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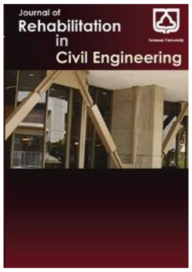

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Seismic Vulnerability Assessment of Jacket Type Offshore Platforms

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ABSTRACT

Most of oil and gas offshore platforms are located in the seismic regains. Therefore, Seismic vulnerability evaluation of the offshore platforms is one of the essential vital issues in the structural systems. In this article, jacket type offshore platforms are examined by incorporating the pushover analyses and nonlinear time history analyses, in such a way that, first some push over analyses are performed to detect the more critical members of the jacket platform in consonance with the range of their plastic deformations. Subsequently, nonlinear time history analyses are performed, concentrating on the critical members, to examine the vulnerability of the jacket platform under intensive earthquake loads. Pursuant to the numerical results, the combination of the push over analyses and nonlinear time history analyses proposes a reliable and swift seismic assessment procedure to evaluate the seismic vulnerability of the offshore platforms. Moreover, seismic vulnerability of the offshore structures is dependent on the critical member locations and their load bearing situations in the offshore structures.

1. Introduction

Oil- and gas-related offshore structures are considered as vital structures all over the world, and any damage to them can affect the world energy conditions, and lead to adverse economic and environmental consequences. On the other hand, most of these structures are located in seismic regions, and the past earthquakes have revealed that they are vulnerable. Consequently, it is necessary to upgrade the seismic behavior of the existing gas- and oil-related offshore structures. Apparently, to upgrade their seismic behavior, it is required to evaluate their seismic vulnerability in various conditions, with the highest possible precision. It is believed that the most reliable type of analysis for seismic evaluation is Nonlinear Time History Analyses (NLTHA); nonetheless, this type of analysis is very time-consuming. Thus, a quick procedure for

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seismic evaluation is greatly acknowledged, particularly for professional practice in engineering offices.

Since the late 70s, several researchers have discussed the application of nonlinear dynamic or NLTHA and its usage in seismic design or seismic evaluation of offshore structures which some are reviewed here briefly. Watt and his colleagues (1978, [10]) examined the earthquake survivability of a typical concrete gravity platform by a series of nonlinear dynamic analyses and applying some extreme magnitude earthquakes. The characteristics of the structure, its foundation, and the earthquake inputs were varied among 15 analysis cases. They utilized the scaled natural and artificial accelerograms with free field velocities up to 40 inches per sec. They concluded that suitably designed platforms are capable of surviving extreme ground shaking conditions likely to be associated with rare intense earthquakes in the Gulf of Alaska.

Kamil (1978, [5]) proposed a procedure for the nonlinear design of offshore structures subjected to extreme loads such as strong-motion earthquakes. He proposed a design based on the safety level earthquake (SLE) and inspected for the operating level earthquake. A reduced inelastic response spectrum applied for the SLE. He suggested the preliminary design by using a response spectrum approach. Subsequently, nonlinear analyses performed utilizing artificial time histories of ground motions, compatible with the inelastic spectrum. Then, the preliminary design modified to obtain the final design. Finally, the reliability of the final design could be estimated using a deterministic-cum-probabilistic procedure.

Ueda and Shiraishi (1979, [9]) presented their observation of oscillation and the vibrational characteristics of a deepwater platform with vertical and oblique piles and the ground during earthquakes. Mentioning that in spite of the fact that many deep-water platforms have been constructed, there were no earthquake records to evaluate the earthquake-resistant design of deep-water terminals. They examined a platform which was a 200,000 DWT oil tanker terminal supported by 10 vertical and 8 oblique piles, operating in Kashima, Japan. Three sensors were set in the ground and four on the platform. Six earthquake records acquired in March 1978 were analyzed, and the dynamic response characteristics of the platform were explored. Frequency spectra and response spectra were produced from the data. A multi-node nonlinear computation program was developed, and the earthquake responses of a pile foundation platform or a jacket-type platform were calculated, and finally, theoretical results were compared with their observations.

Zayas and his colleagues (1981, [11]) presented a state-of-the-art on the computer analysis of the inelastic structural response of braced steel offshore structures for seismic loading. Mentioning that such analyses had been applied before in the offshore industry, but their reliability and limitations had not been verified against experimental data. Zayas and his colleagues compared the analytical results with experimental results obtained for two X-braced tubular steel frames in order to assess the modeling techniques used in that time. Several types of analytical brace models were reviewed, and comments on the inelastic structural theoretical bases, correlation with experimental results and practicability, were included. From those models, a particular phenomenological brace model was selected and implemented, and the required input parameters were identified. Differences in the internal distributions of member inelastic deformations were evaluated when the frame analyses were compared to the experimental results. Those differences resulted in a different deterioration of frame lateral load capacities compared to those observed experimentally. Nonlinear solution schemes were evaluated, and a step-by-step solution procedure with unbalanced load correction was adopted. Suggestions for improving frame and brace modeling were offered and an ameliorated physical theory brace model employing plasticity theory was suggested, offering more realistic modeling of member deformations and depending less on empirical data.

