PROBABILITY-BASED ASSESSMENT OF JACKET-TYPE OFFSHORE
Platforms by Using Incremental Wave Analysis

Hossein Ebrahimian, Fatemeh Jalayer, Rodolfo De Dominicis, Gaetano Manfredi

University of Naples Federico II, Dept. Structures for Engineering and Architecture, Italy
ebrahimian.hossein@unina.it, fatemeh.jalayer@unina.it, rodolfo.dedominicis@unina.it, gamanfre@unina.it

INTRODUCTION

The age distribution for jacket-type offshore platforms in the Persian Gulf shows that relatively large numbers of installations have already passed their design life, which is around 25 years in that area. Despite the fact that reconstruction and repair of an existing installation are more economical than construction of a new one, assessment of existing jacket-type offshore platforms beyond their design life has been an issue for consideration. If a structure is intended to be used beyond its design life, a thorough control of the structural safety must be executed. Strictly speaking, desired extension of service life may create a need for requalification of a structure.
Being events with extreme consequences, small frequency, and large uncertainty associated with their occurrence, extreme waves are among the most significant input to a jacket-type offshore platform. Hence, probabilistic methods are required for taking into account the safety compliance with regard to life extension of existing jacket structure.
Offshore design and assessment guidelines (e.g. [1-4]) provide recommendations for target reliability levels, consequences of failure as well as assessment criteria for extreme wave loads. The intended target to analyze is the probability of load exceeding the strength of the structure so that the structure will fail as a result of load exceeding the strength of the jacket structure, the piles, or the topside and its connections to the jacket's structure (for more details see [5-10]).
However, quantitative assessment of risk to a jacket platform under extreme waves poses significant challenges to analysts. It is a multi-disciplinary problem incorporating probabilistic wave hazard analysis of the designated site, geotechnical engineering to quantify the pile-soil interaction, structural engineering to quantify the structure's response and the resulting damage, as well as finance, public policy and construction cost to estimate social and economic consequences of this damage.
Recently, a novel probabilistic framework called *Probabilistic Incremental Wave Analysis* (PIWA) was investigated in order to estimate the mean annual frequency of exceeding various levels of response from elastic to Collapse Prevention (CP) limit state associated with jacket platforms under extreme wave loads [11]. In this methodology, the performance objective can be stated in terms of the mean annual frequency (MAF) of exceeding a given level of structural demand parameter or a desired limit state (e.g. the CP limit state). Since this approach gains its advantages generally from the newly established *Incremental Wave Analysis* [11-13], the proposed methodology is called Probabilistic Incremental Wave Analysis (PIWA). The PIWA simplifies the probabilistic assessment procedures by decoupling the wave loading hazard and structural demand (which is base shear in current study) via an 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 Demand Parameter (DP) are concentrated in wave hazard of the site. This probabilistic framework has conceptual similarity with the proposed probabilistic performance-based seismic assessment of building structures [14].
This study aims to discuss more elaborately the PIWA procedure by implementing wave height-based scheme called the Collapse Wave Height (CWH)-based approach. It is proposed for calculating the MAF of exceeding the CP limit state (see also [11]). Accordingly, various sources of uncertainty are taken into account within the probabilistic evaluation of jacket structures by considering variability in sea state parameters, in the prediction of the wave force on jacket's structure, and finally uncertainty in Pile-Soil Interaction (PSI) (the latter group of uncertainty was

not considered in previous study [11]). The application of the PIWA approach is illustrated through a case study jacket platform located in South Pars Gas Field in the Persian Gulf region.

1 NECESSARY ELEMENTS

This section presents a brief summary of the necessary elements and main concepts associated with this probabilistic procedure including: sources of uncertainty, sampling procedure, the case-study offshore platform, and finally the IWA concept and consequently, the associated Multiple-Stripe Analysis (MSA).

1.1 Sources of Uncertainty

The uncertainties in probabilistic evaluation of jacket structures are broken into four main categories: (1) variability in sea state parameters and inherent randomness in the wave process, (2) uncertainties in the prediction of the wave force on jacket's structure, (3) uncertainties in structural model, and finally (4) the uncertainty in the Pile-Soil Interaction (PSI).

