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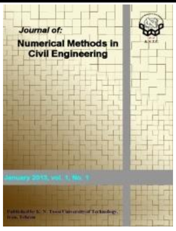

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Effect of earthquake directional uncertainty on the seismic response of jacket-type offshore platform

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Abstract:

In the seismic risk assessment of structures, two main random variables are involved, namely the vulnerability of the structure and the seismic action. The aim of the study presented here is to analyze the seismic behavior of the jacket-type offshore platforms as an expensive and vital structure. Furthermore, the influence of the incidence angle of the seismic action is also investigated by using twelve ground motions, rotating the direction of both orthogonal components by 22.5° (from 0° to 180°). Three damage states have been used in result interpretation. The variability of structural response to the direction of seismic input becomes larger as the level of inelastic behavior increases. In the present study, the critical directions were determined and the vulnerability of jacket type offshore structure was examined by fragility curve based on methodology suggested by Pacific Earthquake Engineering Research Center (PEER) for three damage limit states by both multicomponent incremental dynamic analysis (MIDA) and directional multicomponent incremental dynamic analysis (DMIDA). Finally, it is found that structural behavior in the collapse zone is sensitive to directional uncertainty can change the results by up to 10%.

1. Introduction

In addition to epistemic uncertainties, there are some seismic uncertainties with a considerable effect on structural responses. Although IDA deals with existing uncertainty in a set of ground motions well, there is another type of seismic uncertainty that noticeably affects seismic demand. This uncertainty stems from existing uncertainty in the unknown orientation of earthquakes called directional uncertainty. The incident angle of seismic excitation imposed on a structure located in seismic regions is one of the uncertainty sources in structural analyses and designs. These uncertainties in site conditions, earthquake epicenter, and wave propagation properties make this hypothesis reasonable that the earthquake may impose different actions on the structure in different incident angles.

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Investigation of the incident angle of orthogonal components of the earthquake on a structure and considering the maximum values of demands which correspond to the critical direction of demands is an important problem because the structure may experience more demands in a direction that necessarily doesn't lay down on the axis of analysis. This is while in reality, the direction of the dominant component of excitations might not be one of the main directions of the building axes, and applying the main component in a direction other than the main axes' direction may lead to higher internal forces and stresses in the building's structural elements. There are different investigations that evaluated directional uncertainties for structures on land, especially. The nonlinear dynamic analysis introduces some uncertainties concerning the orientation along which the horizontal components of ground motion were recorded (the orientation of ground-motion reference axes), the orientation of seismic incident angle, and the record scaling.

Stronger vulnerability of structures under bi-directional shaking relative to that under uni-directional excitation has

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been demonstrated in numerous studies [e.g., 1-8]. In 1974, Penzien and Watabe's researches showed that the most intense of earthquake components occurs in the epicenter direction [9]. Due to the uncertainty of epicenter direction toward the location of a structure, research efforts focused on the direction of the maximum response experienced in a building [10-11]. Also, Davila and Cruz studied maximum actions that occurred in concrete frame components under linear-dynamic analysis in different directions and compared results with ones that are brought from combinations of responses on two major directions. They concluded that SRSS of responses in two major directions estimates demands 25 percent less than which is obtained from analyses in different directions [12]. In 2005 a formulation was presented for the critical angle of incident and maximum response which was obtained by imposing the three-component ground motions [13]. Results show critical actions in the displacement of the elements and nodes vary up to 80 percent, considering different incident angles. Non-linear processing leads researchers to study the non-linear behavior of structures under directional excitation. Rigato and Medina studied the effect of incident angles on columns drift and ductility demands of regular and irregular buildings. They showed that these effects increase structure responses from 10 to 60 percent [14]. Also, Contaglo et al. demonstrate that concrete structures that suffer irregularity in the plan are vulnerable to the incident angle of excitation considerably. However, structures with a regular plan less affect by directional excitations [15]. Some new researches in directionality effects of seismic excitation focused on building responses incorporating damage index. Emami and Halabian imposed orthogonal components of ground motion in a range of angles to three concrete frames with the regular plan and considered rotation of response receiving axes to find a maximum response. They showed that directionality effects increase ductility demand and damage index between 10 to 30 percent [16]. Also, Reyes and Kalkan studied the rotation of ground motions pairs on a set of symmetric and asymmetric structures. He concluded that for a given ground motions pair, the rotation angle leading to maximum elastic response is different than that for maximum inelastic response [17]. If the ground motion is considered as a random process the principal axes can be determined as the set of axes along which the covariance between the accelerograms disappears [9]. In this case, the accelerograms are considered uncorrelated. Lagaros investigated the effect of directional uncertainty on the IDA approach and integrated this approach with directional uncertainty introduced as Multicomponent Incremental Dynamic Analysis (MIDA) method. In this method, two components of the earthquake were applied to structures with variations of incident angles from 0 to 360 degrees to show the variation of structural responses against different incident angles [18]. Dynamic effects have been evaluated in many structures [19-20]. Rupali et al used the response spectrum method to evaluate demand response in different incident angles, from 0 to 90 degrees with an increment of 10 degrees, and the variation of responses against different incident angles was concluded [21]. Detailed investigation [22, 23] per nonlinear response history analysis has broadly concluded that the incidence angle resulting in maximum seismic demand cannot be specified. Considering directionality effects as applied in previous researches result in severe demands on building, so jacket type offshore platform and a set of dynamic analyses are performed using rotated ground motions by multicomponent incremental dynamic analysis. Jacket type offshore structures are much expensive and vital, so it is very important to check their vulnerability. According to the previous studies, the importance of considering uncertainties in performing a structural analysis is undeniable since the outcome of structural analysis is supposed to be used as an input for risk assessment of a structure by using fragility analysis. Therefore, the more reliable and accurate responses are obtained by the structural analysis, the more reliable results are concluded by fragility analysis.