Banon and his colleagues (1994-a, [3], and 1994-b, [4]) have worked on the assessment of fitness for offshore platforms. Their work has two parts: I) analytical methods and inspections, and II) risk management, maintenance, and repair. They concluded that the acceptance criteria for existing structures should not necessarily be the same as those for new designs, and they should be dependent on the consequences of structural failure. They also claimed that because of a large population of aging offshore platforms worldwide, reassessment of platforms to determine their fitness had gained remarkable attention by the oil and gas industry and regulatory agencies in the world. They finally stated that the reassessment process could be time-consuming and costly. Because it requires many steps such as: gathering information on design and physical condition of the platform, modeling of all-important damage found, structural evaluation of the platform, calculation of reliability indices, determination of mitigation and repair schemes and consequently, a reassessment process covers a large span of technical topics and require more effort and expertise compared to a new design.

In a study on seismic and vibration mitigation of offshore platform systems (Lee, 1998, [8]) an improved design method for the traditional A-type or V-type offshore template platform systems was proposed to mitigate the vibration induced by the marine environmental loadings and the strong ground motions of earthquakes. He carried out a nonlinear dynamic analysis in the time domain. The analysis was focused on the displacement and rotation induced by the input wave forces and ground motions, and the mitigation effect for these responses was evaluated when the viscoelastic damping devices were applied. A step by step integration method was modified and utilized in the nonlinear analysis. He mentioned that the proposed design approach with viscoelastic dampers was efficient for the mitigation of vibrations in the structural system subjected to both wave motion and strong ground motion.

Kawano and his colleague (2003, [7]) did an examination on seismic response evaluations of an offshore structure with uncertainties, mentioning that for an offshore structure located in a seismic area, it is essential to clarify the dynamic response characteristics as a result of seismic forces, especially, its nonlinear dynamic response to severe earthquakes. They carried out the dynamic response analyses by applying the increment method in the time domain, considering the uncertainties with respect to the strength of structural materials as well as the dynamic loads. They emphasized the importance of clarification of these uncertain parameter contributions to the responses in order to get reliable nonlinear dynamic responses. They applied Monte Carlo simulation for their examination and showed that since the uncertain parameter effects on the response evaluations play the important contribution on the nonlinear response, it is essential to clarify these effects on the nonlinear maximum dynamic response quantities.

Asgarian and Aghakouchack (2004, [1]) did research on nonlinear dynamic analysis of jacket type offshore structures subjected to earthquake utilizing fiber elements. Mentioning that jacket type offshore platforms in seismically active areas should meet two specific levels of earthquake requirements, named strength and ductility levels, and that overall structural response of this type of platforms in the nonlinear range of deformation, greatly depends on the buckling mode, post-buckling and hysteresis behavior of jacket braces as well as nonlinear behavior of jacket frame elements. They have tried to formulate the Fiber Beam-Column Post Buckling Element and implement that formula in the non-linear program DRAIN-3DX to predict buckling, post-buckling and hysteresis behavior of tubular struts and portals. The formulated element, in which both material and geometric nonlinearities are considered, was employed to simulate the nonlinear dynamic response of sample jacket type offshore structures subjected to earthquake time history. They proposed that the predicted overall response matched well with the available experimental and other analytical results.

Asgarian and Ajamy (2006, [2]) examined the nonlinear dynamic behavior of offshore structures applying incremental dynamic analysis (IDA). Mentioning that nonlinear dynamic analysis for offshore structures has been a major challenge in marine structural and earthquake engineering. They studied the behavior of jacket type offshore platforms through IDA, using twenty earthquake records in different levels and presented the results in terms of drift and displacements. They utilized the fiber element of OPENSEES software for modeling the members' nonlinear behavior and bilinear stress-strain curves with 5% strain hardening for the material behavior.

Nguyen and Le in 2015 presented finite element algorithms and dynamic analyses of the jacket type offshore structure underwater wave and wind impact by applying the stoke's second-order wave theory. The result of that study was the scientific basis for the calculation, design, and selection of the appropriate parameters, the optimization of fixed offshore structures such as buildings DKI, serving defense, security and contributing to improving the capacity to defend Vietnam's sovereignty over seas and islands.