The first category can directly be estimated from the probabilistic wave hazard analysis of the designated offshore site (see [11] and [15]). The second category contains the main parameters influencing the wave force on jacket structure which are comprised of drag coefficient (C_d), inertia coefficient (C_m), and marine growth (*MG*) (see [9] and [11]). Uncertainties in the structural model account for the variability of the physical properties and behavior of the jacket structure for a given design realization. These uncertainties, herein, are composed of the yield stress of jacket legs, $f_{y,L}$, the yield stress of jacket horizontal and diagonal braces, $f_{y,B}$, the modulus of elasticity, E_s, and the vertical loads and masses [11]. It is noteworthy that previous researches considered only the yield stress of steel material (see e.g. [5] or [9]).

Finally, uncertainties in the PSI account for the variability in the characteristics of adjacent soil layers. Uncertainties corresponding to PSI were not widely taken into account for assessment against extreme waves in previous researches. To take into account the effect of PSI, beam-on-nonlinear-Winkler-foundation (BNWF) method is used in this study ([16-17]). In these models, which are widely used in practice, the pile is treated as a beam supported on a Winkler spring foundation; i.e., a series of independent horizontal and vertical nonlinear springs distributed along the pile. To capture lateral response of the piles, soil reaction-force versus pile displacement is modeled with the *p-y* springs; the axial resistance of the soil is provided by a combination of (1) axial load transfer along the sides of the pile modeled with *t-z* springs (i.e. relationship between mobilized soil-pile shear transfer and local pile deflection at any depth) and (2) end bearing resistance at the pile tip described by *q-z* springs (i.e. the relationship between mobilized end bearing resistance and axial tip deflection). The characteristics of these interface springs are estimated based on the properties of the soil deposit in the case study offshore site according to the suggested provisions and recommendations (see [2], [16-17]).

In this study, the soil conditions disclosed by the boreholes (performed at the site) predominantly comprise very silty carbonate clays with varying degrees of cementation and cemented inclusions. The materials are generally moderately overconsolidated in the top 20 m, becoming lightly overconsolidated with depth. The undrained shear strength profile increases from 8 kPa (very soft) at the seabed to 450 kPa (very hard) at the terminal depth of the borehole [15]. Therefore, the main properties of clay layers, which affect the characteristics of nonlinear *p-y*, *t-z* and *q-z* springs, should be taken into account as uncertain parameters in PSI. Accordingly, the undrained shear strength of soil, C_u, the unit weight, γ, and half of the failure strain, ε_{50}, are considered as uncertain parameters in order to probe the effect of uncertainties associated with the PSI on the response of jacket platforms found in cohesive soils (see [13] and [15].

Table 1 illustrates the mean or median values of uncertain parameters (based on the type of distributions) correspond to the best estimates employed in the deterministic model as well as the coefficient of variation (COV). Two different values for C_d, C_m, and *MG* indicate the above and below of 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 (see [11] for the description of the parameters;

nevertheless, that study considered seven uncertain parameter compared the current research with ten uncertain parameters allocating three extra parameters for the PSI).

Table 1. The statistical characteristics of selected uncertain parameters

Uncertain Parameters	Symbol	Mean or 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	[11], [13], and [15]
Inertia coefficient	C_m	1.60, 1.27	0.10	Lognormal	[11], [13], and [15]
Marine growth	MG	75 mm, 50 mm	0.50	Lognormal	[11], [13], and [15]
Parameters influencing uncertainties in structural model					
Loads and masses	m, W	computed	0.10	Normal	[11], [13], and [15]
Yield stress of Legs	$f_{y,L}$	335 MPa, 345 MPa	0.07	Lognormal	[11], [13], and [15]
Yield stress of Braces	$f_{y,B}$	335 MPa, 345 MPa	0.07	Lognormal	[11], [13], and [15]
Modulus of elasticity	E_s	2.0601×10^5 MPa	0.03	Lognormal	[11], [13], and [15]
Parameters influencing uncertainties in the pile-soil interaction					
Undrained shear strength	C_u	*	0.3	Normal	[13], [15]
Unit weight	γ	*	0.1	Normal	[13], [15]
Strain occurs at one-half the maximum stress	ε_{50}	*	0.4	Normal	[13], [15]

