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Assessment of offshore platforms under extreme waves by probabilistic incremental wave analysis

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ABSTRACT

In this study, a novel probabilistic framework named Probabilistic Incremental Wave Analysis (PIWA) is established in order to assess the performance of jacket offshore platforms under extreme waves. The PIWA can take into account the uncertainties in three main elements consisting of sea state parameters, structural response and collapse capacity. The main advantage of the PIWA approach is reflected in decoupling of the wave hazard and structural analyses via an intermediate variable known as the wave height intensity measure. Despite the fact that most of the uncertainties associated with structural response are concentrated in wave hazard, this will enable the PIWA to estimate the probability of failure accurately. Moreover, both static and dynamic wave analyses can be utilized in the PIWA procedure. In this approach, multiple incremental wave analyses are employed to estimate the distribution of structural demand for a wide range of wave height intensities. Subsequently, the mean annual frequency of exceeding a structural limit state is calculated for which this research addresses two different methodologies including demand-based and wave height-based approaches. Furthermore, a new probabilistic-based Reserve Strength Ratio (RSR) is proposed and the probability of exceeding various levels of RSR is provided. To reduce the large number of simulations and hence improving the computational effort in the PIWA procedure, a combination of Latin Hypercube Sampling and Simulated Annealing optimization technique is utilized as an efficient sampling scheme. The PIWA procedure is employed in probabilistic assessment of an existing jacket offshore platform located in the Persian Gulf as well.

1. Introduction

Offshore jacket structures have been utilized in the petroleum industry for decades. From an economical perspective, it is much more preferable to renovate an existing installation compared to constructing a new one. Therefore, the structural safety control of platforms beyond their design life has always been an issue for consideration.

Recently, recommendations for assessment of existing offshore platforms have been proposed [1–7], in which not only target reliability levels and consequences of failure were explicitly addressed but also assessment criteria for wave and seismic loads have been developed. For instance, based on ISO 19902 [7], if one of the platform assessment initiators exist such as a change in the exposure category, inadequate deck height, increased loading on the jacket, and damage found during inspection (for more details see e.g. [3]), the structure shall undergo an assessment. The analysis levels for the aforementioned purposes include linear with component check, linear elastic redundancy, nonlinear (pushover), and finally structural reliability. If the structure is found to be acceptable at any of the analysis levels noted above, no higher levels of analysis are necessary. As a result, it is realized that the highest level of assessment is the structural reliability analysis. Moreover, extreme wave loads are low-probability, large-consequence and large-uncertainty hazards. Thus, the assessment of existing offshore platforms for life-time extension or operative conditions can only be completed in a probabilistic method.

Due to high uncertainties associated with the assessment of offshore platforms, there has been an increasing interest in establishing criteria in the past 20 years which are based on explicit considerations of reliability and risk methodologies. However, the probability of failure is never identified or determined as an integral part of conventional design and assessment practice of offshore structures.

Manuel et al. [8] suggested a procedure by which design level loads are derived from the estimated ultimate level loads based on assumptions regarding the reserve strength characteristics and the target/desired reliability for the jacket. Moreover, the convenient

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analytical formats for reliability computations found in [8–11] are based on the estimation of the probability of failure in terms of the structure's capacity considering its uncertainty as determined by nonlinear static pushover analyses and a probabilistic description of the external wave loads. However, some of these methods have to take into account uncertainty in the structural model in conjunction with the uncertainty in wave loads which imposes a large number of simulations in a probabilistic assessment of jacket platforms.

In addition, nonlinear static pushover analyses employed in these approaches cannot properly estimate the structural behavior at different performance levels as well as the ultimate capacity and collapse behavior of jacket platforms against extreme wave loads. There are three main motivations for this criticism: First, the monotonically increased 100-year design wave load pattern cannot always dictate the real collapse mode because the wave loads causing collapse usually have an annual rate of exceedance much smaller than 10^{-2}. Second, the wave height inducing collapse in the majority of platforms is much higher than the deck height limit, dictated by offshore design codes (e.g. [3,7,12]). When the platform reaches the ultimate capacity, it will experience wave-in-deck loading not taken into account by current pushover analysis. Finally, the pushover capacity curve is not able to reflect the response of platform in different wave hazard levels.

To overcome the aforementioned disadvantages, an innovative approach called Incremental Wave Analysis (IWA) is proposed in the current manuscript (for more details see [13]). The IWA is a nonlinear sequential static or dynamic analysis of a wave-induced jacket platform in order to estimate the structural response for a full range of wave height intensities. In addition, it can estimate the ultimate capacity and the real collapse mode of the platforms accurately. A relatively similar concept called Incremental Dynamic Analysis (IDA) has been established in recent years for seismic assessment of building structures [14].

In the present paper, the evaluation of jacket structural performance can be expressed in terms of the Mean Annual Frequency (MAF) of exceeding a given level of response (or the structural demand parameter), which can be interpreted as the probabilistic applications of the IWA as well. Hence, we have called this novel methodology "Probabilistic Incremental Wave Analysis". In contrast to probabilistic methods found in [9–11], the PIWA simplifies the procedures by decoupling the wave hazard and structural analyses via the intermediate variable known as the wave height intensity measure. The benefit of this approach is that the number of analyses can substantially be reduced because most of the uncertainties in structural demand parameter are concentrated in the wave hazard of the site. This probabilistic framework has conceptual similarity with the recently proposed probabilistic performance-based seismic assessment of building structures [15–18].

A crude Monte Carlo simulation is widely utilized for reliability analysis of jacket platforms (see e.g. [10,11]), which requires a large number of simulations to evaluate more precisely the distribution of ultimate capacity and hence the probability of collapse. To reduce the numerous simulations which require repetitive and time-consuming analyses, this study employs Latin Hypercube Sampling (LHS) [19] in conjunction with the Simulated Annealing (SA) approach [20] as an optimization technique to generate each set of realizations of considered random variables. The main advantage of the SA method is to develop a strategy for the optimal ordering of samples of random variables so that not only the undesired correlation between random variables can be avoided but also the target correlation matrix can be provided.

As a result, the main motivations of this study can be summarized as: First, providing a probabilistic framework (PIWA) for estimation of the MAF of exceeding different levels of the demand parameter (base shear in the current study). Second, estimating the probability of failure as the MAF of exceeding collapse prevention limit state for which two different demand-based and wave height-based methodologies are addressed. Third, elucidating the applications of the PIWA approach through a case study jacket platform located in South Pars Gas Field in the Persian Gulf region. Finally, applying the combination of LHS with the SA optimization technique for the sampling scheme within the PIWA procedure and hence reducing the number of simulations substantially.

2. Necessary elements

A comprehensive discussion on the necessary elements for the PIWA procedure will be presented before introducing this approach. These elements include different sources of uncertainties within wave-induced jacket platforms, the sampling scheme and the IWA concept.

