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Assessment of offshore structures under extreme wave conditions by Modified Endurance Wave Analysis

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A B S T R A C T

Recently, various approaches have been introduced to estimate the response of offshore structures in different sea states by stepwisely intensifying records. In this article, a more practical approach entitled Modified Endurance Wave Analysis (MEWA) considering the random and probabilistic nature of wave loading and utilizing optimal time duration is introduced. Generation procedure of this approach is described based on two practical wave theories: random and constrained new-wave. In addition, assessment of a simplified model representing a typical fixed offshore platform under extreme wave conditions in the Persian Gulf is performed making use of MEWA. A comparative analysis has been also carried out to investigate the accuracy and computational costs of MEWA. The results indicate that MEWA can be a time-saving and also reliable method both in design and assessment of offshore platforms.

1. Introduction

The main goal of evaluation of structures under extreme waves is to ensure that they can resist storm loading in different sea state conditions. In other words, the response of the structure must be

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acceptable for demanding requirements such as production activity, safety and serviceability of the offshore structure [1]. Also, changing in platform usage, modifications to the platform conditions and a re-evaluation of the environmental loading emphasize the necessity of assessment [2]. However, because of complicated geometries, Fluid Structure Interaction (FSI) and soil-pile-structure interaction, the assessment is a challenging procedure in offshore engineering practice.

Because of the dynamic nature and randomness of sea waves, time domain is a trustworthy method for accurate evaluation of structural performance especially in deep water and flexible structures [3]. In this method, dynamic behavior of the platform can be considered under random wave loading exerted to the members as a function of time. However, despite advantages of the time history method, it is conceptually complicated and time consuming and therefore it has limited application in usual assessment practices.

During the last few decades, several studies have been carried out to assess offshore structures under extreme wave loading such as Kjeøy et al. [4], Morandi et al. [5] and Vanraaij and Gudmestad [6]. Advances in computer processing power make it possible to analyze the platform under different extreme wave conditions using time history-based methods. Golafshani et al. introduced Incremental Wave Analysis (IWA) to assess the structural performance under various wave excitations [7]. Conceptually, they took advantage of Incremental Dynamic Analysis (IDA) which is a well-known method in seismic assessment of structures [8]. To overcome the computational cost limitations, they proposed that instead of considering a 3-h interval, it is practical to take into account only the maximum wave height of the record. This way cannot properly indicate the realistic nature of wave loading; therefore, the authors emphasized the possibility of obtaining unreliable results in other case studies. Zeinoddini et al. presented Endurance Wave Analysis (EWA) in which constrained new-wave is used as a reliable theory for simulation different extreme events [9]. In this method, time of the record is decreased significantly and a fixed time interval is utilized for each sea state. In addition, linear increase was proposed for representing the growing trend of the significant wave height.

Notwithstanding valuable advantages of the EWA, this method excludes the extreme value statistics in estimation of the response of offshore platforms. Moreover, this method requires modifications especially in selection of time duration and increasing trend. Modified Endurance Wave Analysis (MEWA) is a probabilistic approach offering a reliable procedure to acquire the optimum duration of the records and a practical way for increasing trend of the excitation. For considering the stochastic nature of wave loading, 500 time history records are generated, and extreme response distribution of the demand parameters is studied. Widespread use of return period in assessment procedures has convinced the authors to use this concept as increasing trend of the significant wave height. This method can be efficiently employed both for design and assessment of offshore structures.

