclusion

As a general objective, this study aims to represent the results of probabilistic seismic assessment of fixed pile-founded offshore platforms. A 3D model of the platform, considering the effects of soil-pile-structure interactions and seismic site responses has been made. Accordingly, the drawn findings of PSDM evaluations obtained from PSDA for a recently installed typical fixed pile-founded offshore platforms of the Persian Gulf, has been exhibited. For the introduced PSDMs, incremental dynamic analysis (IDA) has been also performed and the significantly various results for different IM scales as well as the platform levels have been established. Since the fragility analysis of fixed pile-founded offshore platforms is presumed to be in the initial development levels, the limited studies have been explored. Among these investigations, the limit states represented by Yasseri and Ossei [30] have been considered as appropriate and employed to produce seismic fragility curves for the studied platform. The results of this paper is proposed to be utilized for the seismic assessments and fragility analysis of the similar platforms. Proposed for future studies, it is suggested to extract fragility curves for other kind of offshore platforms such as offshore wind turbines.

6. References

[1] Bea, R.G., Puskar, F.J., Smith, C., Spencer, J.S., (1988), Development of AIM (assessment, inspection, maintenance) programs for fixed and mobile platforms. In: Proceedings of the offshore technology conference. Paper OTC 5703.

[2] Yasseri, S. and Ossei, R., (2004), Seismic Fragility Analysis of Pile-founded Offshore Platforms, Proceeding of the Fourteenth International Offshore and Polar Engineering Conference, Toulon, France, Paper Number: ISOPE-I-04-028

[3] Asgarian, B., Aghakouchak, A. A., Alanjari, P. and Assareh, M. A., (2008), Incremental Dynamic Analysis of Jacket Type Offshore Platforms Considering Soil-Pile Interaction, 14th World Conference on Earthquake Engineering, Beijing China.

[4] Park, M., Koo, W., Kawano, K., (2011), Dynamic response analysis of an offshore platform due to seismic motions, Eng. Struct., Vol. 33 (5), p. 1607–1616.

[5] El-Din, M. N. and Kim, J., (2014), Seismic performance evaluation and retrofit of fixed jacket offshore platform structures, J. Performance of Constructed Facilities.

[6] Sharifian, H., Bargi, K., Zarrin, M., (2015), Ultimate strength of fixed offshore platforms subjected to near-fault earthquake ground vibration. Shock Vib., P. 1–19.

[7] Elsayed, T., El-Shaib, M., Gbr, k., (2014), Reliability of fixed offshore jacket platform against earthquake collapse, J. Ships Offshore Struct., Vol.11 (2), p. 167–181.

[8] Babaei, S., Amirabadi, R., Taghikhany, T., (2016), Assessment of Semi-Active Tunes Mass Damper Application in Suppressing Seismic-Induced Vibration of an Existing Jacket Platform, International Journal of Maritime Technology, Vol. 6, p. 1-10.

[9] Konstandakopoulou, F.D., Evangelinosb, K.I., Nikolaouc, I.E., Papagiannopoulosd, G.A., Pnevmatikose, N.G., (2019), Seismic analysis of offshore platforms subjected to pulse-type ground motions compatible with European Standards, Soil Dynamics and Earthquake Engineering, https://doi.org/10.1016/j.soildyn.2019.105713

[10] Jafari, A., & Dezvareh, R. (2021). Evaluation of dynamic effects in the response of offshore wind turbines using incremental wind-wave analysis. Research in Marine Sciences, 6(1), 860-868.

[11] Jafari, A., & Dezvareh, R. (2020). Performance based assessment of offshore wind turbine platform using the constrained new wave method. Journal of Oceanography, 11(43), 71-80.

[12] Jafari, A., & Dezvareh, R. (2021). Determination of collapse prevention (CP) of offshore wind turbine with jacket platform. Iranian Journal of Marine Science and Technology, 24(96), 35-43.

[13] Dezvareh, R. (2019). Providing a new approach for estimation of wave set-up in Iran coasts. Research in marine sciences, 4(1), 438-448.

[14] Lupoi, G.; Franchin, P.; Lupoi, A.; Pinto, P.E., (2006), Seismic fragility analysis of structural systems, J. Eng. Mech., Vol. 132, p. 385–395.

[15] Rosowsky, D.V.; Ellingwood, B.R., (2002), Performance-based engineering of wood frame housing: Fragility analysis methodology. J. Struct. Eng., Vol. 128, p. 32–38.

[16] Jia, H.; Zhao, J.; Li, X., (2018), Probabilistic pounding analysis of high-pier continuous rigid frame bridge with actual site conditions, Earthquakes Struct., Vol. 15, p. 193–202.