2. Functional Jacket-Type Offshore Platform

In this study, SPD13 as a JTOP located in South Pars Gas Field Phase-13 in Persian Gulf region was considered herein. Shaking a structure in the direction orthogonal to the main shaking direction increases the structural response. Drifts in the main shaking direction, indicating that two-dimensional (2D) analyses would not estimate the three-dimensional (3D) response well then 3D numerical model was generated. This platform is a four-legged jacket, and there is a grouted pile at the end of each leg. The four piles at the corners of platform are extended 99 m below the mud line elevation. The jacket dimensions in the horizontal plane at the top and bottom (mud line) are about 13.716 m × 20 and 28.376 m × 32.859. The mean water depth is 57.2 m (Figure 1).

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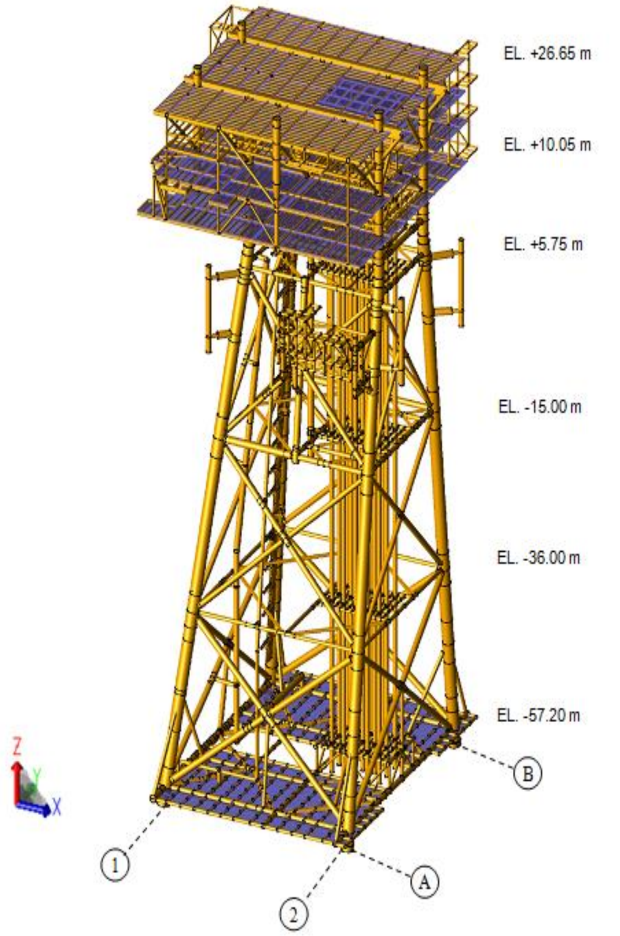

Fig. 1: SPD13 wellhead platform sacs model

Furthermore, bedrock elevation in this area is nearly at 110 m depth beneath the mud line, and schematic view of this jacket is shown in Figure 2.

Regarding figure 2, BNWF was employed to consider nonlinear pile-soil interaction. In the BNWF model, movements around pile, where pile-soil interaction occurs, is called near-field movement, and movements far from the pile are called far-field or free-field movement. We simulated far-field movements by 1-D wave propagation analysis program for geotechnical response analysis of deep soil deposits (DEEPSOIL) [24]; moreover, for near-field movements, we simulated these movements by independent springs (p-y, t-z, and q-z) connected horizontally and vertically to pile elements. P-y springs indicate soil reaction force versus lateral displacement response of pile in different layers; t-z springs consider the shear force transferred between the soil and the pile in various depths, and q-z springs provide end bearing resistance. The concept of dynamic BNWF model is shown in Figure 2 by springs. In addition, the dynamic model used for p-y elements in this paper is illustrated in Figure 3.