2.1. Source of uncertainties

The uncertainties in a probabilistic evaluation of jacket structures can be broken into three main categories including variability in sea state parameters and inherent randomness in the wave process, uncertainties in the prediction of wave force on the jacket's structure and finally uncertainties in structural model [11]. The first category can be estimated directly from the probabilistic wave hazard analysis of the site. The second category, which contains the main parameters influencing the wave force on the jacket's structure, can be summarized as drag (C_d) and inertia (C_m) coefficients as well as marine growth (MG). The uncertainties in the structural model account for the variability in the element and system levels of the jacket's structure for a given design realization. In the current study, the structural parameters which are considered to be random (i.e. element-level variables) are yield stress of jacket legs, $f_{y,L}$, the yield stress of jacket horizontal and diagonal braces, $f_{y,B}$, and the modulus of elasticity, E_s. In addition, vertical loads and masses are considered as system-level variables. However, the yield stress of steel material was only considered in previous researches [10,11]. It is noteworthy that separating the yield stress of legs from braces in current study will cause the appearance of different collapse modes.

It should further be noted that the structural response of different jacket platforms in the Persian Gulf region which are subjected to regular and irregular waves reveals that the dynamic analyses utilizing regular waves not only can diminish the complexities within irregular wave analyses but also can provide a conservative estimate of demands (for more details see [13]). Therefore in this study, the jacket's structure is subjected to regular waves from Stokes' 5th order wave theory. In addition, the wave period is assumed to be constant within each level of wave height intensity. According to the provisions by DNV [12], the most probable individual wave period to be employed with a long term extreme wave height can be expressed as

$$T_{H_{\max}} = a \cdot H_{\max}^{b} \tag{1}$$

where $a = 2.94$ and $b = 0.5$ are empirical coefficients proposed by DNV. In the current study, the same coefficients are utilized as they are close to the observations in the Persian Gulf region.

Although considerable uncertainties exist within pile–soil interaction parameters, this research does not address them because of two major reasons. First of all, this article focuses primarily on the establishment of the PIWA procedure not inclusion of all existing sources of uncertainties. Moreover, the uncertainties within pile–soil interactions are not apparent yet as another source of uncertainties.

Table 1
The statistical characteristics of selected random variables.

Random variable	Symbol	Mean/median	COV	Type	Reference
Parameters influencing variability of the wave force on jacket structure					
Drag coefficient	C_d	0.65, 1.10	0.25	Lognormal	JCSS [21], Skallerud [11]
Inertia coefficient	C_m	1.60, 1.27	0.10	Lognormal	JCSS [21], Skallerud [11]
Marine growth	MG	75 mm, 50 mm	0.50	Lognormal	JCSS [21], Skallerud [11]
Parameters influencing uncertainties in structural model					
Loads and masses	m, W	Computed	0.10	Normal	Ellingwood et al. [22]
Yield stress of legs	$f_{y,L}$	335 MPa, 345 MPa	0.07	Lognormal	JCSS [21]
Yield stress of braces	$f_{y,B}$	335 MPa, 345 MPa	0.07	Lognormal	JCSS [21]
Modulus of elasticity	E_s	2.0601×10^5 MPa	0.03	Lognormal	JCSS [21]

As a result, seven random variables ($N_{\text{var}} = 7$) are taken into account and their statistical characteristics are summarized in Table 1. The mean or median values of random variables (based on the type of distributions) correspond to the best estimates employed in the deterministic model. Two different values for C_d, C_m, and MG indicate the above and below of the splash zone, respectively. In addition, two different values for yield stress of steel material are illustrated based on the thickness of jacket members, which are in accordance with the design specifications of the case study jacket platform.

Generally, the correlations between random variables are of two basic types, (a) Correlations between parameters of a given structural component and (b) Correlations between parameters of different structural components [23]. In this study, no correlation is considered within an element; whereas the parameters influencing the wave force on the jacket platform as well as structural model random variables are perfectly correlated for different components [21]. This assumption was made to reduce the number of random variables, and thus the computational effort.

2.2. Sampling scheme

A variety of techniques have been proposed to address the effect of different sources of uncertainties on the probabilistic estimation of structural response. These approaches range from simplified first-order second-moment (FOSM) reliability method to more general Monte Carlo type simulation and response surface techniques as well. The studies carried out by several researchers explored the effect of modeling uncertainty with the FOSM method [23–28]. In the last study, it was investigated that the FOSM reliability method can become inaccurate for highly nonlinear systems. The Monte Carlo method [29] can be employed as an alternative approach, in which different sampling techniques can be utilized to generate the realizations of random variables. Pure Monte Carlo simulations based on a random sampling approach cannot be applied to computationally time-consuming problems; since it requires a large number of simulations and repetitive calculations of structural response.

The Latin Hypercube Sampling technique (LHS) [19] can be employed in order to reduce the number of simulations, N_{sim}, in addition to gain an acceptable level of accuracy for the statistical characteristics of response. The LHS is a special type of Monte Carlo simulation which uses the stratification of the theoretical probability distribution functions of input random variables. Its roles in different aspects of reliability engineering have been described in [30,31]. Stein [32] has shown that the LHS reduces the variance of the response function compared to the crude Monte Carlo method. In spite of the high efficiency of LHS technique, there are generally two issues concerning statistical correlation [20]: First, diminishing undesired and spurious correlation between random variables generated during sampling procedure, particularly in the case of a very small number of simulations (tens); second, introducing the prescribed statistical correlations between pairs of random variables defined by the target correlation matrix $\boldsymbol{T}$. Hence, in order to impose a prescribed correlation matrix into the sampling scheme, an optimization problem for minimizing the difference between the target correlation matrix $\boldsymbol{T}$ and the actual correlation matrix $\boldsymbol{A}$ (estimated from samples) should be solved. To have an opportunity to escape from local minima and finally find the global minimum, a novel stochastic optimization approach called Simulated Annealing (SA) has been recently proposed [20]. In the present paper, this optimization technique is employed in the sampling process in order to generate the appropriate set of N_{sim} structural models.

To conduct a sampling procedure according to LHS with the SA technique, a computer code has been prepared. Based on the above optimization scheme, 15 simulations ($N_{\text{sim}} = 15$) can produce the target correlation with appropriate accuracy for 7 random variables ($N_{\text{var}} = 7$) indicated in Table 1. In order to demonstrate the efficiency of this algorithm, the target correlation matrix $\boldsymbol{T}$ is presented in the lower triangle of the following matrix; while the upper triangle illustrates the generated (actual) correlation matrix $\boldsymbol{A}$ after application of the SA algorithm:

$$\triangleright\boldsymbol{T},\ \triangleleft\boldsymbol{A} = \begin{pmatrix} C_d & C_m & MG & f_{y,L} & f_{y,B} & E_s & Mass \\ \mathbf{1.0} & 0.011 & 0.0027 & -0.0021 & -0.005 & 0.0041 & -0.0081 \\ 0 & \mathbf{1.0} & 0.0009 & -0.0111 & -0.0006 & 0.004 & 0.0111 \\ 0 & 0 & \mathbf{1.0} & 0.0015 & 0.0073 & -0.0117 & -0.0052 \\ 0 & 0 & 0 & \mathbf{1.0} & -0.0101 & -0.0021 & 0.008 \\ 0 & 0 & 0 & 0 & \mathbf{1.0} & 0.0052 & 0.0025 \\ 0 & 0 & 0 & 0 & 0 & \mathbf{1.0} & -0.0084 \\ 0 & 0 & 0 & 0 & 0 & 0 & \mathbf{1.0} \end{pmatrix} \begin{matrix} C_d \\ C_m \\ MG \\ f_{y,L} \\ f_{y,B} \\ E_s \\ Mass \end{matrix}.$$

It is noteworthy that the LHS in conjunction with the SA technique leads to appropriate results; while the number of simulations is substantially small in comparison with the crude Monte Carlo method.