A simplified method which is based on static pushover analysis was proposed by Zolfaghari and his colleagues in 2015 to assess the seismic performance of existing jacket type offshore platforms in regions ranging from near-elastic to global collapse. Subsequently, an existing jacket type offshore platform in the Persian Gulf was presented to demonstrate that procedure, and finally, a comparison was made between the above-simplified method and interaction incremental dynamic analyses results. In consonance with results, the simplified method is very informative and practical for current engineering purposes. It was able to predict seismic performance elasticity to global dynamic instability with reasonable accuracy and little computational effort.

Influence of hydrodynamic forces and ice during earthquakes was examined by Jia in 2016. He proposed that in comparison to land-based structures, offshore structures cherish unique effects of fluid-structure interactions: the hydrodynamic forces as a result of the relative velocity and acceleration between structural members and their surrounding waters.

Lotfollahi-Yaghin and his colleagues in 2016 investigated the efficiency of a tuned liquid damper in controlling the dynamic responses of offshore jacket-type platforms under earthquake loads. This type of dampers consisting of a number of fluid-containing tanks should be installed on the top side of the platform. Hydrodynamic loads induced by the sloshing of the fluid inside the tank act as resistant forces against the vibration and can thus control the structural response. In that research, a jacket-type platform having dimensions appropriate for the Persian Gulf climate was modeled and then dynamically analyzed by the modal and time history analyses subjected to the records of El Centro, Kobe, and Tabas earthquakes. The tuned liquid dampers were optimally designed, and after the verification of finite element results, the dynamic responses of the jacket-type platforms with and without the tuned liquid damper system were compared.

A review of vibration control methods and their application in marine offshore structures was done by Kandasamy and his colleagues in 2016. First, a review of the general approaches following the conventional categorization of passive, active, semi-active, and hybrid was presented. Next, a review of the specific marine offshore vibration control methods and a comparison of the approaches were examined. According to the results, the general trend is progressing towards semi-active and hybrid vibration control from passive or active control, as they provide more practical approaches for implementation, possessing the advantages of passive and active control systems.

It’s been observed that in spite of several studies in which NLTHA has been applied, combining the static and dynamic analyses for achieving a quick assessment method has not been mentioned yet. This paper presents a quick procedure for seismic vulnerability evaluation of offshore structures by combining the Push Over Analysis (POA) and the NLTHA. The POA is preformed first to recognize the more critical members of the jacket, based on the range of their plastic deformations. Then the NLTHA is performed to acquire more precisely the amount of vulnerability of critical members. To demonstrate the efficiency of the proposed method, an offshore structure of jacket type has been considered with 304 feet high, and its members are all of the tubular section. Several NLTHA has been performed to find out the effect of earthquake intensity on the vulnerability of the jacket structure by using the 3-components accelerograms of 100 earthquakes, covering a wide range of frequency content, all normalized to some specific levels of peak ground acceleration. In these analyses, the stress and strain values have been of the main concern, particularly plastic strains in critical members. The variations of maximum stress and strain values in critical member versus different features of the input earthquakes have been examined to find out which feature has the dominant effect; including frequency content, spectral intensity, duration, energy, and so on. The results of POA and NLTHA of the considered jacket structure are presented and discussed briefly in the following sections of the paper.

2. Introducing the Jacket Structure and Its Features

The examined offshore platform is jacket type and has been installed in Lavan oil field in the Persian Gulf in 1970. It is 304 feet high and has a deck of 96 feet by 94 feet, being carried on four inclined legs of 3 feet diameter. The total weight of the jacket and deck is over 290 million pounds (more than 131,000 tonf), and its members are all of the tubular section. Figure 1 indicates the geometry of the jacket and platform.

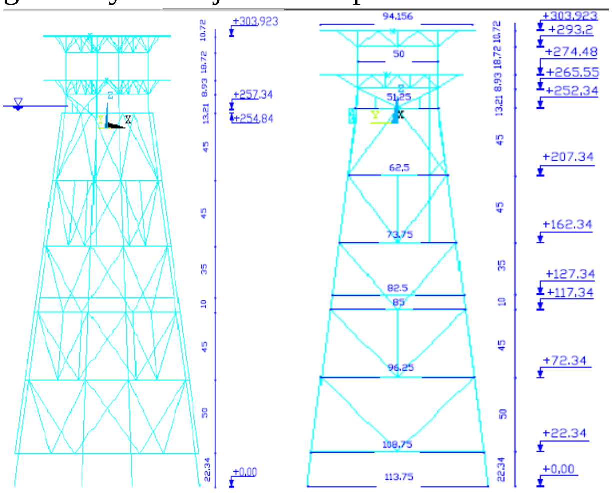

Fig. 1. The geometric features of the jacket structure.

The material of the jacket structure is high-strength steel with the modulus of elasticity of 2.1E6 kgf/cm2 and yielding stress of 3600 kgf/cm2, giving a yielding strain of 0.171%. Based on the mentioned structural specifications of the jacket, its modal properties up to 8 modes are depicted in Table 1, and the modal shapes of the first three modes are illustrated in Figure 2.