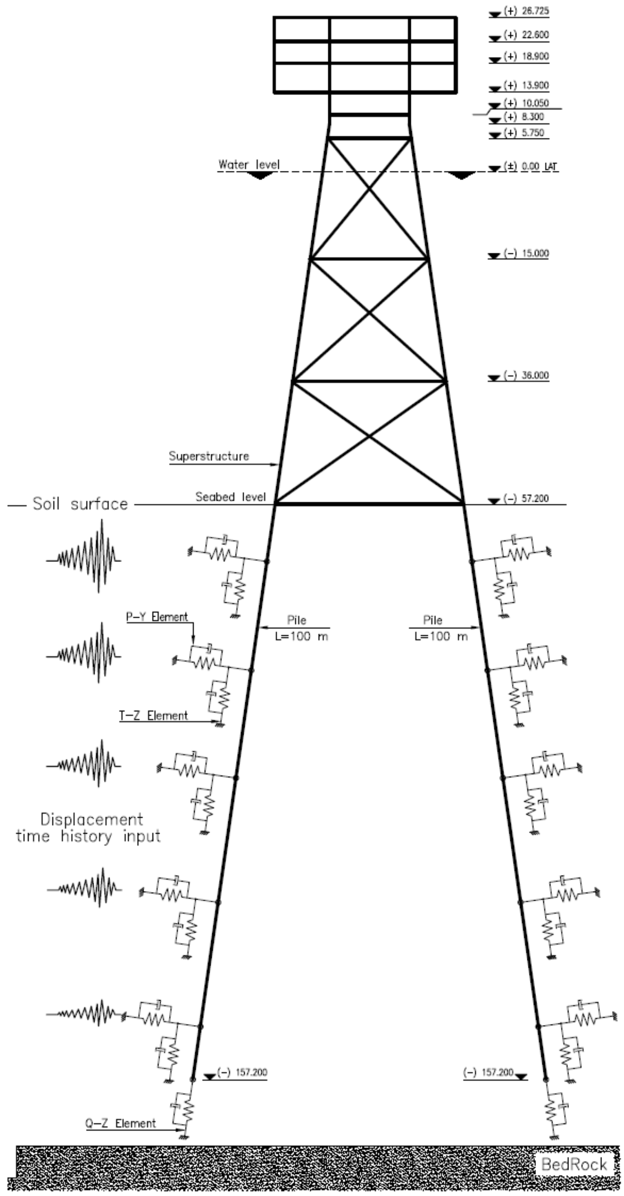

Fig. 2: Schematic view of SPD13 wellhead platform

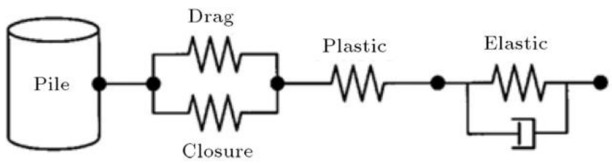

Fig. 3: Components of nonlinear p-y elements

Displacement time history calculated by DEEPSOIL was applied to p-y element along the pile. The BNWF model can take the variation of soil properties with depth, nonlinear soil behavior, dissipating energy by hysteric damping for radiation damping [25], and gapping effect [26] into account. Penzien et al. [27], Kagawa and Kraft [28], Nogami et al. [29], Boulanger et al. [30], and Naggar and Bentley [31] proposed several models of the BNWF. In this study, nonlinear p-y material with gapping capability (Figure 3), in addition to t-z and q-z materials, was modeled using the

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element described by Boulanger et al. (1999) [30]. The characteristics of these springs were estimated by employing recommendations in API RP-2A WSD [32]. For conserving free-field site response analysis, the authors used pressure dependent hyperbolic model for soil column behavior of this model, and Darendeli's models [33] were selected to be the reference curves. According to the information presented in reports of the South Pars phases, the overall layering of the region consists of soft clay sediments in depth throughout the area. As the depth increases, it rises from stiff clay deposits to very stiff ones and Sandy layers with low to moderate cementation that has been reported in table 1.

Table 1: Soil layer characteristics

Soil type (figure 2)	Soil layer depth (m)
Clay	0-10.6
Sand	10.6-14.1
Clay	14.1-15.2
Sand	15.2-17.2
Clay	17.2-42.2
Sand	42.2-45.1
Clay	45.1-53.5
Sand	53.5-54
Clay	54-57.5
Gravel	57.5-61.6
Clay	61.6-89.8
Sand	89.8-90.65
Clay	90.65-110.4

This structure is asymmetrical in plan and height, both in terms of geometry and mass distribution.

3. Methodology

To investigate the effect of directionality of two horizontal orthogonal components of earthquake ground motion, the following steps are used. In this section, the influence of the intensity level on the critical incident angle and the diversification of the MIDA curves with respect to the incident angle is examined in an effort to be considered in the directional multi-component incremental dynamic analysis (DMIDA) framework. The difference of the MIDA framework from the original one component version of the IDA, proposed by Vamvatsikos and Cornell [40], stems from the fact that for each record a number of IDA representative curves can be defined depending on the two components, while in most cases, the IDA curve is representative of the one component of an earthquake [41,42] and the difference of DMIDA is the extraction of MIDA in different incidence angles of earthquake pair components.

The first step, in order to perform DMIDA-based fragility analysis, is to select a suite of natural records to be used for performing the MIDA study. Based on previous studies it was found that ten to twenty records are sufficient for predicting with acceptable accuracy the seismic demand of a mid-rise building [36], for this reason, twelve horizontal Ground Motion (GM) records were selected from PEER Ground Motion Database [34]. The ground motion records are listed in Table 2, where the epicenter distances are 15-70 km, the PGA values for the longitudinal and transverse directions are 5.5 to 7.5 hertz, soil type is C and, the fault rupture mechanism is given for the twelve records in table 2. The second step concludes two main procedures for implementing MIDA. In the first one, the two horizontal components of the records are applied along two orthogonal principal axes, and According to the second procedure MIDA is performed over a sample of incident angles from 0 to 180 degrees that is DMIDA. In this study, we have rotated the axis of the coordinates of the incidence by 22.5 degrees [35,55], and the angles of 0, 22.5, 45, 67.5, 90, 112.5, 135, 157.5, and 180 are the angles of the earthquake incidence according to figure 4. final step is to adopt the 16%, 50%, and 84% fractile MIDA and DMIDA curves for developing the limit state fragility curves. It should be mentioned that according to API standard, seismic load and other environmental loads such as wave, wind, and current should not apply simultaneously to offshore structures [32].