[17] Asgarian, B.; Sadrinezhad, A.; Alanjari, P. Seismic performance evaluation of steel moment resisting frames through incremental dynamic analysis. J. Constr. Steel Res. 2010, 66, 178–190.

[18] Fattahi, F.; Gholizadeh, S. Seismic fragility assessment of optimally designed steel moment frames. Eng. Struct. 2019, 179, 37–51.

[19] Bakalis K. and Vamvatsikos D., (2018), Seismic Fragility Functions via Nonlinear Response History Analysis, Journal of Structural Engineering, ASCE, 144(10): 04018181 DOI: 10.1061/(ASCE)ST.1943-541X.0002141

[20] Mander, J.B.; Dhakal, R.P.; Mashiko, N.; Solberg, K.M., (2007), Incremental dynamic analysis applied to seismic financial risk

assessment of bridges. Eng. Struct., Vol. 29, p. 2662–2672.

[21] Jia, H.; Lan, X.; Luo, N.; Yang, J.; Zheng, S.; Zhang, C., (2019), Nonlinear Pounding Analysis of Multispan and Simply Supported Beam Bridges Subjected to Strong Ground Motions. Shock Vib.

[22] Jia, H.; Lan, X.; Zheng, S., (2019), Assessment on required separation length between adjacent bridge segments to avoid pounding. Soil Dyn. Earthq. Eng., Vol. 120, p. 398–407.

[23] Mackie, K., and Stojadinovic, B., (2004), Fragility curves for reinforced concrete highway overpass bridges, 13th World Conference on Earthquake Engineering Vancouver, B.C., Canada

[24] Muntasir Billah, A.H.M., & Shahria Alam, M., (2014), Seismic fragility assessment of highway bridges: a state-of-the-art review, Structure and Infrastructure Engineering: Maintenance, management, Life-Cycle Design and Performance, DOI: 10.1080/15732479.2014.912243

[25] Berahman, F., Behnamfar, F., (2007), Seismic fragility curves for un-anchored on-grade steel storage tanks: bayesian approach, J. Earthquake Eng. Vol.11(2), p. 166–92.

[26] Heidary-Torkamani, H., Bargi, Kh., Amirabadi, R., and McCllough, N.J., (2014) Fragility estimation and sensitivity analysis of an idealized pile-supported wharf with batter piles, Soil Dynamics and Earthquake Engineering, Elsevier, Vol.61-62 p. 92–106, http://dx.doi.org/10.1016/j.soildyn.2014.01.024

[27] Hariri-Ardebili, M.A., and Saouma, V.E., (2016), Probabilistic seismic demand model and optimal intensity measure for concrete dams, Structural Safety, Elsevier, Vol. 59 p. 67–85, http://dx.doi.org/10.1016/j.strusafe.2015.12.001

[28] Tian, L.; Pan, H.; Ma, R., (2019), Probabilistic seismic demand model and fragility analysis of transmission tower subjected to near-field ground motions, J. Constr. Steel Res., 156, 266–275.

[29] Peng, O., Cheng, W., Jia, H., and Guo, P., (2020), Fragility Analysis of Gantry Crane Subjected to Near-field Ground Motions, Appl. Sci., Vol. 10, 4219; doi:10.3390/app10124219

[30] Yasseri, S., Ossei, R., (2004), Seismic Fragility Analysis for Pile-founded Offshore Platforms, Proceeding of the 14th International Offshore and Polar Engineering Conference, Toulon, France, ISOPE-I-04-028.

[31] Jahanitabar AA, Bargi Kh., (2017) Time-dependent seismic fragility curves for aging jackettype offshore platforms subjected to earthquake ground motions, J Struct Infrastruct Eng Mainten, Manage, Life-cycle Des Perform, Vol. 14(2):192–202.https://doi.org/10.1080/15732479.2017.1343360

[32] Ajamy, A., Asgarian, B., Ventura, C.E., Zolfaghari, M.R., (2018), Seismic fragility analysis of jacket type offshore platforms considering soil-pile-structure interaction, Journal of Engineering Structures 174 (1), 198–211.

[33] Zarrin, M., Asgarian, B., & Abyani, M., (2019), Probabilistic Seismic Collapse Analysis of Jacket Offshore Platforms, Journal of Offshore Mechanics and Arctic Engineering, Vol. 140, DOI: 10.1115/1.4038581

[34] Abyani, M., Bahaari, M.R., Zarrin, M., and Nasseri, M., (2019), Effects of sample size of ground motions on seismic fragility analysis of offshore jacket platforms using Genetic Algorithm, Ocean Engineering Vol. 189, 106326, https://doi.org/10.1016/j.oceaneng.2019.106326

[35] ASCE, FEMA 356, (2000) Prestandard and commentary for the seismic rehabilitation of buildings, Publication No. 356, Washington (DC): Federal Emergency Management Agency.