2. Concept of EWA and MEWA methods

EWA is a new time history-based approach for estimating the performance of platforms by stepwisely increasing wave profile named Intensifying Wave Train Function (IWTF). Basic concepts of EWA method can be described by a hypothetical experiment as shown in Fig. 1. In this experiment, three different platforms with unknown dynamic characteristics are exposed to an IWTF. At the beginning, the structures are subjected to a time history wave loading corresponding to a certain significant wave height (H_s), peak spectral period (T_p) and time duration (t_d). Since the amplitude of the excitation is quite low, all three platforms remain stable after this loading. In the next steps, the significant wave heights increase linearly whereas time duration is the same as the prior one. As time goes on, a point is reached when one of the platforms (platform C) exceeds its serviceability limit. As time passes more, the excitation becomes severe, such that platform C collapses, platform A is damaged severely but platform B still continues its serviceability. According to this experiment, the more endurance time, the better structural performance. In this method, any reasonable Engineering Demand Parameter (EDP) can be considered and the analysis will be continued until the desired level of excitation has been covered.

Generally, in this study, three challenges in EWA method are considered and MEWA is proposed based on these points:

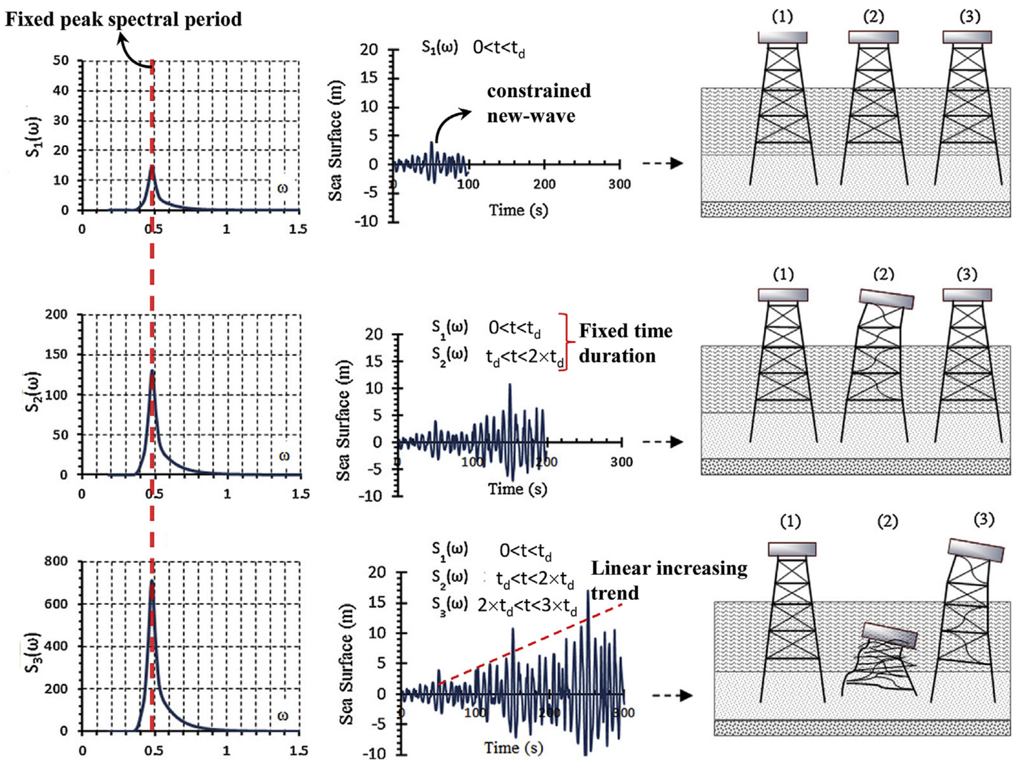

Fig. 1. The hypothetical test showing the concept of EWA method [9].

1) Due to random nature of extreme waves, deterministic approaches cannot achieve reliable results; therefore, an adequate number of time history analyses should be performed. Moreover, the response of the structure should be assessed by considering probability distribution of the EDPs. In this way, various probability distributions such as Generalized Extreme Value (GEV), lognormal, gamma and normal can be efficiently utilized; and based on the acceptable risk limits, reliable response can be estimated for the offshore structure.
2) In the EWA method, time duration is fixed to a specific value for all excitation levels. In this case, if time duration assumed to be large, not only the calculation cost is increased but also the main objective of the EWA (i.e. time-saving) is conflicted. On the other hand, both the critical response of the platform and the stationary form of the sea surface may be ignored for short time duration. As it will be explained later, for each sea state, time duration can be varied and selected based on a factor of the peak spectral period.
3) No consideration on increasing trend of H_s was carried out in the EWA methodology and only linear increase was suggested. According to the application of return period in assessment of offshore platforms, the increase of H_s with regard to wave return period can result in convenient application for ocean engineering practice. Also, due to nature of wave loading, peak spectral period must be enhanced with increase of significant wave height.