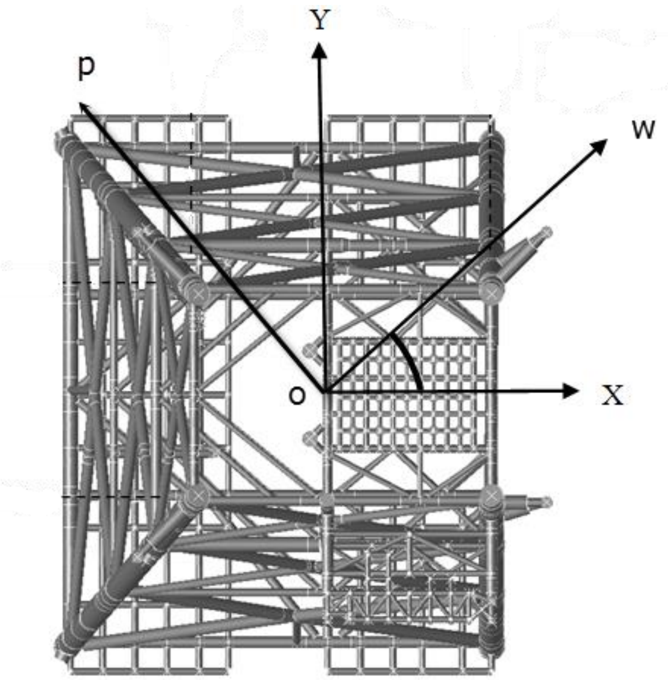

Fig. 4: Structural axes and rotational system

4. Directional Multicomponent Incremental Dynamic Analysis (DMIDA) and MIDA Results

The main objective of an IDA study is to define a curve through the relation of the intensity level with the maximum seismic response of the structural system. The intensity level and the seismic response are described through an intensity measure (IM) and an engineering demand parameter (EDP), respectively. The IM should be a monotonically scalable ground motion intensity measure like the peak ground acceleration (PGA), peak ground velocity (PGV), the ξ=5% damped spectral acceleration at the structure's first-mode period (SA(T1,5%)), and many others [36]. In the current work, housner intensity (HI) is selected based on the results of Babaei et al. for this kind of structure [37]. The two components of the records are scaled to HI preserving their relative scale. This is achieved by scaling the component of the record having the highest HI, while the second one follows the scaling rule preserving their relative ratio [18]. In the current work, the maximum inter-story drift ratio is chosen, belonging to the EDPs which are based on the maximum deformation, because there is an established relation between inter-story drifts and performance-oriented descriptions such as immediate occupancy, life safety, and collapse prevention [38,39]. the relation of IM-EDP is defined similarly to the one component version of the IDA, i.e. both horizontal components of each record are scaled to a number of intensity levels to encompass the full range of structural behavior from elastic to yielding that continues to spread, finally leading to global instability. According to the MIDA framework a set of natural records, each one represented by its longitudinal and transverse components, are applied to the structure in order to account for the randomness on the seismic excitation. In this work, a new procedure for applying MIDA is proposed which is based on the idea of considering variable incident angles for each record. Figure 5 presents the MIDA results for one record for different directions. Figure 6 shows the results of the MIDA analysis for the state in which the earthquake is applied along the principal axes. The results of the proposed implementation take into account the randomness both on the seismic excitation and the incident angle that is called directional multi-component incremental dynamic analysis (DMIDA), are presented in figure 7. In this figure, each diagram had the maximum value of EDP between the different incidence directions for one related earthquake record.

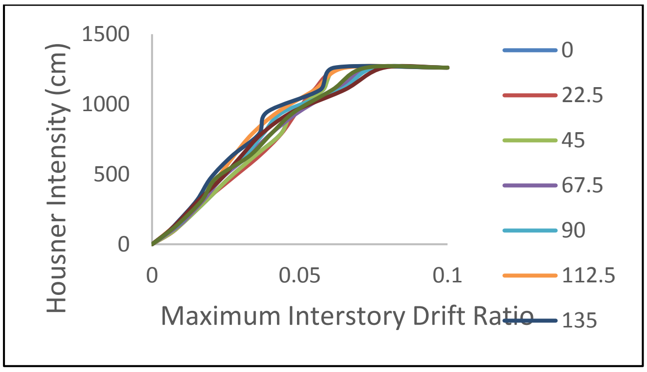

Fig. 5: MIDA diagrams for one record in all directions

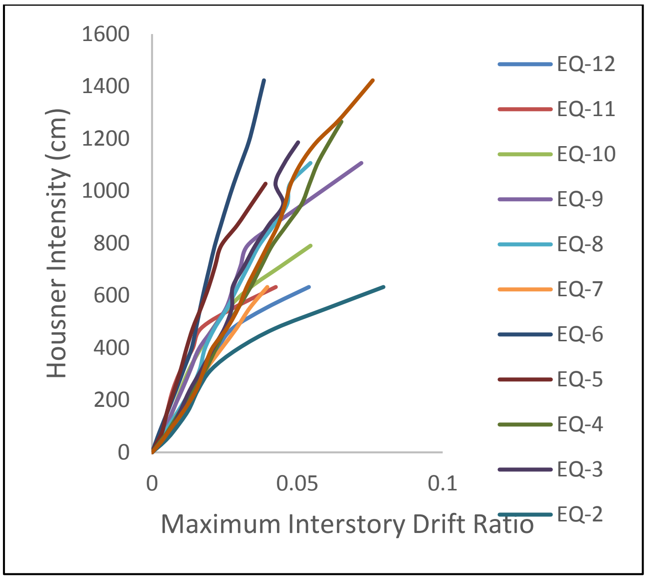

Fig. 6: diagrams of MIDA results (the earthquake is applied along the principal axes) for all EQs