2.3. Incremental wave analysis (IWA)

This section aims to briefly clarify the IWA introduced primarily by the authors [13]. This novel approach, which has the ability to evaluate the platform's behavior in the most accurate method, is one of the essential parts of the proposed PIWA algorithm.

To conduct Static or Dynamic IWA (SIWA or DIWA), the structural model should be subjected individually to incremental wave heights. For each individual wave height, nonlinear static or dynamic analysis is carried out and the structural demand parameters (base shear, overturning moment, displacements, drifts and other displacement-based or strength-based structural responses) are obtained accordingly. By performing this procedure, there should be finally a particular wave height at which the platform cannot undergo the wave loading, and the incremental

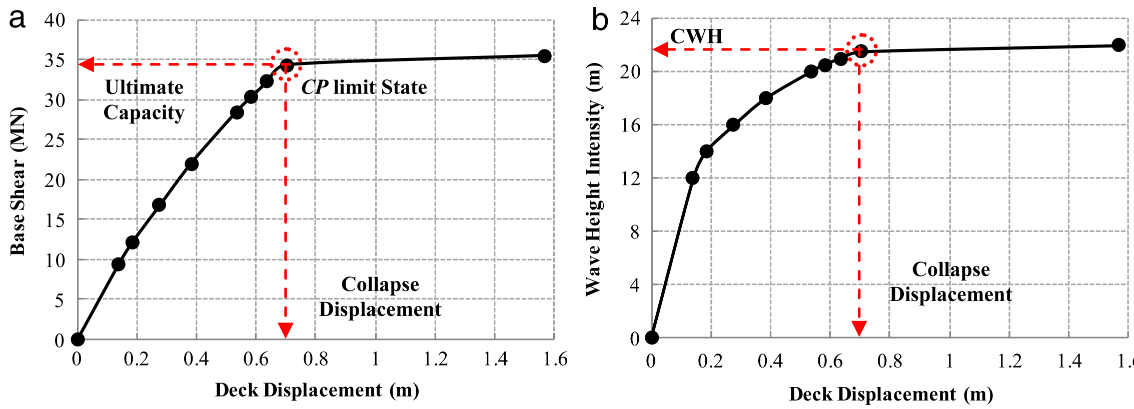

Fig. 1. Different illustrations of the IWA curve, (a) base shear vs. global deck displacement and (b) wave height vs. global deck displacement, together with indicators for CP limit state.

analysis should be terminated. Plotting the base shear versus deck displacement extracted from each individual analysis will generate a specific curve reflecting the required structural response (demand) at different wave intensity levels as expressed in Fig. 1(a). Consequently, the point on the cited curve where a sudden change in the local slope occurs (approximately less than 15% of the initial slope) will be defined as the Collapse Prevention (CP) limit state of the platform. This characteristic of the CP limit state can be observed in the generated curve representing wave height versus deck displacement (Fig. 1(b)) as well.

Generally, it should further be noted that an instantaneous increase in displacement is detected after the CP limit state initiation, i.e. the displacement increases to an arbitrarily large value for an arbitrarily small increase in wave height intensity. Thus, the displacement response will become too large to be meaningful for high values of wave height intensity owing to the threshold of an overall lack of stability in the jacket's structure, which can be interpreted as another indicator ensuring the CP limit state. This concept is clearly illustrated in Fig. 1. With reference to this figure, the base shear and wave height corresponding to the CP limit state can be expressed as ultimate capacity and Collapse Wave Height (CWH) indicators of the platform, respectively.

3. Case study: SPD2 platform

In order to clarify the PIWA procedure, the SPD2 jacket platform located in South Pars Gas Field in Persian Gulf region is employed. A general configuration of modeled platform is illustrated in Fig. 2. The SPD2 jacket located in 65 m water depth consists of six legs and three battered faces. The jacket plan dimension is about 16.00 m × 27.50 m at topside elevation and 23.4 m × 37.7 m at the mud line. The structure is fixed to the ground by 6 through-leg grouted piles. Therefore, a nonlinear pile–soil-structure interaction is considered in the 3-D model of the SPD2 platform. A simplified elastic model of the topside including main frames is employed. All topside loads are applied on the main joints as equivalent point loads. The finite element program USFOS [33], which has the capability to perform nonlinear static and dynamic analyses of wave-induced jacket platforms, is utilized. The effect of joint flexibility and strength in tubular connections is considered in analysis of the SPD2 jacket platform as well. This is important particularly for old structures where joint cans were not used. However, the SPD2 jacket can be categorized as a newly installed platform for which no joint failure has been occurred during different nonlinear wave analyses conducted in current research. Thus, the uncertainties in the joint strength and flexibility parameters are not considered in this study.

Since the same design specifications and physical configuration conform to the offshore platforms in South Pars Gas Field, the results of the proposed probabilistic assessment procedure in the subsequent sections are valid for jacket structures in this area of the Persian Gulf.

Fig. 2. Global view of the SPD2 platform, the jacket structure with piles.

4. Probabilistic incremental wave analysis (PIWA)

The PIWA is a robust methodology which can assess the performance of jacket offshore platforms by probabilistically predicting the structural response under extreme wave loads. In this method, the structural response is quantified via a proper Demand Parameter (DP), i.e. base shear in this study. Generally, the PIWA procedure can be summarized in the following three main steps.

4.1. Multiple-stripe analysis for structural demand parameter

In order to estimate the true distribution of the structural DP over a wide range of wave height intensities, Multiple-Stripe

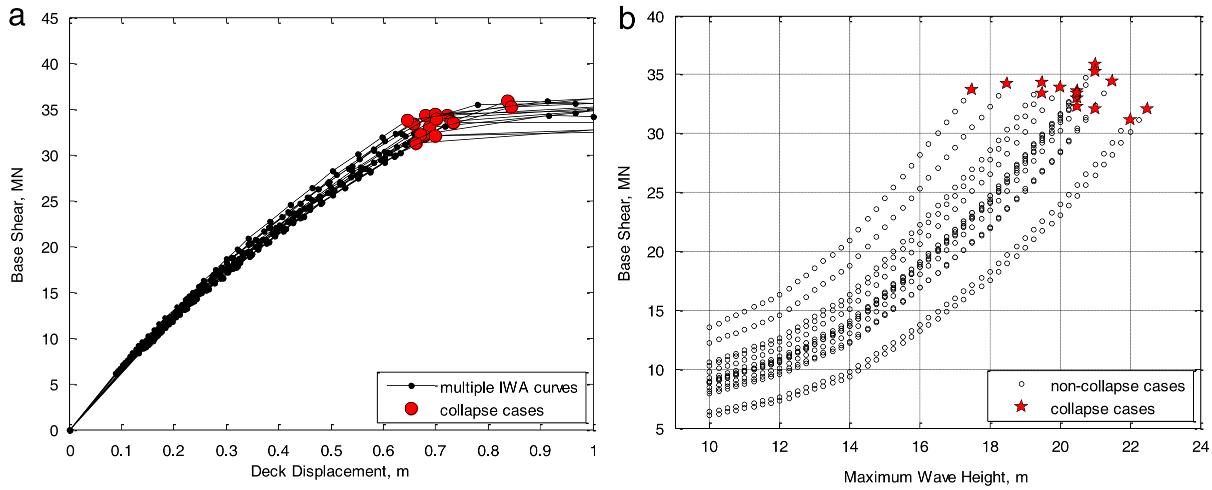

Fig. 3. (a) Multiple static IWA (SIWA) curves and (b) associated Multiple-Stripe Analysis (MSA) data of the base shear response with collapse cases marked on them.