* Deterministic values are measured in each layer

In this study, no correlation is considered among these uncertain parameters; moreover, the uncertain parameters corresponding to the wave force on the jacket platform as well as those associated with the structural model are perfectly correlated for different components (see [11] and [15]). In addition, the uncertain parameters corresponding to the PSI are fully correlated for different soil layers (see [13] and [15]). These assumptions are made to reduce the number of uncertain parameters, and thus the computational effort (for more details see [11] and [18]).

1.2 Sampling procedure for considering the effect of uncertainties

A variety of techniques have been proposed to address the effect of different sources of uncertainty on probabilistic estimation of structural response, and the pros and cons of these methodologies are investigated. These approaches range from simplified first-order second-moment (FOSM) reliability methods to more general Monte Carlo type simulations and also response surface technique (see [11] and [15] for a general description of these methods).

In order to reduce the number of simulations, N_{sim}, while gaining an acceptable level of accuracy for the statistical characteristics of the response function, a combination of Latin Hypercube Sampling approach (LHS) and Simulated Annealing (SA) optimization technique is shown to yield reasonable estimates in the PIWA framework (for more details see [11] and [15]). Accordingly, N_{sim}=30 can give rise to the appropriate accuracy for 10 considered uncertain parameters (N_{var}=10) indicated in *Table 1*.

1.3 Case study: SPD2 Platform

The SPD2 jacket-type platform located in South Pars Gas Field Phase 1 in the Persian Gulf region is employed in this study. This wellhead platform is located in 65 meter water depth, and consists of six legs together with one battered face. The jacket plan dimension is about 16.00m×27.50m at topside elevation and 23.4m×37.7m at the mud line. The structure is fixed to the ground by 6 through-leg grouted piles. This offshore facility was operational in 2002; thus, nearly above one third of the design life of this platform has been expired. A general description of this platform and its related characteristics as well as the soil characteristics of the offshore site can be found in [11], [13], and [15]. A 3-dimensional model of the jacket structure including piles is constructed in the finite element program USFOS (2009) [19], which has the capability to perform nonlinear static and dynamic analyses of wave-induced jacket platforms. General configurations of the platform (real and shown in its 3D model with and without pile) are illustrated in *Fig. 1*. 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.

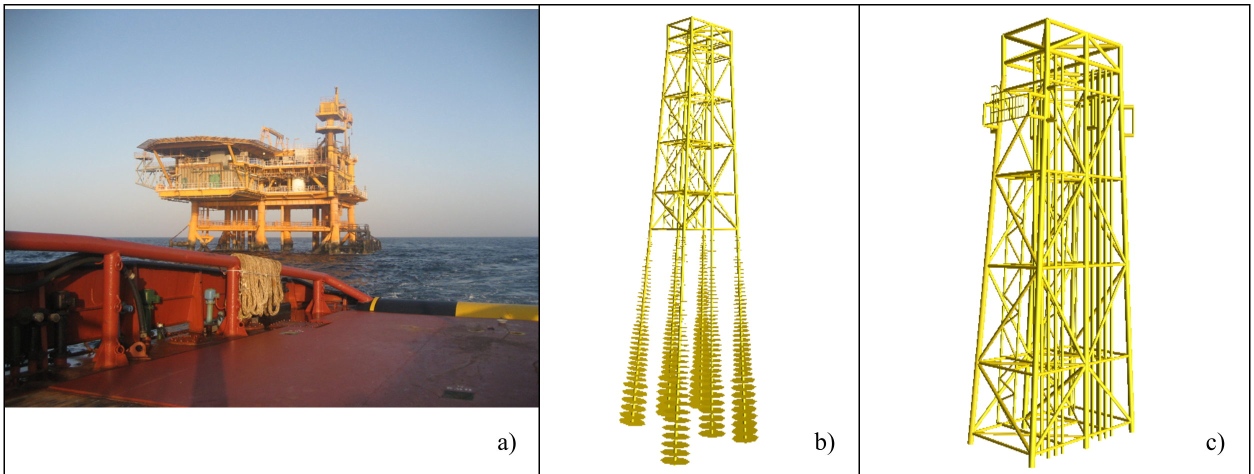

Figure 1. a) General configuration of the SPD2 platform, b) 3D model of the SPD2 platform with piles, c) 3D model above mud line