Table 1. Modal frequencies and periods of the jacket structure.

Mode No.	1	2	3	4	5	6	7	8
Freq. (Hz)	0.41	0.43	0.75	1.31	1.32	1.43	1.44	1.49
Period (sec)	2.41	2.34	1.33	0.76	0.76	0.70	0.70	0.67

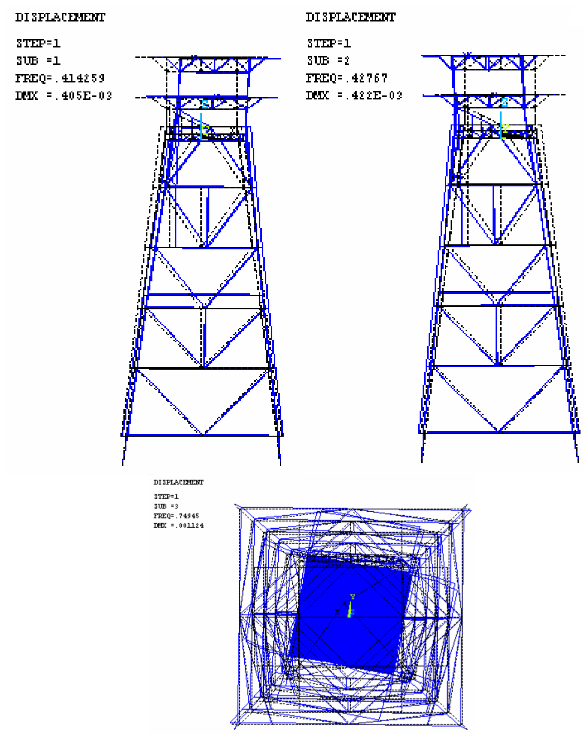

Fig. 2. Modal shapes of the first three modes of the jacket structure.

It can be seen in Table 1 that there is just a slight difference between the frequencies of the 6th and upper modes up to the 11th. This means that the structure has several closely-spaced modes, and this makes its modal analysis a very deliberate one, in which using the ordinary modal combination methods like SRSS is not adequate. Moreover, it is seen in Figure 2 that although the first two modes correlated to the lateral motions of the structure in the two main directions (X and Y), the third mode is a torsional mode.

Regarding the high number of elements in the jacket structure and the large volume of dynamic analyses outputs on the one hand, and the very long time which is required for nonlinear time history analysis (NLTHA) on the other, for seismic evaluation of the jacket structure, at first a set of push-over analyses (POA) is performed to find out the critical members of the structure so that in the time history analyses only the results of the critical members are selected as outputs. The performed POA and NLTHA, and their results are presented in the following sections.

3. Push Over Analysis

The POA was performed to recognize the more critical members of the jacket based on the range of their plastic deformations. To imply this, a concentrated load was applied at the master joint of the top level of the upper platform once in one main direction (X) and again in the other main direction (Y). Since the jacket structure is a little asymmetric, the POA was repeated for opposite directions (–X and –Y) as well. The more critical members of the structure identified based on the plastic deformations are indicated in Figure 3.

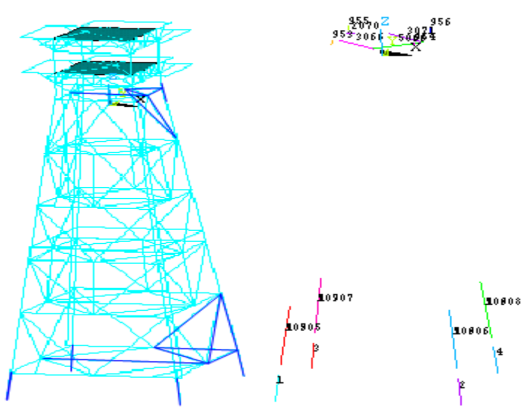

Fig. 3. The more critical members of the jacket structure identified by POA.

Based on the POA, the yielding forces and yielding displacements of the jacket structure were obtained as illustrated in Table 2.

The closeness of stiffness values in X and –X directions, as well as Y and –Y directions, indicate the satisfactory precision of POA. The close values of frequencies of the first two modes are also confirmed by the close values of stiffness in the two main directions of X and Y, which are around 50,175,000 lb/ft and 49,260,000 lb/ft, respectively. Although the stiffness values at X and Y directions are close, the yielding forces in these two directions are different. This can be due to the difference in the dimensions of the two main directions. Even though, in spite of very close stiffness values in the opposite X and –X and also Y and –Y directions, the yielding forces in these opposite directions are a little different. This little difference can be because of the non-uniform distribution of vertical loads on the legs of the jacket, which results in a little difference in the values of normal stress in the legs sections as depicted in Figure 4.