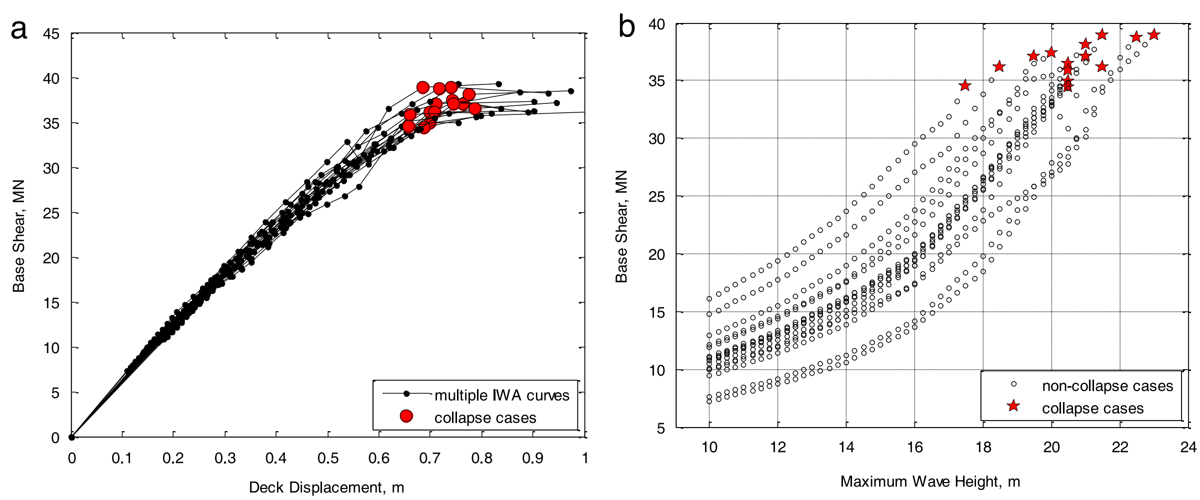

Fig. 4. (a) Multiple dynamic IWA (DIWA) curves and (b) associated Multiple-Stripe Analysis (MSA) data of the base shear response with collapse cases marked on them.

Analysis (MSA) [17,18] should be carried out. The MSA, which is the first step in the PIWA procedure, is in essence the re-compilation of the results of the multiple IWA for the set of N_{sim} structural models at multiple levels of wave height intensity.

The multiple static and dynamic IWA in terms of base shear versus global deck displacement for the case study SPD2 platform utilizing a set of 15 structural models ($N_{sim} = 15$) are presented in Figs. 3(a) and 4(a). Consequently, Figs. 3(b) and 4(b) represent the MSA data based on static and dynamic incremental wave analyses. The points demonstrating the CP limit state to be the threshold of extreme displacement responses or global softening of the IWA curves are marked on Figs. 3 and 4 as the "collapse cases". As displayed in Figs. 3(b) and 4(b), each stripe may contain more than one collapse case.

4.2. Mean annual frequency of exceeding a specific DP level (DP hazard)

One of the major parts of the PIWA procedure is the estimation of the DP hazard curve, λ_{DP}, representing the MAF of exceeding a specified level of response. The DP hazard provides a direct measure for the performance of structures because it relates to the annual frequency of experiencing the event $DP > x$. Utilizing the total probability theorem [34], evaluation of the MAF of exceedance can significantly be simplified by decoupling the wave hazard and multiple-stripe analyses by means of an intermediate variable known as the wave height intensity measure, H_{max}. Thus, λ_{DP} can be expanded with respect to all possible values of wave height intensity, h:

$$\lambda_{DP}(x) = \nu \cdot P[DP > x] = \sum_{all\ h} P[DP > x \mid H_{max} = h] \cdot (\nu \cdot P[H_{max} = h]) \quad (2)$$

where $P[DP > x \mid H_{max} = h]$ is the conditional probability of exceeding a specified DP value, x, for a given level of wave height intensity, h, which can directly be estimated by the distribution of DP via the MSA. Moreover, the expression $P[H_{max} = h]$ is the likelihood that the wave height intensity will equal a specified value, h, which is extracted directly from the probabilistic wave height hazard analysis of the site. In addition, ν represents the annual rate of occurrence of the events (storms) or the number of 3-h sea states in one year, which is equal to $365.25 \times 8 = 2922$ [12]. To generate the expression for the DP hazard derived for continuous variables, Eq. (2) can be re-written in the following form:

$$\lambda_{DP}(x) = \int_h P[DP > x \mid H_{max} = h] \cdot \left| \frac{d\lambda_{H_{max}}}{dh} \right| dh = \int_h G_{DP|H_{max}}(x \mid h) \cdot |d\lambda_{H_{max}}(h)| \quad (3)$$

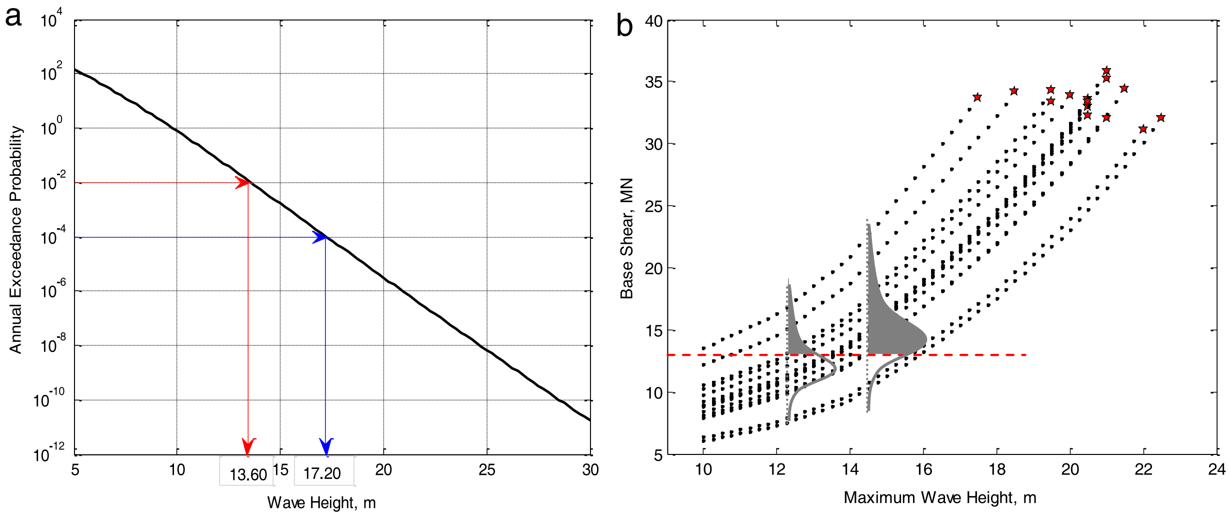

Fig. 5. (a) Mean annual frequency of exceeding maximum wave height (wave height hazard curve) and (b) lognormal distribution of the base shear data in multiple stripes.

where $G_{DP|H_{\max}}(x \mid h)$ is referred to as the conditional Complementary Cumulative Density Function (CCDF) of the DP for a given wave height level, h, and $\lambda_{H_{\max}}(h)$ illustrates the site-specific wave height hazard in terms of the MAF of exceeding the wave height intensity level, h. Fig. 5(a) displays $\lambda_{H_{\max}}$ for the SPD2 jacket platform's site. Additionally, $\mathrm{d}\lambda_{H_{\max}}$ represents the differential of the wave height hazard curve.