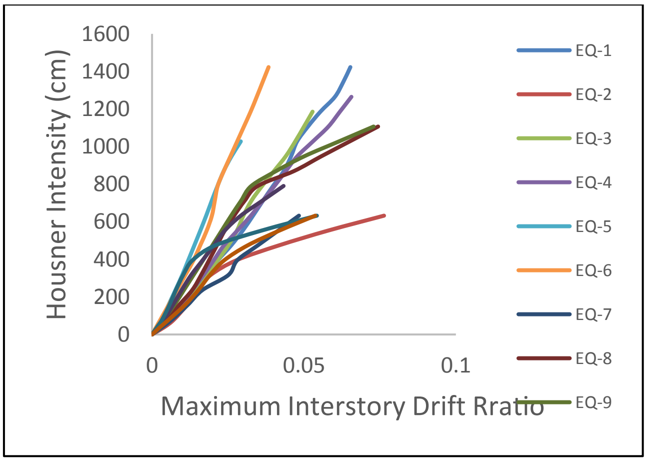

Fig. 7: diagrams of DMIDA results for maximum EDP

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Table 2: Seismic characteristics of 12 ground motion records

Rec no.	N.G.A.	Earthquake Name	Year	Station	V_s (m/s)	P.G.A.(g)	Mechanism
1	340	Coalinga-01	1983	Parkfield-fault zone 16	384.26	0.18 0.14	Reverse
2	359	Coalinga-01	1983	Parkfield - vineyard cany 1E	381.27	0.23 0.18	Reverse
3	369	Coalinga-01	1983	Slack Canyon	648.09	0.17 0.13	Reverse
4	954	Northridge-01	1994	Big Tujunga Angeles Nat F	550.11	0.25 0.17	Reverse
5	1020	Northridge-01	1994	Lake Hughes #12A	602.1	0.26 0.17	Reverse
6	190	Imperial Valley-06	1979	Superstition Mtn Camera	362.38	0.2 0.11	Strike slip
7	787	Loma Prieta	1989	Palo Alto - SLAC Lab	425.3	0.28 0.195	Reverse/oblique
8	991	Northridge-01	1994	San Gabriel - E Grand Ave	401.37	0.215 0.15	Reverse
9	1070	Northridge-01	1994	San Gabriel - E Grand Ave	401.37	0.26 0.14	Reverse
10	330	Coalinga-01	1983	Parkfield - Cholame 4W	410.4	0.13 0.13	Reverse
11	357	Coalinga-01	1983	Parkfield - Stone Corral 3E	565.08	0.15 0.11	Reverse
12	246	Mammoth Lakes-06	1980	Benton	370.94	0.18 0.11	Strike slip

For better summarization and interpretation, the results of FRACTILE 16, 50 and 84% for both MIDA and DMIDA have been calculated and presented in figure 8. As can be seen, directional uncertainty has shifted the results of the X-method, and this value is higher in the near-break region than in the elastic region. In addition, it shows that this uncertainty affects the seismic behavior of the structure and reduces the seismic capacity of the structure. The region that the maximum inter-story drift ratio is more than 0.06 is the collapse region.

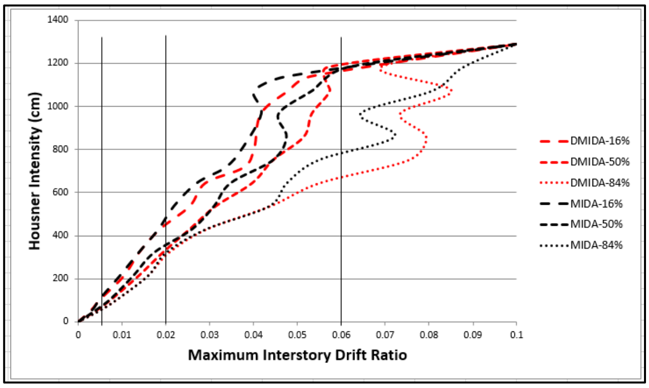

Fig. 8: Summarized diagrams of MIDA and DMIDA results

Figure 9 presents the probability of occurrence of each incident angle in all results. The critical incident angles were 0, 157.5, and 180 degrees, indicating that the critical areas in which the incident angles should be applied to the structure are the areas closest to the principal axis as was shown in figure 10.

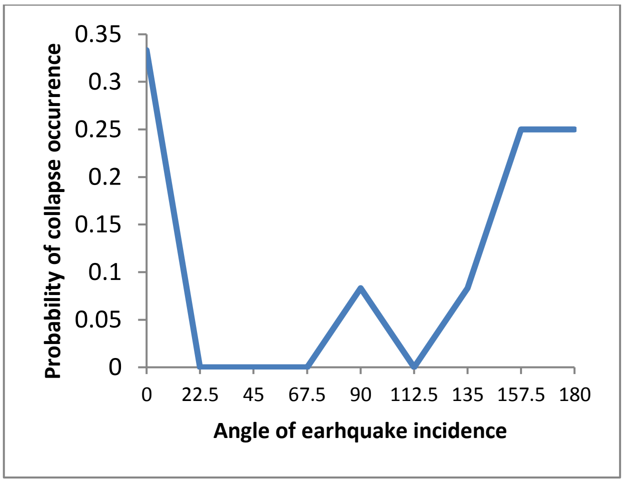

Fig. 9: The probability of collapse occurrence for each angle as critical

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5. Probabilistic Safety Assessment of Jacket-Type Offshore Platform