Table 2. Yielding forces-displacements of the jacket structure obtained by POA.

	Push (X)	Push (-X)	Push (Y)	Push (-Y)
Yielding Force (lb)	88,499,185	92,158,817	84,579,280	86,261,183
Yielding Displ. (ft)	1.760	1.840	1.715	1.753
Stiffness (lb/ft)	50,278,384	50,076,738	49,316,160	49,213,052

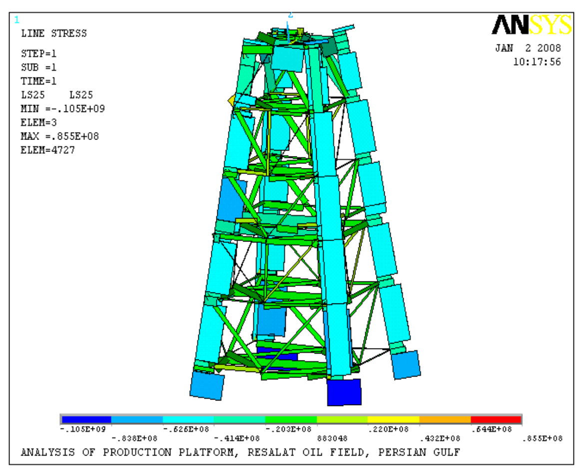

Fig. 4. Diagram of normal stress values in the jacket legs due to vertical loads.

As it is shown in Figure 4, the stress values are a little higher in a part of the two left legs (according to the figure) and in one of the piles as well. These slightly higher stress values in some members under vertical load can cause the start of plastic deformation in these members when the structure is pushed in one direction some step(s) earlier than the counterpart members in the other side of the jacket when the structure is pushed in the opposite direction.

4. Time History Analysis

By applying the critical members, realized by POA, the NLTHA was performed by employing the 3-components accelerograms of 100 earthquakes, covering a wide range of frequency content form low to high, all normalized to the same Peak Ground Acceleration (PGA) levels of 0.3g, 0.65g, and 1.0g. These PGA levels were utilized to find the effect of earthquake intensity on the behavior of the jacket structure. In all of NLTHA the stress and strain values, particularly plastic strains in critical members, identified by POA, were of the main concern. To decrease the volume of NLTHA output the stress and strain values at only four locations in the section of critical members (say at 0, 90, 180, and 270 degrees in the tubular section) were calculated. The number of locations in the sections of structural members, in which the strain value exceeded the elastic level in each time history, was chosen as the main damage index in NLTHA. Since four locations in each section were contemplated to experience plastic deformation and this could be the case at either end sections of each member the maximum number of locations with plastic deformation could be eight in each member. In some of these locations, the strain value could exceed the rupture level (which was the strain value of 0.0034 according to Von Mises plasticity criterion). Considering that these locations were just in the critical members, identified by POA, and illustrated in Figure 3, number of these locations were acquired for all 100 NLTHA cases with the PGA value of 1.0g, which indicated that just 29 earthquakes were able to create plastic deformation or rupture cases in the jacket critical structural members, as shown in Table 3.

Table 3. The 29 more effective earthquakes and the number of locations with plastic deformation or rupture in the jacket structure obtained from NLTHA.

No.	Earthquake Name	No. of plastic locations (Np)	No. of rupture locations (Nr)	Np + Nr
1	Chi-chi, Taiwan 9	92	54	146
2	Bajestan	77	40	117
3	Chi-chi, Taiwan 4	72	42	119
4	Boshrueh	70	38	108
5	Turkey	68	29	97
6	Erzincan, Turkey	68	28	96
7	Northridge 2	64	38	102
8	Sedeh 2	63	34	97
9	Imperial Valley 2	60	32	92
10	Bandarabbas	60	26	86
11	Imperial Valley	60	26	86
12	Chi-chi, Taiwan 2	60	26	86
13	Khash	57	30	87
14	Rayen	57	27	84
15	Sedeh	57	25	82
16	Birjand	56	14	70
17	Duzce, Turkey	55	21	76
18	Imperial Valley 1	54	14	68
19	Northridge 3	52	23	75
20	Ferdows	52	21	73
21	Gheshm	51	14	65
22	Chi-chi, Taiwan 10	49	19	68
23	Tehran	49	12	61
24	Tehran 23	47	21	68
25	Abaregh	45	7	52
26	Deyhook	43	16	59
27	Chi-chi,Taiwan 3	38	2	40
28	Rudbar	37	8	45
29	Bandar Khamir	34	2	36