Fig. 2 shows comparison between the EWA and MEWA records. As seen in part (a), in the EWA record, t_d is fixed and H_s increases linearly. Also, as shown in generated EWA records, T_p is assumed to be constant for all sea states. In part (b), it is pointed out that MEWA is a stochastic approach that t_d is changed in each sea state, H_s increases based on the return period and T_p is specified in conjunction with H_s. Thus, MEWA method has more optimized computational time, convenient application and

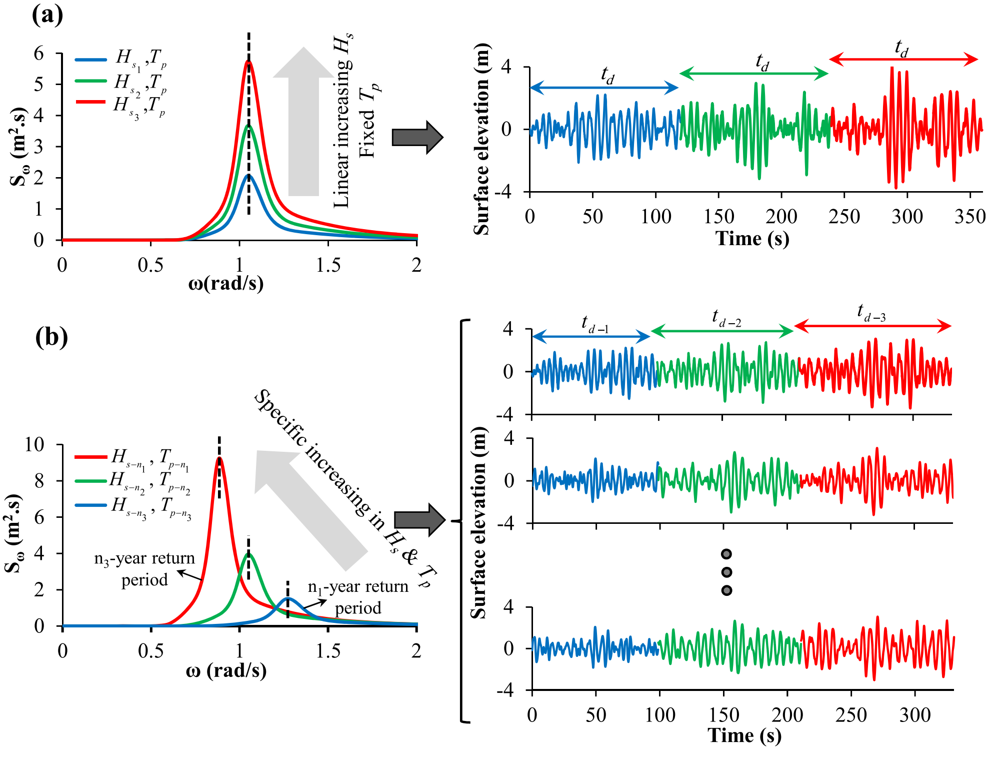

Fig. 2. A sample of (a) EWA and (b) MEWA records and their corresponding spectrums.

compatible profile with wave phenomena. It should be also noted that using constant zero-crossing period in generation of the records can be useful and computationally attractive especially in the reliability approaches. More detailed descriptions can be found in Refs. [10–12].