Representation of seismic vulnerability of structures in terms of fragility curves is a well-accepted and appropriate way for a meaningful risk assessment [45-53]. A fragility curve represents structural reliability in the form of a probability distribution function, which is a function of the ground motion intensity. Structure damageability in the form of fragility curves can be incorporated easily in the evaluation of the seismic performance of offshore structures. For the purpose of the present study, HI is considered to represent ground motion intensity and the maximum inter-story drift ratio is the engineering demand parameter. Three limit states are selected: abnormal, extreme, and complete structural damage states. The inter-story drift limits are equal to 0.5%, 2.0%, for abnormal and extreme according to ISO 19901-2 [43] and 6.0% for complete structural damage state according to Jahanitabar et al. [44].

To perform damage analysis, the outputs of previous phases i.e. EDPs and DMs are used. In this phase, the conditional probability of structure responses exceeding damage states of Si at a specific PGA level is obtained as fragility curves (Eq.1) [54,35].

$$P[S > s|PGA] = P[X > x_i|PGA] = 1 - \Phi\left[\frac{\ln x_i - \lambda}{\zeta}\right] \quad (1)$$

Where ϕ is normal cumulative distribution function, x_i is the upper bound for s_i (i=I, II, III), and λ *and* ζ are mean value (μ) and standard deviation (σ) of the sample population (x) in each scaled level (Eq.2, 3, and 4).

$$\lambda = ln\mu\zeta^2, \zeta^2 = ln[1 + \delta^2], \delta = \frac{\sigma}{\mu} \quad (2, 3 \text{ and } 4)$$

As common post-processing of developed fragility curves, it is typical to use simplified fragility curves. These curves are typically expressed by lognormal cumulative distribution functions (lognormal CFD). Using lognormal CFD makes the application of fragility curves more convenient (Eq.5).

$$F_A(a) = \int_0^a \frac{1}{\sqrt{2\pi\zeta Aa}} exp\left[-\frac{1}{2}\left(\frac{lna - lnm_A}{\zeta_A}\right)^2\right] da \quad (5)$$

Where A is the random variable of the PGA, m_A is the median of A, and ζ_A is the logarithmic standard deviation of A. In figure 11, fragility curves are developed for all damage states and for IDA and DMIDA analysis. According to this figure, the seismic performance of the structure is affected by directional uncertainty as the intensity of earthquakes increases, and in the collapse point especially. For the abnormal level earthquake damage state, the fragility curve was shifted 1 percent, for the extreme level earthquake, 3%, And in collapse point, 10 percent. This result has proved the previous research that said directional uncertainty has many effects on the inelastic zone on the seismic structural behavior.

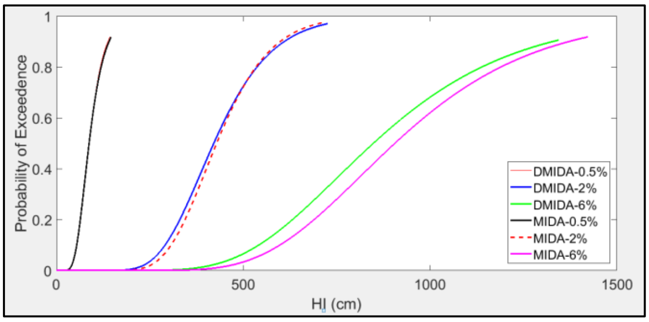

Fig. 10: Fragility curve for both MIDA (dash line) and DMIDA (continuous line) for three damage states

6. Comments and Conclusions

In this paper, the systematic methodology of PEER PBEE analysis is adopted in order to assess the seismic performance of the jacket type offshore structure of the Persian Gulf against directional uncertainty of earthquakes. According to the step-by-step procedure of the methodology of PEER, 12 ground motions are selected, and MIDA analysis is performed in 9 incident directions to measure DMs and EDP, respectively. To show the effect of directional uncertainty on the seismic performance of the structure, the third phase of this methodology is performed by developing fragility curves. By comparing fragility curves, it is shown that the severity of directional uncertainty reveals when the damage measures change from moderate damage to extensive damage. In addition, by using and comparing fragility curves for the selected incident angles, the critical directions among these incident angles for abnormal, extreme, and collapse damage states are the principal X-axis, the angles close to it as 22.5 and 157.5. The chosen structure is asymmetrical in plan and height, both in terms of geometry and mass distribution, Therefore, these results can be generalized to symmetric structures. Also, changing the results by 10% is a point that should be considered in reliability analyzes.

References

[1] Lopez, O. A., and Torres, R., (1997), “The critical angle of seismic incidence and the maximum structural response”, Earthquake Engineering and Structural Dynamics, Vol. 26, no. 9, pp. 881– 894.

[2] Lopez, O.A., Chopra, A.K., Hernandez, J.J., (2000), “Critical response of structures to multicomponent earthquake excitation", Earthquake Engineering and Structural Dynamics, 29:1759–1778.

[3] MacRae, G.A., Mattheis, J., (2000), "Three-dimensional steel building response to near-fault motions", Journal of Structural Engineering, 126(1): 117-126.

[4] Khoshnoudian, F., and Poursha, M., (2004), "Response of three-dimensional buildings under bi-directional and unidirectional seismic excitations", Proceedings of 13th world conference on earthquake engineering.