Considering the number of plastic locations as the damage criterion, the first 15 earthquakes out of the 29 ones, mentioned in Table 3, can be selected as the most damaging earthquakes for the jacket structure. On this basis, the accelerograms of these earthquakes were scaled once to 0.65g and again to 0.30g for more NLTHA cases. On the other hand, paying attention to the results of NLTHA for various critical members, illustrated in Figure 3, it can be realized that in each case each of these elements experienced some different level of damage. On this basis, a damage percent can be defined for each member depending on the number of plastic locations (0 to 8). Contemplated five levels of damage of: 1) 0%, namely elastic behavior, 2) less than 25%, 3) between 25% and 50%, 4) between 50% and 75%, and 5) more than 75%, some damage probability density functions can be obtained as indicated in Figure 5 for element No. 1 (in the lowest part of the jacket legs – see Figure 3) as the most critical element of the jacket structure.

It is seen in Figure 5 that the damage probability in element No. 1, which has an almost normal distribution for PGA values of 0.3g and 0.65g, increases with growth in the PGA level. Similar graphs can be presented for other members, which are not presented here because of lack of space and can be found in the main report of the study (Karimiyan, 2007, [6]). If the number of rupture cases is considered as the damage indicator, by applying the results presented in Table 3 for rupture cases, and using the five aforementioned states, the rupture probability density functions can be obtained as portrayed in Figure 6 again for element No. 1.

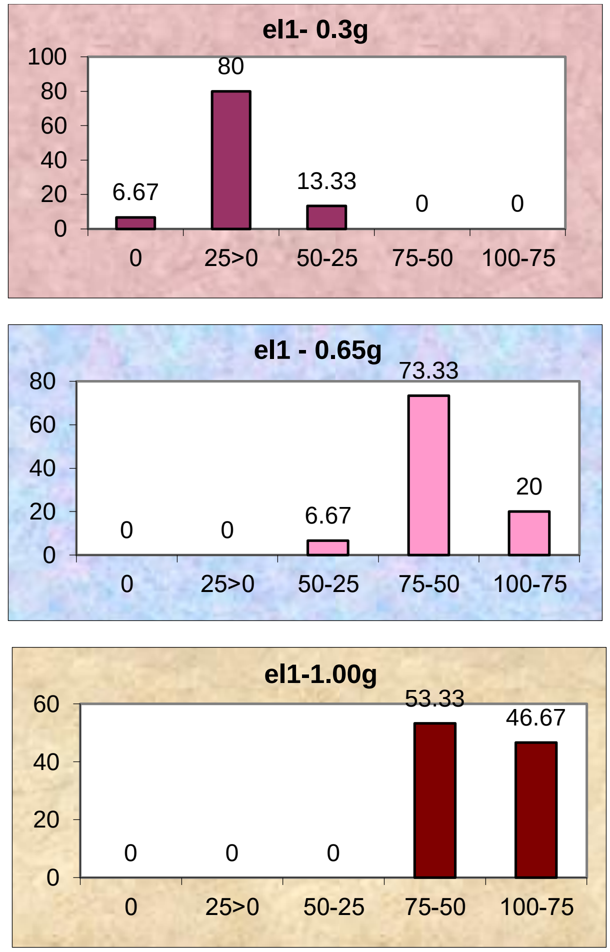

Fig. 5. Damage percent of element No. 1 for various PGA levels in NLTHA

It is seen in Figure 6 that the rupture probability in element No. 1, which its statistical distribution is not far from normal, increases with growth in the PGA level. Again more similar results can be found in the main report of the study (Karimiyan, 2007, [6]).

As the rupture strain for the steel material with fy = 3600 kgf/cm2 is 0.0034, for maximum strains greater than the rupture strain in various PGA levels, probability of failure for each member was calculated. For instance, the probability of failure for various PGA levels in NLTHA in element No. 2 is given in figure 7.