3. Specifications of MEWA records

3.1. Stochastic approach

Response of offshore structures cannot be reliably predicted in a deterministic way, because sea waves have irregular profiles changing randomly in time and space [13,14]. In the MEWA, the probabilistic approach is inspired from probabilistic seismic assessment of building structures [8]. In this regard, the random nature of wave loading and its effects on the response of offshore structures should be considered. For each sea state (corresponding to intensity measure in seismic approach), a large number of time history analyses is performed and EDPs such as deck displacement and base shear are determined. To reduce the number of numerical simulations, the probability distribution that yields close agreement with response of the structure can also be used to determine the distribution curve. The goal of this procedure is to determine the stochastic response for each excitation level (sea state) and to achieve practical probabilistic performance curves which will be discussed in more details later.

3.2. Time duration

The use of time domain simulations presents advantages in reliable estimation of extreme responses of offshore structures under storm loading. Various methods can be used in generation of time series records in which consideration of optimal time duration is invaluable because of computational costs. In this way, report of ITTC seakeeping committee has been indicated the importance of this issue and proposed total number of wave cycles for determination of the time duration [15]. In the EWA method, the time duration of the generated records based on random waves is fixed to 100 s for all excitation levels that cannot simulate the ocean surface of a sea state properly. As usual, 3 h is a common time duration suggested for the record of random waves; however, this is not an appropriate interval for all sea states. This is especially important in the collapse analysis under wave loading in which severe sea states are exerted to the structure. In this regard, DNV-RP-C205 states that the period of the stationary sea surface can vary from 30 min to 10 h [16].

In the presented method, time interval of the analysis is proposed as a function of structural period (T_s) and peak spectral period (T_p) and is defined by Eq. (1).

$$\text{Time interval} = \max(TDF_p \times T_p,\ TDF_s \times T_s) \tag{1}$$

where TDF_p and TDF_s are Time Duration Factor (TDF) of peak spectral period and structural period, respectively. TDF_s should be determined such that the time interval of the simulation for linear systems is taken as slightly larger than the memory in the system which is about 50 s. For highly non-linear systems, e.g. parametric rolling of ships, it should, however, be much larger: 300–500 s. In here, assessment of fixed offshore structures in the linear range is studied; and therefore "$TDF_p \times T_p$" is the dominant term. In addition, if the time interval determined based on the wave period is less than 50 s, the effects of structural term "$TDF_s \times T_s$" should also be studied.

To obtain an optimal time interval for each sea state, characteristics of the sea surface elevation such as maximum wave crest elevation and significant wave height are considered. Random wave and constrained new-wave theories are utilized in generation of the MEWA records and with considering the variation of wave characteristics, reasonable TDF_p is obtained.

To investigate the applicability of TDF_p for random surface elevation, significant wave height (H_s) and maximum wave crest elevation ($\eta_{\max}$) are used as desired criteria. As shown in Fig. 3, $\eta_{\max}$ can be formulated based on sea surface elevation ($\eta(t)$) as follows:

$$\eta_{\max} = \text{Max}\left(Abs\left(\eta(t) \quad : \quad t \in [0\ ,\ TDF_p \times T_p]\right)\right) \tag{2}$$

Due to the random nature of sea waves, for each return period, 500 generated records are considered according to the Persian Gulf storm sea states. Fig. 4 exhibits the variation of mean and coefficient of variation (CV) for the mentioned criteria via different TDF_ps and return periods. As it was expected, for higher return periods, the means and CVs have larger amounts. Also, after a certain TDF_p, the variation of means and CVs decreases, confirming the assumption that required time duration can be assumed as a factor of T_p. This tendency can be observed in higher TDF_p for $\eta_{\max}$ indicating the more sensitivity of this parameter. However, 1000 T_p seems to be a reliable time duration in the generation