Scattered along multiple stripes of constant wave height (Figs. 3(b) and 4(b)), the distribution of the DP for a given wave height intensity can be estimated empirically. Moreover, lognormal distribution can be properly fitted to the DP data within individual stripes as illustrated in Fig. 5(b). The advantage of estimating $\lambda_{DP}(x)$ in Eq. (3) is that the number of needed analyses can be substantially reduced because most of the uncertainties in the DP are concentrated in the wave height hazard of the site. With regards to Figs. 3(b) and 4(b), individual stripes may enclose collapse cases, together with non-collapse data. The expression for $G_{DP|H_{\max}}(x \mid h)$ can simultaneously be expanded based on collapse (C) and non-collapse (NC) cases by using the total probability theorem (in the case of seismic assessment see [35]):

$$G_{DP|H_{\max}}(x \mid h) = G_{DP|H_{\max},NC}(x \mid h) \cdot P_{NC|H_{\max}}(h) + G_{DP|H_{\max},C}(x \mid h) \cdot (1 - P_{NC|H_{\max}}(h)) \quad (4)$$

where $P_{NC|H_{\max}}(h) = 1 - P_{C|H_{\max}}(h)$ is the conditional probability of observing non-collapse cases given a wave height intensity level, h, and $G_{DP|H_{\max},NC}(x \mid h)$ is the conditional CCDF of DP, x, given that no collapse case has occurred. The expression for $P_{NC|H_{\max}}(h)$ can be empirically estimated by the ratio of non-collapse cases to the total number of simulated response points on the stripe, N_{sim}. It is noteworthy that the collapse points obtained in this study are only serving as a proxy for global instability. Thus, reaching the threshold of the CP limit state does not necessarily mean that the jacket structure has collapsed.

Regarding Eq. (4), the conditional probability of exceeding any DP given collapse, $G_{DP|H_{\max},C}(\cdot)$, is assumed to be equal to 1. Therefore, Eq. (4) can be re-written as:

$$G_{DP|H_{\max}}(x \mid h) = G_{DP|H_{\max},NC}(x \mid h) \cdot P_{NC|H_{\max}}(h) + (1 - P_{NC|H_{\max}}(h)). \quad (5)$$

Substituting Eq. (5) into Eq. (3), the alternative expression for the DP hazard of jacket structures in the region from elastic response to the CP limit state becomes:

$$\lambda_{DP}(x) = \int_h [G_{DP|H_{\max},NC}(x \mid h) \cdot P_{NC|H_{\max}}(h) + 1 - P_{NC|H_{\max}}(h)] \cdot |\mathrm{d}\lambda_{H_{\max}}(h)|. \quad (6)$$

In the limiting case, for large DP values, x, $G_{DP|H_{\max},NC}(x \mid h) \cong 0$ for all wave height intensity values, and the DP hazard (Eq. (6)) becomes:

$$\lambda_{DP}(x) = \int_h (1 - P_{NC|H_{\max}}(h)) \cdot |\mathrm{d}\lambda_{H_{\max}}(h)| = \int_h P_{C|H_{\max}}(h) \cdot |\mathrm{d}\lambda_{H_{\max}}(h)|. \quad (7)$$

As illustrated in Eq. (7), for large demand values, the expression for the DP hazard is independent of the value x and will become constant in the limiting case.

Furthermore, the failure probability for t-year design life, P_f^t, can be evaluated directly from the Poisson distribution as

$$P_f^t = 1 - \exp(-\lambda_{DP} t) = 1 - \exp\left(-\frac{t}{T_R}\right) \quad (8)$$

where T_R is the return period which is the reciprocal of λ_{DP}.Whereas the base shear has been chosen as the DP in assessing jacket structures against extreme wave loads by current offshore standards [3,6,7]; further the base shear hazard curve resulting from Eq. (6) is illustrated in Fig. 6. Both multiple SIWA and DIWA procedures are employed in estimation of the base shear hazard curve. Moreover, the expression for $G_{DP|H_{\max},NC}(x \mid h)$ within Eq. (6) is estimated by lognormal and empirical distributions. The two curves illustrated in Fig. 6 are very close to imply that for the case study jacket platform, the non-collapse data of the base shear response can be properly modeled by a lognormal distribution.

One can utilize this hazard curve to determine the base shear resulting from the extreme environmental condition corresponding to the hazard level in the proximity of an allowable annual probability. The base shear responses associated with 100-year and 10,000-year return periods are summarized in Table 2. The results indicate that for a given hazard level, the dynamic wave analyses result in higher demands compared to the static wave analyses.

4.3. *The CP limit state frequency by the ultimate capacity-based approach*

In this section, the MAF of exceeding the CP limit state based on the ultimate capacity will be presented. The probability of failure,

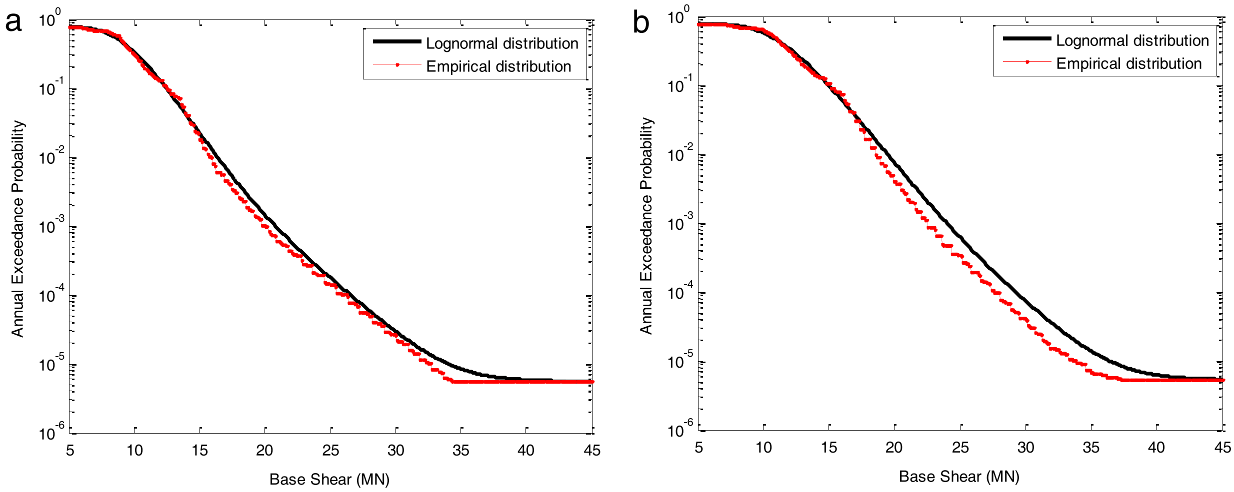

Fig. 6. Base shear hazard curve estimated by employing (a) SIWA and (b) DIWA.