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

[37] FEMA 351, (2000b), Recommended seismic evaluation and upgrade criteria for existing welded steel moment-frame buildings. SAC Joint Venture, Federal Emergency Management Agency, Washington DC.

[38] American Society of Civil Engineers (ASCE), (2007), Seismic rehabilitation of existing buildings. ASCE/SEI 41-06, American Society of Civil Engineers/Structural Engineering Institute, Reston, VA.

[39] Shome, N., (1999), Probabilistic Seismic Demand Analysis of Nonlinear Structures. PhD. Thesis, Dep. Civil and Envir. Eng. Stanford University, Stanford, CA.

[40] Shome, N., Cornell, C.A., Bazzurro, P., & Caraballo, J.E., (1998), Earthquakes, records, and nonlinear responses. Earthquake Spectra, 14(3), 467–500.

[41] Vamvatsikos D., (2002), Seismic performance, capacity and reliability of structures as seen through incremental dynamic analysis. PhD Dissertation, Department of Civil and Environmental Engineering, Stanford University.

[42] Cornell, CA., Jalayer, F., Hamburger, RO., Foutch, DA., (2002), Probabilistic basis for 2000 SAC/FEMA steel moment frame guidelines, J Struct Eng Vol. 128(4) p. 526–533. https://doi:10. 1061/(ASCE)0733-9445

[43] Fisher, RA., (1925), Statistical Methods for Research Workers, Edinburgh, UK: Oliver and Boyd
[44] Tang, WH., Ang, A., (2007), Probability concepts in engineering: Emphasis on applications to civil and environmental engineering, 2nd edn. Wiley, Hoboken, ISBN: 978-0-471-72064-5
[45] Altman, N., Krzywinski, M., (2016), Points of significance: p values and the search for significance, Nat Methods 14:3–4. https://doi:10.1038/nmeth.4120
[46] Tothong P, Luco N (2007) Probabilistic seismic demand analysis using advanced ground motion intensity measures. Earthquake Eng Struct Dyn. https://doi.org/10.1002/eqe.696
[47] Pacific earthquake engineering research center (2006) PEER NGA Database. Berkeley: University of California. http://peer.berkeley.edu/nga/
[48] Babaei, S., Amirabadi, R., Sharifi, M., (2021), Evaluation of Optimal IM-EDP pairs for Typical South Pars Fixed Pile-Founded Offshore Platforms, International Journal of Maritime Technology, Vol. 15:29-49, http://ijmt.ir/article-1-740-en.html
[49] Babaei, S., Amirabadi, R., Sharifi, M., Ventura, C., (2021), Optimal probabilistic seismic demand model for fixed pile-founded offshore platforms considering soil-pile-structure interaction, Structures, Vol.33: 4330-4343, https://doi.org/10.1016/j.istruc.2021.07.040
[50] Babaei, S., Amirabadi, R., Taghikhany, T., Sharifi, M., (2021), Optimal ground motion intensity measure selection for probabilistic seismic demand modeling of fixed pile-founded offshore platforms, Journal of Ocean Engineering, Vol. 242, https://doi.org/10.1016/j.oceaneng.2021.110116
[51] Babaei, S., Amirabadi, R., Sharifi, M., (2021), Sufficiency assessments of ground motion intensity measures employing kullback-leibler theory (applied for typical south pars offshore platforms), Numerical Methods in Civil Engineering Journal, Vol. 5(4). http://nmce.kntu.ac.ir/article-1-340-en.htm
[52] NEHRP (2001) NEHRP recommended provisions for seismic regulations for new buildings and other structures. Washington, DC, USA: Building Seismic Safety Council
[53] Rossetto, T., Ioannou, I., Grant, D., and Maqsood, T., (2014), Guidelines for empirical vulnerability assessment, GEM Technical Rep. 2014-08 V1.0.0. Pavia, Itally: GEM Foundation.
[54] D'Ayala, D., A. Meslem, D. Vamvatsikos, K. Porter, and T. Rossetto. (2015), Guidelines for analytical vulnerability assessment of low/mid-rise buildings, Pavia, Italy: Vulnerability Global Component Project.
[55] Jaiswal, K., Wald, D., and D'Ayala, D., (2011) Developing empirical collapse fragility functions for global building types, Earthquake Spectra 27 (3): 775–795. https://doi.org/10.1193/1.3606398.
[56] Kappos, A.J., Stylianidis, K.C., & Pitilakis, K. (1998). Development of seismic risk scenarios based on a hybrid method of vulnerability assessment. Natural Hazards, Vol.17, p.177–192.
[57] Shinozuka, M., Feng, M.Q., Lee, J., and Naganuma, T., (2000), Statistical analysis of fragility curves, J. Eng. Mech. Vol. 126 (12), p. 1224–1231. https://doi.org/10.1061/(ASCE)0733-9399(2000)126:12(1224).
[58] Baker, J.W., (2015), Efficient analytical fragility function fitting using dynamic structural analysis, Earthquake Spectra, Vol.31(1), p. 579–99.
[59] Aslani, H., 2005. Probabilistic Earthquake Loss Estimation and Loss Disaggregation In Buildings, Doctoral Thesis, Stanford University, Stanford CA, 355 pp.
[60] Pagni, C.A. and L.N. Lowes, 2006. Fragility functions for older reinforced concrete beam-column joints. Earthquake Spectra, 22 (1), Feb 2006
[61] Badillo-Almaraz, H., A.S. Whittaker, A.M. Reinhorn, and G.P. Cimellaro, (2006) Seismic Fragility of Suspended Ceiling Systems, Technical Report MCEER-06-0001, Multidisciplinary Center for Earthquake Engineering Research, Buffalo, NY, 225 pp.
[62] Porter, K.A., and A.S. Kiremidjian, (2001), Assembly-Based Vulnerability and its Uses in Seismic Performance Evaluation and Risk-Management Decision-Making, Report No. 139, John A. Blume Earthquake Engineering Center, Stanford, CA, 214 pp., http://keithp.caltech.edu/publications.htm
[63] Mutlu, B., Fredsoe, J., (1997), Hydrodynamics Around Cylindrical Structures, Advanced Series on Ocean Engineering, vol. 26. Technical university of Denmark, Denmark
[64] API, (2000), Recommended Practice for Planning, Design and Constructing Fixed Offshore Platforms—Working Stress Design. American Petroleum Institute, Washington, DC.
[65] Matlock, H., (1970), Correlations for Design of Laterally Loaded Piles in Soft Clay, Second Annual Offshore Technology Conference, Houston, Vol.1204, p. 557 - 594.
[66] Reese, L. C., & Cox, W. R., (1975), Field Testing and Analysis of Laterally Loaded Piles in Stiff Clay, Offshore Technology Conference, OTC 2312.