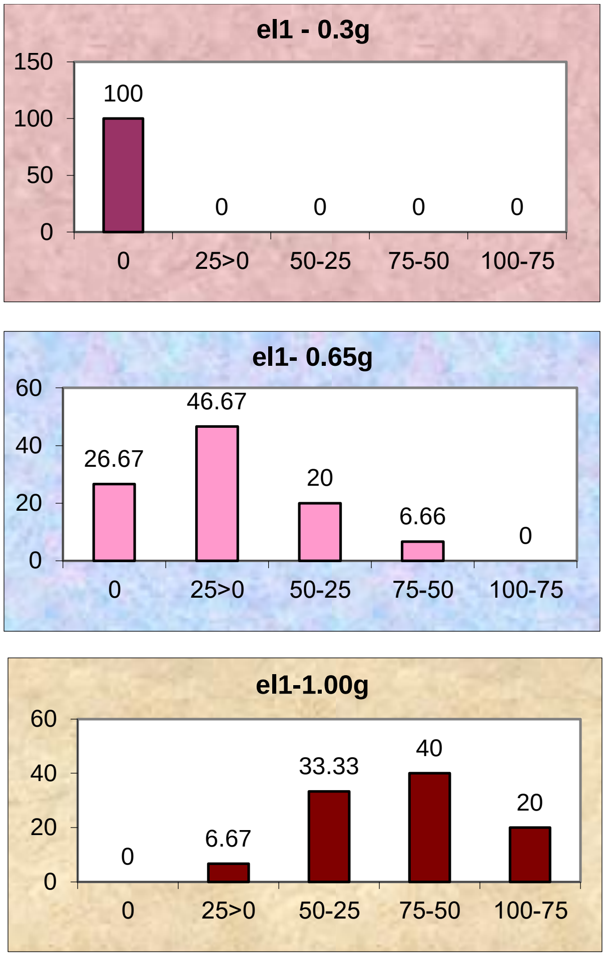

Fig. 6. Rupture percent of element No. 1 for various PGA levels in NLTHA.

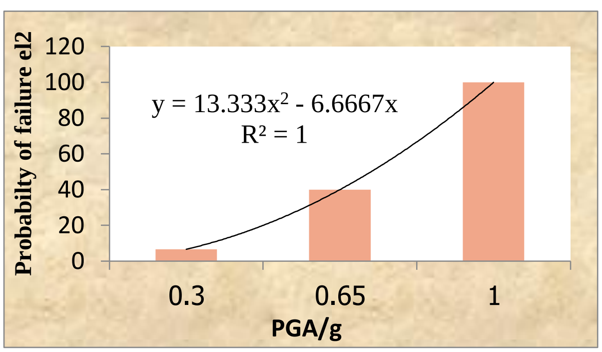

Fig. 7. Probability of failure in element No. 2 for various PGA levels in NLTHA.

Furthermore, for another result, figure 8 illustrates the relation between the spectral response of acceleration and the number of locations in jacket members which experiences plastic and failure deformations.

In this way, 15 critical earthquake records were arranged according to the minimum value of spectral response of acceleration to the maximum value, in three acceleration levels of 0.3g, 0.65g, and 1g, and the plasticization and failure diagrams are depicted in figure 8.

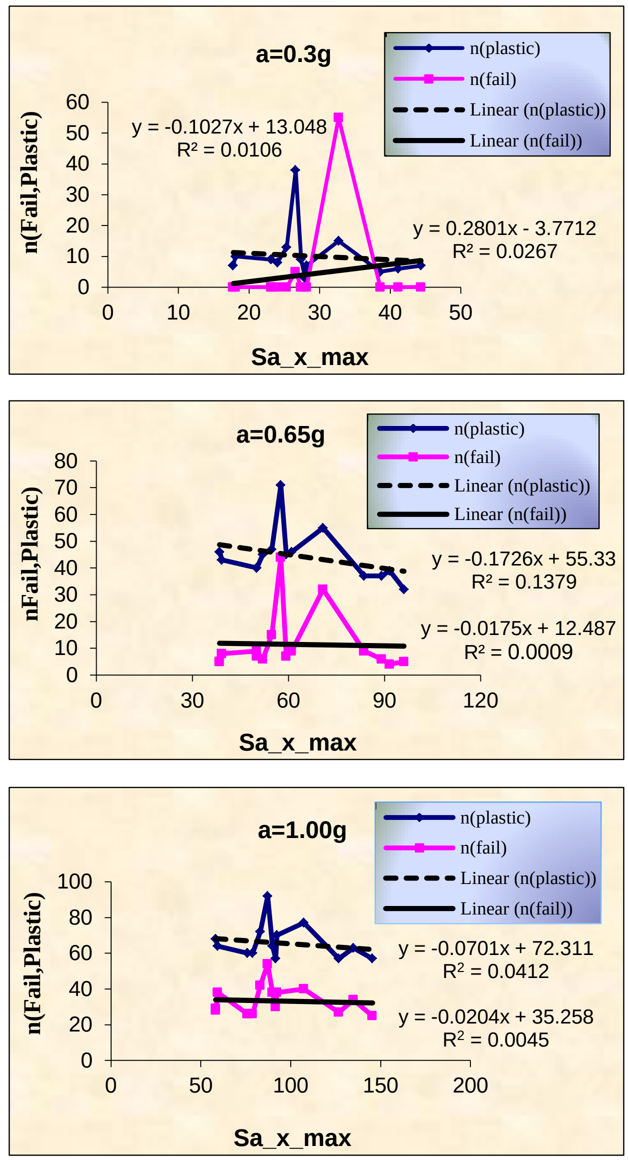

Fig. 8. The relation between the spectral response of acceleration and the number of locations with plastic and failure deformations.