1.4 Incremental Wave Analysis (IWA) and the associated Multiple-Stripe Analysis (MSA)

The IWA approach introduced primarily in [12] is a novel and emerging approach, which has the capability to predict more thoroughly the response of the platform and its associated limit-state capacity under extreme waves. The static or dynamic IWA (namely SIWA and DIWA) involves subjecting the jacket structural model to extreme wave with incrementally scaled wave heights, performing a nonlinear static or dynamic analysis for each individual wave height, and subsequently, accomplishing the structural demand parameter of interest (e.g. base shear in this study). The extreme waves are produced based on the regular waves from Stokes' 5th order theory; in addition, the wave period is assumed to be constant within each level of wave height intensity (for more details see [11]).

Plotting the demand or wave height intensity versus deck displacement for each individual analysis produces a curve of the required response parameterized for various wave intensity levels (*Fig. 2*). Consequently, the point on the curve with a tangent slope equal to or less than 15% of the elastic slope is defined to be the Collapse Prevention (CP) limit state of the platform. This means that the (ad hoc) 15% of elastic stiffness detects impending collapse for which the flattening of the curve is an indicator of global instability (i.e., increasing the deck displacement at ever higher rates). It is worth noting that the case-study jacket structure has several collapse modes comprising pile or soil failure modes as well as failure of braces. All of these failure modes are evident in deck displacement while failures of braces are more apparent in maximum inter-level drifts; hence, it is advantageous to detect the global collapse in the IWA curve based on deck displacement. As a result, the base shear and wave height corresponding to the CP limit state are called the ultimate capacity and Collapse Wave Height (CWH) of platform, respectively (see *Fig. 2*).

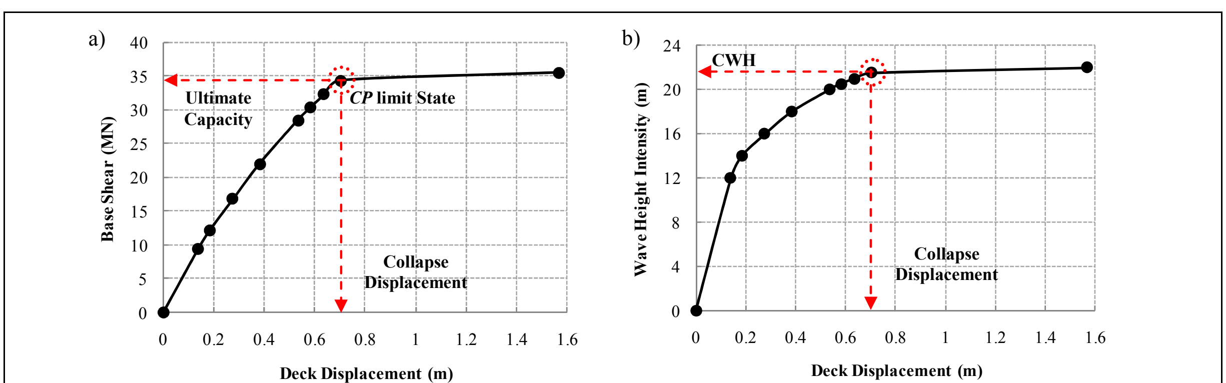

Figure 2. Different illustrations of the IWA curve, a) base shear vs. global deck displacement, and b) wave height vs. global deck displacement, together with indicators for CP limit state