[67] O'Neill, M. W., & Murchinson, J. M., (1983), An Evaluation of p-y Relationships in Sands, A Report to the American Petroleum Institute.
[68] Sap 2000, (2005), Structural Analysis Program, Analysis Reference Manual, Computers and structures, Inc., Berkeley, California, USA.
[69] American Petroleum Institute, (2008), Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms - Working Stress Design, API recommended practice (RP-2A-WSD), 21st Edition, Errata and Supplement.
[70] Rathje EM, Kottke RA, Trent WL., (2010), Influence of input motion and site property variabilities on seismic site response analysis, J Geotech. Geoenviron. Eng. ASCE, Vol. 136(4).
[71] Hashash, Y., Groholski, D., Phillips, C., Park, D., & Musgrove M., (2012), DEEPSOIL 5.1. User Manual and Tutorial.
[72] Shome, N., Cornell, CA., (1999), Probabilistic seismic demand analysis of nonlinear structures, RMS Program, Stanford University, Report No. RMS35. https://blume.stanford.edu/rms-reports. Accessed 2 June 2014
[73] Luco, N., Cornell, CA., (2007), Structure-specific scalar intensity measures for near-source and ordinary earthquake ground motions, Earthq Spectra Vol. 23 p.357–392. http://doi:10.1193/1.2723158
[74] Mackie, K., Stojadinović, B., (2003), Seismic demands for performance-based design of bridges, PEER 312
[75] Housner, GW., (1959), Behavior of structures during earthquakes, J Eng Mech Div Vol. 85 p.109–130
[76] Porter, K., Kennedy, R., and Bachman, R., (2007), Creating Fragility Functions for Performance-Based Earthquake Engineering, Earthquake Spectra, Vol. 23(2), p. 471-489.

List of Abbreviation

Abbreviation	Definition
PSDM	Probabilistic Seismic Demand Modeling
PSDA	Probabilistic Seismic Demand Analysis
IM	Intensity Measure
EDP	Engineering Demand Parameter
IDA	Incremental Dynamic Analysis
SPSI	Soil-Pile-Structure Interaction
PBEE	Performance-Based Earthquake Engineering