[5] Torbol, M., and Shinozuka, M., (2012), "The directionality effect in the seismic risk assessment of highway networks", Structure and Infrastructure Engineering, 10(2): pp.175-188.

[6] Nguyen, V.T., and Kim, D.K., (2013), "Influence of incident angles of earthquakes on inelastic responses of asymmetric-plan structures", Struct. Eng. Mech., 45(3), 373-389.

[7] Reyes, J. C., and Kalkan, E., (2015), 'Significance of rotating ground motions on behavior of symmetric- and asymmetric-plan structures: Part I. Single-story structures", Earthquake Spectra, vol.31, pp.1591–1612.

[8] Kalkan, E., and Reyes, J. C., (2015), "Significance of rotating ground motions on behavior of symmetric- and asymmetric-plan structures: Part II. Multi-story structures", Earthquake Spectra, 31, 1613–1628.

[9] Penzien J., and Watabe M., (1975), "Characteristics of 3-dimensional earthquake ground motions, Earthquake Engineering and Structural Dynamics", 3(4): 365-373.

[10] Menun, C., and Der Kiureghian, A., (1998), "A Replacement for the 30%, 40%, and SRSS Rules for Multicomponent Seismic Analysis", Earthquake Spectra, 14(1):153–63.

[11] Wilson, EL., Suharwardy, I., and Habibullah, A., (1995), "A Clarification of the Orthogonal Effects in a Three-Dimensional Seismic Analysis", Earthquake Spectra, 11(4):659–66.

[12] Davila, F., and Cruz, E., (2004), "Study of the effect of in-plan asymmetry in multistory buildings subjected to uni- and bi-directional seismic motions", 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada, August 1-6, 2004, Paper No. 1042.

[13] Athanatopoulou, AM., (2005), "Critical orientation of three correlated seismic components", Engineering Structures, 27(2):301–312.

[14] Rigato, AB., and Medina, RA., (2007), "Influence of angle of incidence on seismic demands for inelastic single-storey structures subjected to bi-directional ground motions", Engineering Structures, 29(10):2593–601.

[15] Cantagallo, C., Camata, G., and Spacone, E., (2012), "The Effect of the Earthquake Incidence Angle on Seismic Demand of Reinforced Concrete Structures", 15 WCEE, In Lisboa.

[16] Emami, AR., and Halabian, AM., (2015), "Spatial distribution of ductility demand and damage index in 3D RC frame structures considering directionality effects: Spatial Distribution of Ductility Demand and Damage Index", Struct Des Tall Spec Build, 24(16):941–61.

[17] Reyes, J.C., and Kalkan, E., (2013), "Significance of Rotating Ground Motions on Behavior of Symmetric- and Asymmetric-Plan Structures: Part I". Single-Story Structures", Earthq Spectra, 31(3):1591–612.

[18] Lagaros, N.D., (2010), "Multicomponent incremental dynamic analysis considering variable incident angle", Structure and Infrastructure Engineering, 77–94.

[19] Jafari, A., and 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

[20] Dezvareh, R., (2019), "Application of Soft Computing in the Design and Optimization of Tuned Liquid Column–Gas Damper for Use in Offshore Wind Turbines", International Journal of Coastal and Offshore Engineering, 2(4), 47-57.

[21] Rupali, J. and Jaiswal, J., (2017), "Study of Effect of Seismic Excitation Angle for the Analysis of Regular and Irregular RC Frame", Mechanical and Civil Engineering, vol.14, pp.80-83.

[22] Bisadi, V., Head, M., (2011)," Evaluation of combination rules for orthogonal seismic demands in nonlinear time history analysis of bridges", J Struct Eng, ASCE, 16(6):711–7.

[23] Fontara, I.K.M., Kostinakis K.G., Manoukas G.E., Athanatopoulou A.M., (2015), "Parameters affecting the seismic response of buildings under bi-directional excitation", Struct Eng Mech, 53(5):957–79.

[24] Hashash, Y.M.A., (2011) DEEPSOIL V 5.0, User Manual and Tutorial, 2002-2011, University of Illinois at Urbana-Champaign.

[25] Wang, S., Kutter, B.L., Chacko, M.J., Wilson, D.W., Boulanger, R.W. and Abghari, A., (1998), "Nonlinear seismic soil-pile structure interaction", Earthquake Spectra, 14, pp. 377-396.

[26] Matlock, H., Bryant, L.M. and Foo, S.H.C., (1978), "Simulation of lateral pile behavior under earthquake motion", Earthquake Engineering and Soil Dynamics, Pasadena, CA, American Society of Civil Engineers, pp. 600-19.

[27] Penzien, J., Scheffey, C.F., and Parmelee, R.A., (1964)," Seismic analysis of bridges on long pile", Journal of the Engineering Mechanics Division, vol.90, pp. 223-254.

[28] Kagawa, T., and Kraft, L.M., (1980), "Seismic p-y responses of exible piles", Journal of the Geotechnical Engineering Division, 106, pp. 899-918.

[29] Nogami, T., Otani, J., Konagai, K. and Chen, H., (1992), "Nonlinear soil-pile interaction model for dynamic lateral motion", Journal of Geotechnical Engineering, 118, pp. 89-106.

[30] Boulanger, R., Curras, C., Kutter, B., Wilson, D. and Abghari, A., (1999), "Seismic soil-pile structure interaction experiments and analyses", Journal of Geotechnical and Geo environmental Engineering, 125, pp. 750-759.