The multiple IWA is a collection of IWA curves for N_{sim} sets of structural model realizations. Subsequently, the multiple-stripe analysis (MSA) is in essence the re-compilation of the results of the multiple IWA for the N_{sim} sets of structural model, at multiple levels of wave height intensities. In other words, for each specific wave height intensity, the jacket static or dynamic responses for N_{sim} sets of realizations are summarized as a single stripe of data, also known as single-stripe analysis. Accordingly, the collection of single-stripe analyses for sequential levels of wave height intensities can be summarized as the MSA (see [11] and [14]). *Fig. 3* illustrates the results of the MSA for the case study SPD2 platform utilizing 30 sets of structural models considering both SIWA and DIWA. The points associated with the CP limit state, which indicates global collapse of the jacket structure, are marked on those figures as collapse cases.

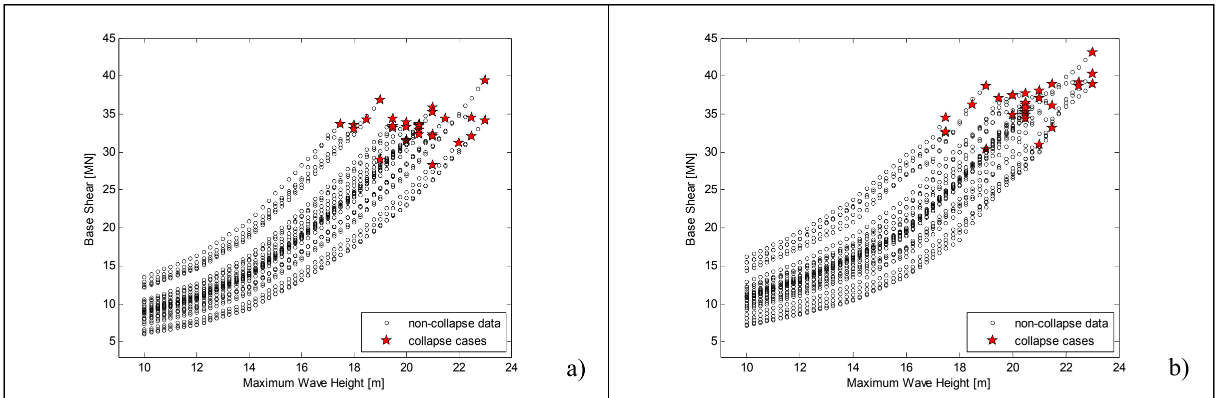

Figure 3. MSA data for the base shear together with collapse cases marked by using a) SIWA; b) DIWA

2 CALCULATION OF LIMIT STATE FREQUENCY

This section provides a methodology for estimating the MAF of exceeding the CP limit state (this limit state is defined in previous section), denoted as λ_{LS}. This methodology is called herein the CWH-based approach. Considering the *CWH* as a random variable, one can say that collapse occurs when the wave height intensity measure, $H_{\max}$, at a prescribed level equal to *h* exceeds the collapse wave height, *CWH*. The limit state frequency can be obtained from the following discrete and continuous expressions by successive applications of Total Probability Theorem [20]:

$$\begin{aligned} \lambda_{LS} = \nu P\left[CWH < H_{\max}\right] &= \sum_{all\ h} P\left[CWH < H_{\max} \middle| H_{\max} = h\right]\left(\nu P\left[H_{\max} = h\right]\right) \\ &= \int_{h} F_{CWH}\left(h\right)\left|\mathrm{d}\lambda_{H_{\max}}\left(h\right)\right| \end{aligned} \tag{1}$$

where *v* represents the annual rate of occurrence of the events (storms) or the number of 3-hour sea states in one year (see [11] and [15]); $P[H_{\max}=h]$ 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 (see [11] and [15]). In addition, F_{CWH} is the Cumulative Density Function (CDF) of the collapse wave height known as the CWH fragility function; $\lambda_{H\max}(h)$ illustrates the site-specific wave height hazard in terms of the MAF of exceeding the wave height intensity level, *h*; consequently, $\mathrm{d}\lambda_{H\max}$ represents the differential of the wave height hazard curve.