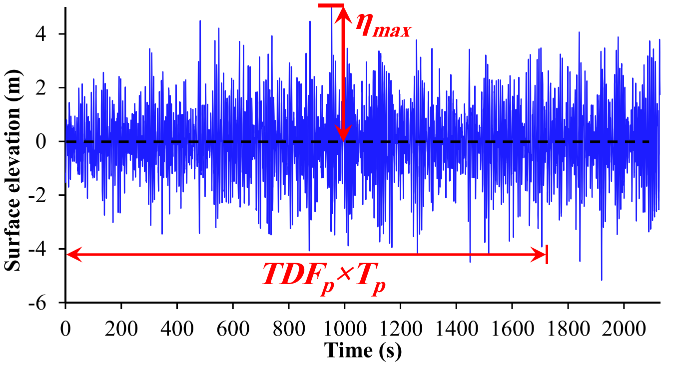

Fig. 3. A record of random wave theory and the maximum amplitude $\eta_{\max}$.

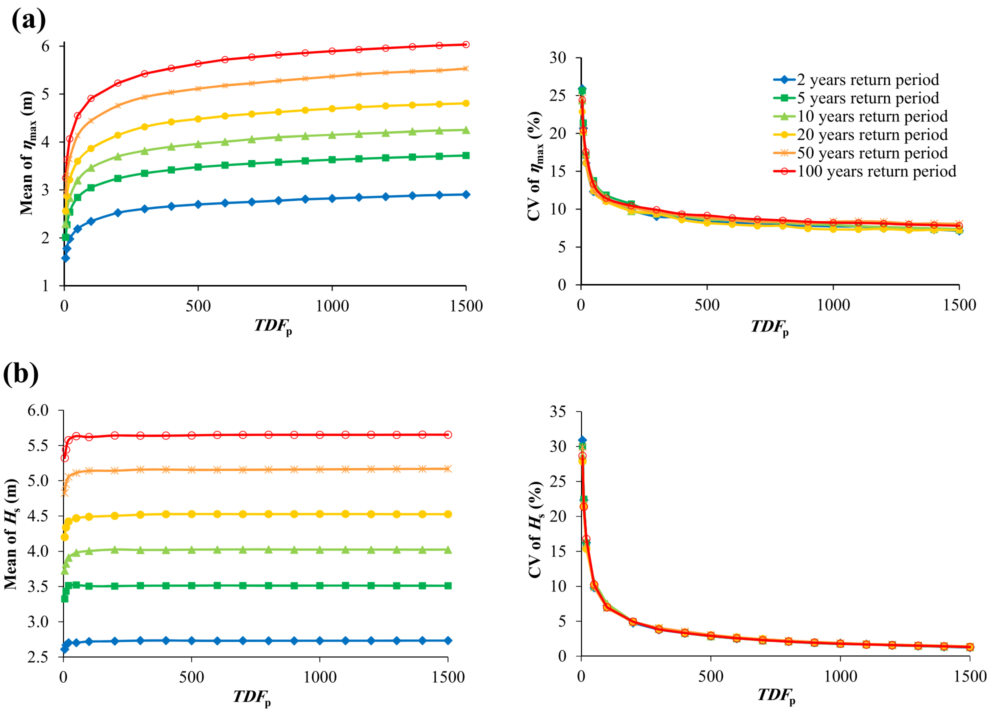

Fig. 4. Mean and CV of (a) η_{max} and (b) H_s for random wave records and different return periods.

procedure of MEWA records. It should be noted that this value for TDF_p is selected based on sea states of the site and engineering judgment; therefore, for other sites another TDF_p may be determined.

Constrained new-wave theory is the combination of new-wave and random wave theories [17]. New-wave is a linear form of the most probable extreme wave in a random sea and has the shape of the autocorrelation function [18,19]. In fact, constrained new-wave is the new-wave theory along with the effects of randomness. As shown in Fig. 5(a), constrained new-wave and corresponding new-wave theories have a common base, profile and same extreme value. Consequently, with regard to the importance of the extreme value and the simplicity of the new-wave profile, consideration of TDF_p can be carried out based on this theory. The proposed criterion for calculating the optimum time is considered as $\eta_{min}(t)/\eta_{max}(t)\ :\ t\in[0,\ TDF_p\times T_p]$ in which η_{min} and η_{max} are the minimum and maximum wave crest elevation in the investigated time interval, respectively, and shown in Fig. 5(a). Fig. 5(b) represents the defined ratio for different TDF_ps in 100-year return period. Similar values are also achieved for other return periods. Hence, after $20T_p$, the ratio is tending towards zero (less than 1%); therefore, $TDF_p = 20$ is a suitable factor for constrained new-wave theory.