Table 2
The base shear responses associated with two extreme environmental conditions.

Return period (year)	SIWA (MN)	DIWA (MN)
100	16.4	19.5
10,000	26.6	29.3

Table 3
The CP limit state frequency obtained from the ultimate capacity-based approach.

Distribution of fragility function	SIWA	DIWA
Empirical	6.1323×10^{-6}	4.5535×10^{-6}
Lognormal	6.2027×10^{-6}	4.6039×10^{-6}

P_f, or the limit state probability, P_{LS}, is composed of all possible combination of $DP = x$ and $C < x$, in which C denotes the ultimate capacity random variable. Utilizing the total probability theorem, the expression for the limit state probability can be written as [36]:

$$P_f = P_{LS} = \sum_{all\ x} P(DP = x \cap C < x)$$
$$= \sum_{all\ x} P[C < DP \mid DP = x] \cdot P[DP = x]. \quad (9)$$

In order to determine the MAF of exceeding the CP limit state derived for continuous variables, Eq. (9) can be re-written as follows:

$$\lambda_{LS} = \int_x F_C(x) \cdot |d\lambda_{DP}(x)| \quad (10)$$

where λ_{LS} is the MAF of exceeding the CP limit state (limit state frequency), F_C is the conditional CDF of the ultimate capacity for the CP limit state known as the fragility function, and finally $d\lambda_{DP}$ is the differential of the DP hazard estimated by Eq. (6). The scattering of the ultimate capacity data are illustrated in Figs. 3 and 4 for the SPD2 jacket. In addition, Fig. 7 shows the fragility curve, F_C, by applying both empirical and lognormal distributions. The median, $\hat{\eta}_C$, and the COV of the ultimate capacity estimated by the lognormal distribution are indicated in Fig. 7. The resulted COV in the current study is close to the lower limit recommended by DNV [4] for the base shear capacity of jacket structures to be 0.05–0.10.

Based on the expression in Eq. (10), CP limit state frequencies are demonstrated in Table 3 for which λ_{DP} is estimated by a lognormal distribution of the base shear responses. As a result, the limit state frequencies are strictly identical regarding both distributions, implying that the lognormal distribution is an appropriate model for the fragility function. In addition, the CP limit state frequencies resulted from static incremental wave analyses are higher than those obtained from dynamic incremental wave analyses.

4.4. The CP limit state frequency by the CWH-based approach

The MAF of exceeding the CP limit state can also be estimated by following a CWH-based approach, i.e. collapse occurs when the wave height intensity exceeds the collapse wave height variable, *CWH*. Therefore, the CWH-based limit state probability can be represented as

$$P_{LS} = \sum_{all\ h} P(H_{max} = h \cap CWH < h)$$
$$= \sum_{all\ h} P[CWH < H_{max} \mid H_{max} = h] \cdot P[H_{max} = h]. \quad (11)$$

Eq. (11) can be re-written in the following continuous form for estimation of the CP limit state frequency by the CWH-based approach:

$$\lambda_{LS} = \int_h F_{CWH}(h) \cdot |d\lambda_{H_{max}}| \quad (12)$$

where F_{CWH} is the conditional CDF of the collapse wave height known as the CWH fragility function. Fig. 8 illustrates the CWH fragility function for the case study jacket platform. The median, $\hat{\eta}_{CWH}$, and the COV of the CWH expressed in Fig. 8 indicate that $\hat{\eta}_{CWH}$ is less sensitive to the static and dynamic wave analyses in comparison with $\hat{\eta}_C$. Furthermore, there is a higher level of dispersion in the CP limit state data based on the CWH indicator.

In addition, the conditional probability of collapse for a given wave height intensity level, h, denoted by $P_{C|H_{max}}(h)$ is expressed in Fig. 8, which implies that it is identical to the empirical distribution of the CWH fragility function. To verify this issue, $P_{C|H_{max}}(h)$ can be written as:

$$P_{C|H_{max}}(h) = P[\text{collapse}|H_{max} = h]$$
$$= P[CWH < H_{max}|H_{max} = h] \triangleq F_{CWH}(h). \quad (13)$$

Alternatively, it can be concluded from Eqs. (7) and (13) that for large DP values, the base shear hazard is asymptotically equal to the CP limit state frequency, λ_{LS} obtained from the CWH-based approach:

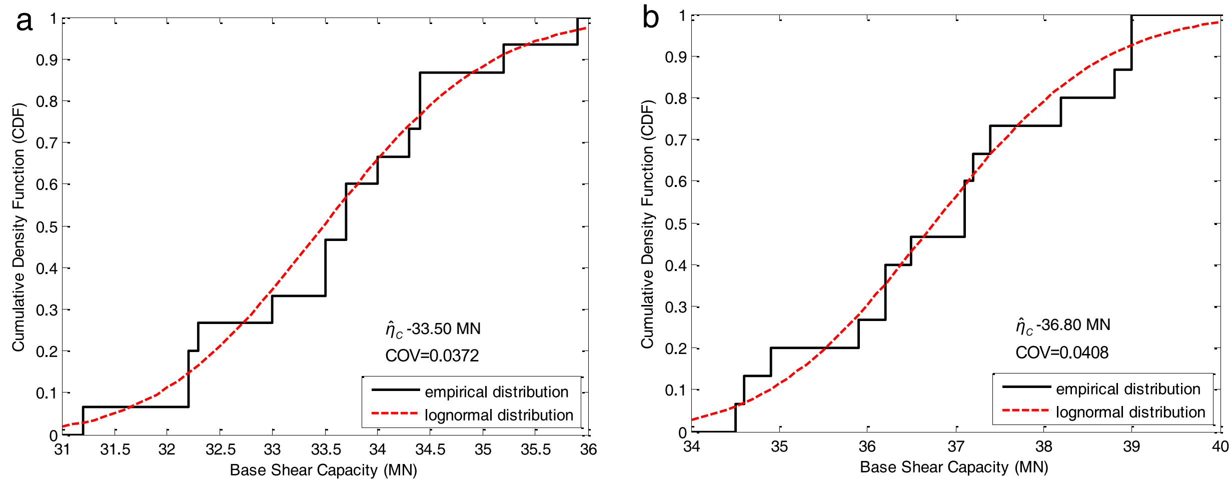

Fig. 7. Comparison of the fragility curves for the ultimate capacity by applying empirical and lognormal distributions based on (a) SIWA and (b) DIWA.