*Fig. 4*a and *Fig. 4*b illustrate the CWH fragility function for the case study jacket platform based on static and dynamic IWA, respectively. The median, η_{CWH}, and the COV of the CWH are indicated on those figures by considering the *CWH* data (*Fig. 3*) to be lognormally distributed. It is revealed that the lognormal distribution parameters are not sensitive to the static or dynamic wave analyses. Moreover, $\lambda_{H\max}$ is displayed in *Fig. 4*c for the SPD2 jacket's site (see [11] and [15] for more details). Different estimates for the CWH-based λ_{LS} are summarized in *Table 2* based on the type of fragility function F_{CWH} (i.e. consider empirical or lognormal distribution of *CWH* data) as well as the method of analyses.

It can 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

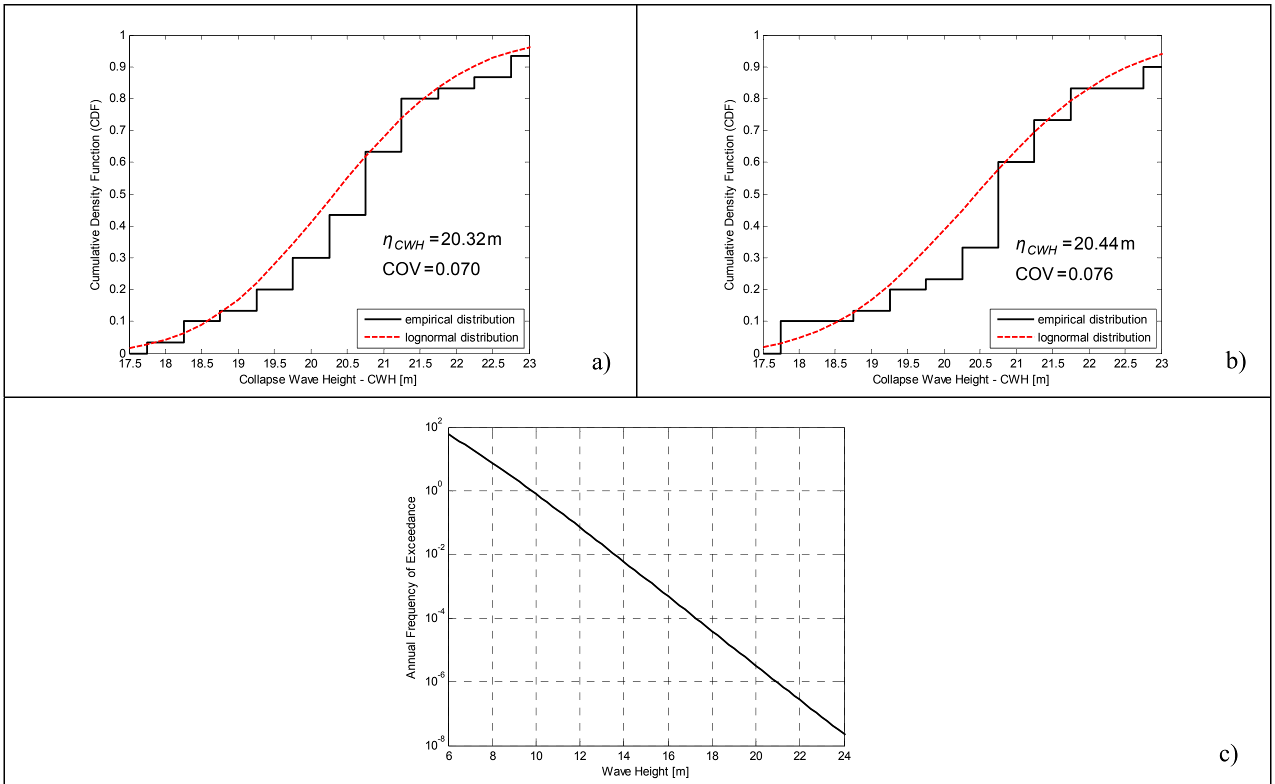

Figure 4. Comparison of the CWH fragility curves by applying empirical and lognormal distributions based on a) SIWA and b) DIWA, c) Mean annual frequency of exceeding maximum wave height (wave height hazard curve, [11] and [15])