3.3. *Increasing trend of significant wave height*

As it was mentioned earlier, intensification of H_s based on the wave return period can improve the application of EWA records. The return period "*n*" indicates that the wave height will be exceeded on average once in every *n* years. Extreme wave height analysis can determine the significant wave height corresponding to a specified return period using statistical methods. In this regard, a data set obtained from direct measurements or meteorological information is required. In the next step, the extreme wave data is fitted by choosing a suitable probability distribution. In the Persian Gulf, the Gumbel

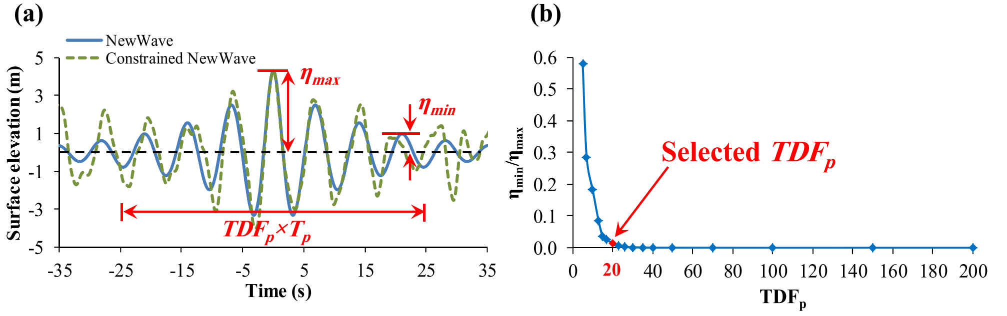

Fig. 5. (a) Defining parameters of TDF_p criterion for new-wave and constrained new-wave theories and (b) the ratio of minimum and maximum amplitudes for different TDF_ps.

distribution which is defined by Eq. (3) is more appropriate for probability distribution of extreme waves [20].

$$H_{s-n} = A\left\{-\ln\left[-\ln\left(1-\frac{1}{\lambda n}\right)\right]\right\} + B \tag{3}$$

in which $H_{s\text{-}n}$ is the significant wave height corresponding to return period n. A and B are the distribution parameters and λ is the sample intensity. For the site in the Persian Gulf region, based on the recorded data, A, B and λ are obtained, 0.71, 2.52 and 1.04, respectively. Also, the peak spectral period for each wave height corresponding to n-year return period is given in Eq. (4).

$$T_{p-n} = a \cdot H_{s-n}^{b} \tag{4}$$

where a and b are empirical coefficients and suggested to be 2.94 and 0.5 in the Persian Gulf [21]. Fig. 6 shows the Gumbel distribution for wave data in the site and represents the significant wave height and peak spectral period for their corresponding return periods. Also, data selected for the increasing trend of the MEWA records are presented in Table 1.

Accordingly, by using this increasing trend in the MEWA method, return period of the platform can be determined by monitoring the EDPs and their allowable limits.