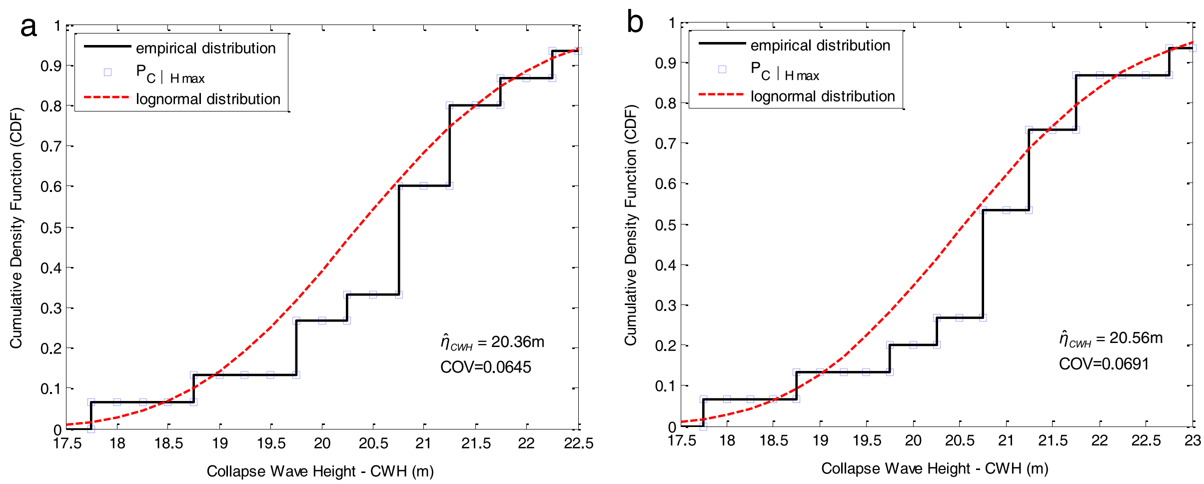

Fig. 8. Comparison of the CWH fragility curves by applying empirical and lognormal distributions based on (a) SIWA and (b) DIWA.

Table 4
The CP limit state frequency obtained from the CWH-based approach.

	SIWA	DIWA
Distribution of CWH fragility function		
Empirical	5.5724×10^{-6}	5.3513×10^{-6}
Lognormal	5.4199×10^{-6}	5.1415×10^{-6}
Lower limiting value of base shear hazard curve		
Empirical	5.5724×10^{-6}	5.3513×10^{-6}
Lognormal	5.5959×10^{-6}	5.4324×10^{-6}

$$\lambda_{DP}(x) = \int_h P_{C|H_{\max}}(h) \cdot |d\lambda_{H_{\max}}(h)| = \int_h F_{CWH}(h) \cdot |d\lambda_{H_{\max}}| \triangleq \lambda_{LS}. \tag{14}$$

Therefore, the base shear hazard curve not only provides an estimate for the MAF of exceeding various levels of base shear response, but also represents directly the MAF of the CP limit state. This fact has been verified through the case study jacket platform. The CWH-based λ_{LS} as well as the lower limit of the DP hazard curve summarized in Table 4 are identical. From Table 4, it can further be concluded that the lognormal distribution is an appropriate model for the CWH fragility function. Moreover, the results dictate that λ_{LS} attained by the CWH-based approach leads to close limit state frequencies regarding static and dynamic wave analyses for the case study structure. As a result, the CWH-based approach has great superiority over the ultimate capacity-based method for estimation of λ_{LS}; while both definitions require the CP limit state fragility function, the latter expression necessitates the evaluation of the DP hazard curve which requires more computational effort. Furthermore, $\hat{\eta}_{CWH}$ as well as the CWH-based λ_{LS} have great consistency regarding different type of analyses.

Hence, as a general rule, there is no need to perform dynamic IWA for estimation of the limit state frequency in the assessment of jacket platforms located in the South Pars Gas Field in the Persian Gulf region.

5. The MAF of exceeding a specific level of RSR (RSR hazard)

The Reserve Strength Ratio (RSR) is a force-based indicator utilized extensively in the assessment of existing jacket offshore platforms. The RSR is estimated in terms of the gross base shear at collapse, V_C, relative to the design base shear, V_D [3]. The design-level base shear usually contains the dead and a fraction of live loads in addition to the 100-year environmental wave load. The deterministic-based RSR can be estimated directly from the current pushover analysis as well as the SIWA, for which both curves are demonstrated in Fig. 9 for the case study jacket platform. The RSR associated with current pushover analysis and SIWA in the N–S direction of the SPD2 platform are estimated to be equal to 2.64 and 2.43, respectively. It is noteworthy that

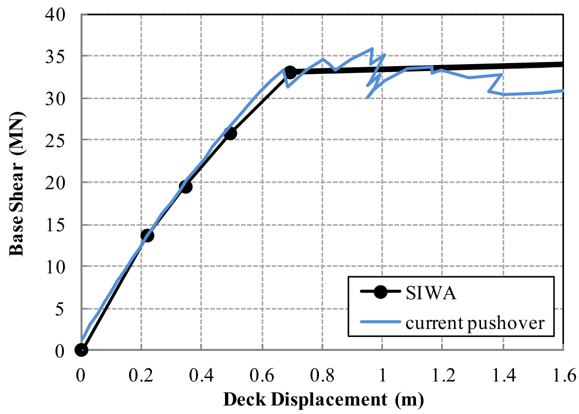

Fig. 9. The SIWA and the current pushover curves in the N–S direction of the SPD2 platform.

the difference between the RSR resulting from both analyses can be more apparent in the case of other jacket offshore platforms (see [13]).

The PIWA framework has the ability to represent a probabilistic definition for the RSR as well as the evaluation of the MAF of exceeding various levels of RSR. In order to define a probabilistic-based RSR denoted by PRSR, V_C should be identified as the base shear corresponding to an annual frequency equal to λ_{LS}. However, the expression for V_C derived from the base shear hazard curve considering the collapse cases (Fig. 6) lies at infinity as explained previously. To locate a finite value for V_C on the hazard curve, the base shear hazard curve is evaluated without taking into account the collapse cases in the numerical integration within Eq. (6). Fig. 10 demonstrates the base shear hazard curve given that no collapse cases are considered. The limit state frequency obtained from the CWH-based approach (Table 4) is indicated by the straight line in this figure. Thus, the base shear capacities, V_C, are estimated to be 33.70 and 36.40 MN based on static and dynamic incremental wave analyses, respectively. Furthermore, in PRSR, V_D is defined as the base shear associated with the 100-year return period, which is outlined in Table 2. Therefore, the PRSR are estimated to be 2.06 and 1.87 according to static and dynamic IWA, respectively, which are smaller than those obtained from a deterministic-based approach. Hence, the current deterministic-based RSR widely utilized in ultimate strength analysis of jacket platforms can lead to unreliable estimations. Moreover, the PRSR evaluated by dynamic wave analyses is more conservative.

In addition, the MAF of exceeding a specified level of RSR (RSR hazard) can further be estimated by the PIWA procedure, where the RSR corresponding to the MAF of exceedance equal to P_o, RSR^{P_o}, is expressed as:

$$RSR^{P_o} = \frac{V^{P_o}}{V_D} \tag{15}$$

where V^{P_o} is the base shear associated with a MAF of being exceeded equal to P_o, given the fact that collapse cases are not considered, and V_D is the same as that in the PRSR. The RSR hazard curves are expressed in Fig. 11 along with the PRSR values marked on them. The RSR associated with the 10,000-year return period from Fig. 11 are 1.62 and 1.50 based on static and dynamic IWA, respectively. These results are in accordance with the provisions of API [3]; the SPD2 jacket is categorized as an unmanned and high consequence of failure platform (level L-1) for which RSR>1.60 is recommended.