It is seen in Figure 8 that the plasticization and failure behavior of the three accelerations of 0.3g, 0.65g, and 1.00g are similar to each other and independent of the PGA levels.

5. Dominant Earthquake Feature in the Jacket Vulnerability

According to the results of several NLTHA, performed on the jacket structure, it is possible to find out which parameters of ground excitation have more correlation with the vulnerability of the jacket structure. The amount of vulnerability can be stated as the number of locations in structural members which experiences plastic deformations or the number of failure cases in structural members based on the ultimate strain of the structural material or the dissipated energy due to plastic deformations in structural members. The parameters of ground excitation can be any combination of the following ones:

- The energy of the record components (Ex, Ey, Ez) or their summation
- Maximum spectral acceleration values of each component (Sax, Say, Saz) or their summation
- The spectral acceleration values of each component at the fundamental period(s) of the structure (Sa1, Sa2, Sa3, …) or their summation

Among above features, the summation of spectral acceleration responses of the jacket structure in its first three modes (Sa1+Sa2+Sa3) for each component of earthquake excitations indicates the best correlation with the level of damage in the structure. Figure 9 presents the correlation between this factor and the number of locations with plastic deformations in structural members of the jacket, as the damage index, for each component of the ground acceleration.

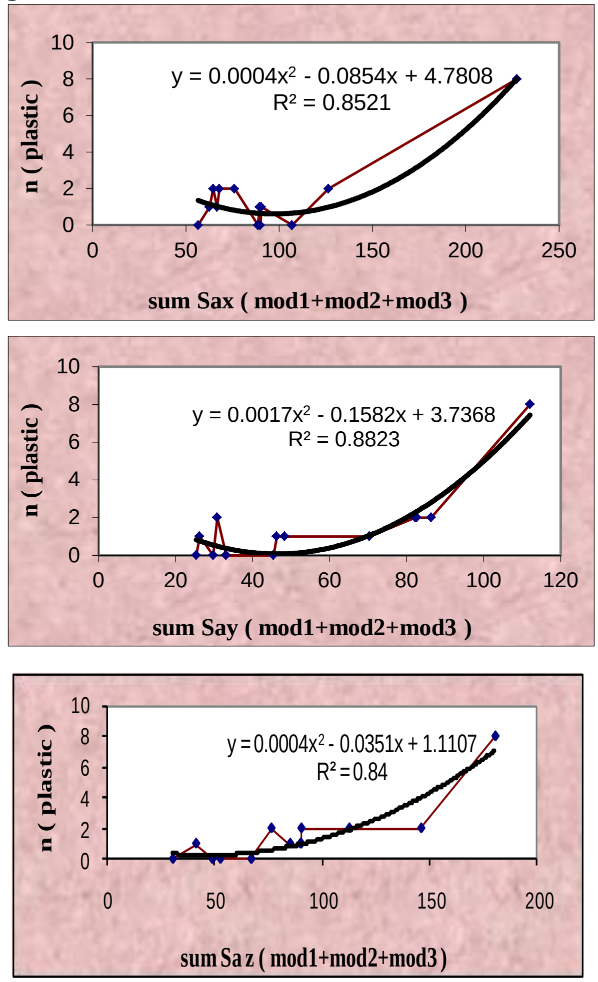

Fig. 9. Correlation between damage index and Sa1+Sa2+Sa3.

On this basis, it can be suggested to apply the summation of modal spectral responses of the structure as the earthquake feature for obtaining the damage of the structure subjected to that earthquake.

6. Conclusions

The main originality issue in this paper is to evaluate the seismic vulnerability of vital offshore structures with the highest possible precision. NLTHA is the most reliable method. Nonetheless, since it is very time consuming, a quick procedure is greatly desired. This paper proposes a quick method by combining the POA and the NLTHA. The POA is preformed first to recognize the more critical members, and then the NLTHA is performed to evaluate more precisely the critical members' vulnerability. The proposed method has been applied to the jacket type structure.

Numerical results indicate that out of the 100 three-component earthquake accelerograms applied in the study, less than 30% could be damaging for the considered jacket structure, even by using a PGA value of 1.0g. This means that, in an overall view, the seismic vulnerability of the jacket structure is relatively low. Notwithstanding, the level of damage is not the same for different members and is dependent on the location of the member in the structure and its geometric orientation and load-bearing situation as well. This implies the application of some important factor for each member based on the three mentioned factors.

As the concluding remarks, it can be said that combining POA and NLTHA is a quick and reliable seismic evaluation method. It can be also claimed that none of the earthquake characteristics alone can be utilized in the vulnerability evaluation as the dominant factor. Instead, a combined factor in which various features of the earthquake, including frequency content, energy, and spectral intensity are taken into account can be suggested. The summation of modal spectral responses of the structure can be a good factor for this purpose.

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