Table 2. The CP limit state frequency by the CWH-based approach

Type of Solution	Distribution of Fragility Function	SIWA	DIWA
Exact	Empirical	5.8575×10^{-6}	7.2374×10^{-6}
	lognormal	7.2117×10^{-6}	7.7720×10^{-6}

3 SUMMARY

This study aims to further explore the recently proposed probability-based framework for jacket-type platforms against extreme waves, which is called probabilistic incremental wave analysis (PIWA, [11]). It is shown how this method can provide a direct estimate of the mean annual frequency of exceeding the collapse prevention (CP) limit state associated with jacket platforms. This goal has been achieved by taking into account appropriate sources of uncertainty within the jacket platform. Accordingly, a wave height-based approach is implemented herein in order to quantify the MAF of exceeding the collapse prevention (CP) limit state, which is called Collapse Wave Height (CWH)-based approach. The CWH is the wave height corresponding to the CP limit state. The proposed method convolves the CWH fragility of the jacket structure and the wave height hazard of the offshore site.

Different aspects and the necessary elements of the PIWA methodology as well as the proposed limit state frequency calculation are introduced step-by-step through a case study jacket platform located in the South Pars Gas Field in the Persian Gulf region. In detail, it is revealed 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.

This investigation can be utilized as an efficient reliability assessment approach for jacket-type offshore platform. It is noteworthy that the results obtained herein for the case study jacket are valid for platforms in this specific site, since these structures encompass the same design specifications and general configuration.

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ebrahimian.hossein@unina.it, fatemeh.jalayer@unina.it, rodolfo.dedominicis@unina.it, gamanfre@unina.it

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ABSTRACT

A methodology for calculating the mean annual frequency (MAF) of exceeding a specified limit state is presented for existing jacket-type offshore platforms under extreme waves. This methodology gains its advantages from the Probabilistic Incremental Wave Analysis (PIWA) framework [1]. PIWA is a novel probabilistic framework which is established in order to assess the performance of jacket offshore platforms under extreme waves. Taking into account various sources of uncertainty, 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 (IWA) [2] are employed to estimate the distribution of structural demand for a wide range of wave height intensities.
Accordingly, a wave height-based approach is implemented herein in order to quantify the MAF of exceeding the collapse prevention (CP) limit state, which is called Collapse Wave Height (CWH)-based approach. This proposed method is employed in probabilistic assessment of an existing jacket offshore platform located in the Persian Gulf region. For that reason, various sources of uncertainty are taken into account within the probabilistic evaluation of jacket structures by considering variability in sea state parameters, in the prediction of the wave force on jacket's structure, and uncertainty in Pile-Soil Interaction (PSI). It is noteworthy that the latter group of uncertainty (related to PSI) was not considered within the previous study [1]. 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 (see [1] for more details). The application of the PIWA approach is illustrated through a case study jacket platform located in South Pars Gas Field in the Persian Gulf region.

CONCLUSIONS

Different aspects and necessary elements of PIWA methodology are introduced step-by-step through a case study jacket platform located in the South Pars Gas Field in the Persian Gulf region. It is concluded that the wave height-based method (called the CWH-based approach) can provide a direct estimate of the mean annual frequency of exceeding the collapse prevention (CP) limit state associated with jacket platforms. This limit state frequency is obtained by convolving the CWH fragility function and the wave height hazard of the offshore site. The CWH fragility function is the Cumulative Density Function (CDF) of the collapse wave height, where the CWH is the wave height corresponding to the CP limit state. It is shown how this limit state is attained from the IWA curve. Furthermore, the wave height hazard is estimated by the probabilistic wave height hazard analysis of the designated offshore site (see [3] for complete derivations).

Appropriate sources of uncertainties are taken into account for the goal of estimating the MAF within the PIWA framework. It is shown 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. The proposed assessment scheme can be utilized as an efficient reliability approach for jacket-type offshore platform against extreme waves. It is noteworthy that the results obtained herein for the case study jacket are valid for platforms in this specific site, since these structures encompass the same design specifications and general configuration.

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