[31] Naggar, M.H.E., and Bentley, K.J., (2000), 'Dynamic analysis for laterally loaded piles and dynamic p-y curves", Canadian Geotechnical Journal, 37, pp. 1166-1183.

[32] API., Recommended Practice for Planning, Designing and Constructing Fixed Offshore Platforms {Working Stress Design (API RP2A-WSD), Washington (DC), American Petroleum Institute (2005).

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[DOR: 20.1001.1.23454296.2022.6.4.7.4]

[DOI: 10.52547/nmce.6.4.29]

[33] Darendeli, M.B., (2001), “Development of a New Family of Normalized Modulus Reduction and Material Damping Curves”, University of Texas at Austin.

[34] Pacific Earthquake Engineering Research Center (PEER), (2107) Data from: PEER Ground Motion Database. Retrieved from http://ngawest2.berkeley.edu/users/sign_in.

[35] Soltani, M., Amirabadi, R., (2019), “Sensitivity analysis of pile supported wharves against directional uncertainty of earthquakes using fragility curves”, International Journal of Maritime Technology, IJMT, Vol.11, pp.33-40.

[36] Giovenale, P., Cornell, C.A., and Esteva, L., (2004), “Comparing the adequacy of alternative ground motion intensity measures for the estimation of structural responses”, Earthquake Engineering and Structural Dynamics, 33(8): 951-979.

[37] 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, pp.4330-43.

[38] Ghobarah, A., Abou-Elfath, H., and Biddah, A., (1999), “Response-based damage assessment of structures”, Earthquake Engineering and Structural Dynamics, 28(1): 79-104.

[39] FEMA 273, NEHRP Guidelines for seismic rehabilitation of buildings, Federal Emergency Management Agency, Washington, DC, (1997).

[40] Vamvatsikos, D., and Cornell, C.A., (2002), “Incremental dynamic analysis”, Earthquake Engineering and Structural Dynamics, 31(3): 491-514.

[41] Vamvatsikos, D., (2006), “Incremental dynamic analysis with two components of motion for a 3D steel structure”, Proceedings of the 8th US National Conference on Earthquake Engineering, San Francisco.

[42] Serdar Kircil, M, and Polat, Z., (2006), “Fragility analysis of mid-rise R/C frame buildings”, Engineering Structures, 28(9): 1335-1345.

[43] ISO, Petroleum and natural gas industries-specific requirements for offshore structures (ISO 19901-2)", Part 2, Seismic Design Procedures and Criteria, Switzerland, International Standard Organization (2004).

[44] Jahanitabar, A.A., and Bargi, Kh., (2018), “Time-dependent seismic fragility curves for aging jacket-type offshore platforms subjected to earthquake ground motions”, J Struct & Infrastruct Eng, 14(2):192–202.

[45] Hwang, H. H. M., and Huo, J.-R., (1994), “Generation of hazard consistent fragility curves for seismic loss estimation studies”, Tech. Rep. NCEER-94-0015, National Center for Earthquake Engineering Research (NCEER), State University of New York, Buffalo, NY, USA.

[46] Fukushima, S., Kai, Y., and Yashiro, K., (1996), “Study on the fragility of system—part 1: structure with brittle elements in its stories,” in Proceedings of the 11th World Conference on Earthquake Engineering, Pergamon, Elsevier Science Ltd., Paper No. 333.

[47] Singhal, A., and Kiremidjian, A. S., (1998), “Bayesian updating of fragilities with application to RC frames”, Journal of Structural Engineering, vol. 124, no. 8, pp. 922–929.

[48] Shinozuka, M., Feng, M. Q., Lee, J., and Naganuma, T., (2000) “Statistical analysis of fragility curves,” Journal of Engineering Mechanics, vol. 126, no. 12, pp. 1224–1231

[49] Karim, K. R., and Yamazaki, F., (2001), “Effect of earthquake ground motions on fragility curves of highway bridge piers based on numerical simulation”, Earthquake Engineering and Structural Dynamics, vol. 30, no. 12, pp. 1839–1856.

[50] Shinozuka, M., Feng, M. Q., Kim, H., Uzawa, T., and Ueda, T., (2003), “Statistical analysis of fragility curves”, Tech. Rep. MCEER- 03-0002, Multidisciplinary Center for Earthquake Engineering Research (MCEER), The State University of New York, Buffalo, NY, USA.

[51] Choi, E., DesRoches, R., and Nielson, B., (2004), “Seismic fragility of typical bridges in moderate seismic zones,” Engineering Structures, vol. 26, no. 2, pp. 187–199

[52] Banerjee, S., and Shinozuka, M., (2007) “Nonlinear static procedure for seismic vulnerability assessment of bridges”, Computer-Aided Civil and Infrastructure Engineering, vol. 22, no. 4, pp. 293– 305.

[53] Banerjee, S., (2007), “Statistical empirical and mechanistic fragility analysis of concrete bridges”, Ph.D. dissertation, University of California, Irvine, Calif, USA

[54] Chiou, J.S., Chiang, C.H., Yang, H.H., and Hsu, S.Y., (2011), “Developing fragility curves for a pile supported wharf”, Soil Dynamics and Earthquake Engineering, vol. 31(5), p.p. 830-840.

[55] Soltani, M., and Amirabadi, R., (2021), “Seismic Vulnerability Assessment Of Pile-Supported Wharves Using Fragility Surfaces”, Journal of Earthquake Engineering, DOI: 10.1080/13632469.2021.1961926.

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