As a result, the RSR hazard curves in Fig. 11 can be proposed for assessment of typical jacket offshore platforms in the South Pars Gas Field area, so that the annual frequency of exceedance associated with the estimated RSR not to be exceeded from an allowable probability (e.g. 10^{-4}).

6. Conclusion

In this paper, a novel probabilistic framework is investigated in order to estimate the mean annual frequency of exceeding various levels of response from elastic to a collapse prevention (CP) limit state associated with jacket platforms against extreme wave loads. Since this approach gains its advantages generally from the incremental wave analysis, the proposed methodology is called probabilistic incremental wave analysis. The PIWA procedure includes three main steps containing multiple-stripe analysis, demand parameter hazard estimation (base shear hazard in current study), and finally evaluation of the limit state frequency. The PIWA framework is introduced step-by-step through a case study jacket platform located in the South Pars Gas Field in the Persian Gulf region. However, the results obtained from the PIWA procedure for the case study jacket are only valid for platforms in this specific site; since these structures encompass the same design specifications and general configuration.

To find the MAF of exceeding the CP limit state, two different approaches including the ultimate capacity-based and the Collapse Wave Height (CWH)-based methodologies are proposed. It is illustrated that the CWH-based procedure generates a much

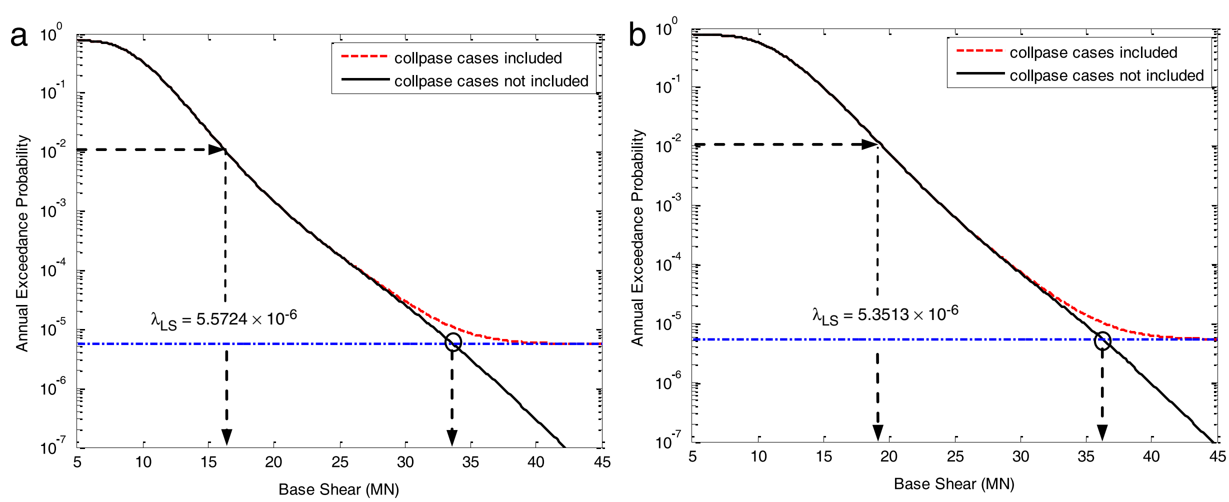

Fig. 10. Estimation of the probabilistic-based RSR (PRSR) from the base shear hazard curve given that no collapse cases are considered, based on (a) SIWA and (b) DIWA.

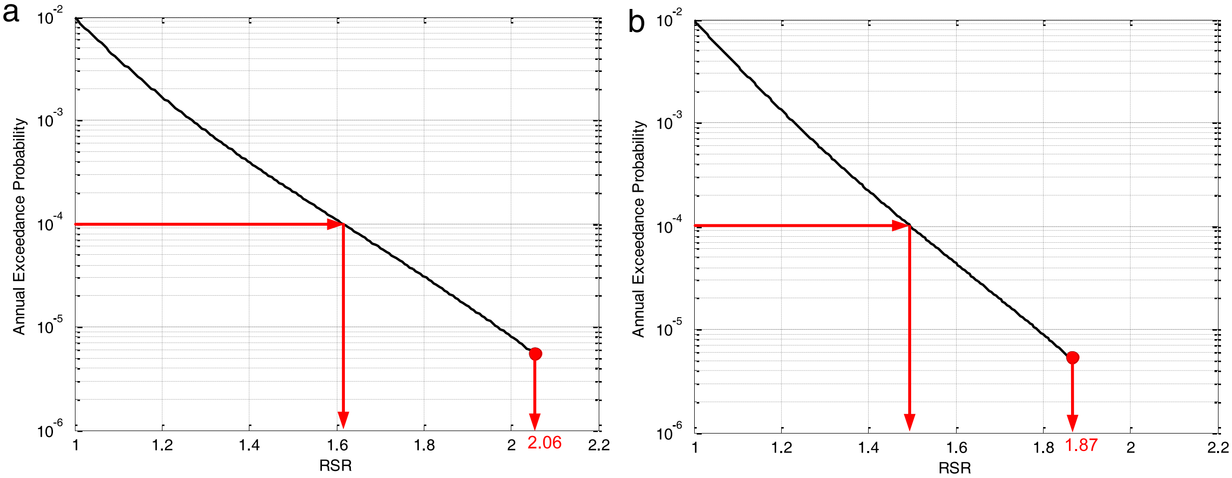

Fig. 11. The RSR hazard curve with the PRSR value marked on the curve based on (a) SIWA and (b) DIWA.

simpler formulation compared to the ultimate capacity-based method. On the other hand, the lower asymptotic limit of the base shear hazard curve is identical to the limit state frequency obtained from the CWH-based approach. In addition, the median of the CP limit state as well as the limit state frequency both estimated by the CWH-based approach have great consistency regarding different type of analyses (static and dynamic incremental wave analyses). Consequently, for the assessment of jacket platforms in the South Pars Gas Field of the Persian Gulf region, it is recommended to estimate the failure probability by the CWH-based approach for which instead of performing computationally time-consuming and complicated dynamic IWA, the simple static IWA can be implemented. In addition, the current study reveals that the lognormal distribution is an adequate model for the base shear at different wave height intensity levels, as well as for the fragility function of the CP limit state.

Moreover, this paper addresses the application of the PIWA framework to represent a probabilistic-based RSR in addition to the RSR hazard curve for estimation of the MAF of exceeding various levels of RSR. Accordingly, it is concluded that the current deterministic-based RSR which is commonly utilized in the assessment of existing offshore platforms, will lead to unreliable estimations. Furthermore, a site-specific RSR hazard curve is proposed by the authors, which is recommended for the assessment of typical jacket platforms in this area of the Persian Gulf.

In this research, an efficient Monte Carlo type simulation, i.e. a combination of LHS and Simulated Annealing optimization technique, is employed for the sampling scheme within the PIWA procedure, which not only can reduce the number of simulations substantially but also can control the statistical correlations among random variables. It is notable that for 7 random variables considered in this study, 15 simulations of samples of random vectors produce the appropriate response distribution.

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