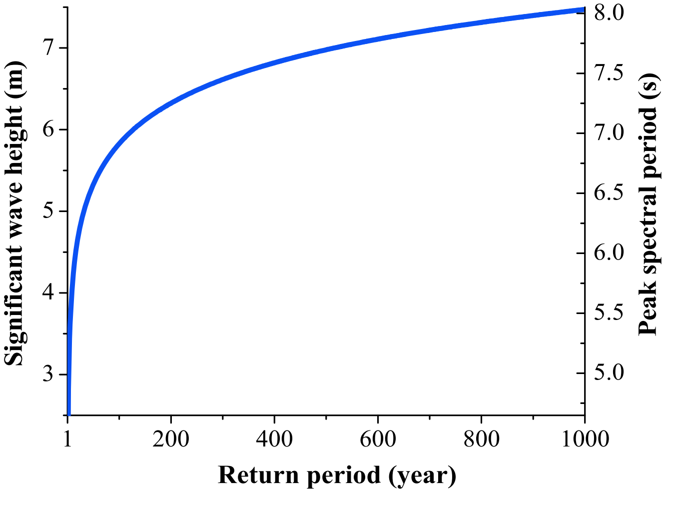

Fig. 6. Significant wave height and its corresponding peak spectral period via return period in the Persian Gulf.

Table 1
Characteristics of the wave excitation for different return periods in the Persian Gulf.

Peak spectral period $T_{p\text{-}n}$ (s)	Significant wave height $H_{s\text{-}n}$ (m)	Return period (n-year)
4.94	2.82	2
5.59	3.62	5
5.99	4.15	10
6.35	4.67	20
6.77	5.33	50
7.10	5.83	100

4. Generation of MEWA records

Different theories (regular or irregular) can be used for generating MEWA records called Modified Intensifying Wave Train Functions (MIWTFs) [22]. To this aim, m separated time history records are joined together to form a single time series in which each record is a representative of the wave spectrum with specific return period. As mentioned above, for k-th profile, time duration is a function of T_{p-nk} and T_s. This factor should be determined according to the selected wave theory; and hence the time interval of k-th profile can be written as follow:

$$\begin{aligned} & t_{k-1} \le t \le t_k \\ & t_0 = 0 \\ & t_k = t_{k-1} + \max\left(TDF_{p-k} \times T_{p-\,n_k}, TDF_s \times T_s\right) \end{aligned} \tag{5}$$

In addition, the significant wave height and its corresponding wave energy spectrum should intensify based on demanding return periods. In other words, the k-th profile represents time history of special wave energy spectrum in which significant wave height is defined with regard to k-th return period. Also, the peak spectral period is increased in conjunction with the significant wave height. Here, the generation procedure of MIWTFs is described based on the random and constrained new-wave theories.

4.1. Modified Intensifying Random Sea Elevation model (MIRSE)

Sea waves have irregularity and randomness in shape, height and wave length, so random wave theory can be accounted as a suitable method for simulation of wave loading. This theory can be considered in both linear and nonlinear forms. Generally, design standards propose linear random wave theory as a reliable method for practical assessment of fixed platforms [2,16]. The linear form is obtained from the summation of small sinusoidal waves (Airy waves) with different heights, frequencies and phases. For proper simulation of the sea state, the corresponding wave energy spectrum should also be divided into at least 1000 equal frequency intervals [16].

MEWA records generated with regard to linear random wave theory are named Modified Intensifying Random Sea Elevation (MIRSE) and can be explained as follow:

$$\eta_{\mathrm{MIRSE}} = \begin{cases} \sum_{i=1}^{N} \sqrt{2S_1(\omega_i)\Delta\omega}\cos(-\omega_i t + \varepsilon_{i1}) & t_0 \le t < t_1 \\ \sum_{i=1}^{N} \sqrt{2S_2(\omega_i)\Delta\omega}\cos(-\omega_i t + \varepsilon_{i2}) & t_1 \le t < t_2 \\ \vdots & \\ \sum_{i=1}^{N} \sqrt{2S_k(\omega_i)\Delta\omega}\cos(-\omega_i t + \varepsilon_{ik}) & t_{k-1} \le t < t_k \\ \vdots & \\ \sum_{i=1}^{N} \sqrt{2S_m(\omega_i)\Delta\omega}\cos(-\omega_i t + \varepsilon_{im}) & t_{m-1} \le t \le t_m \end{cases} \tag{6}$$

where η_{MIRSE} is the surface elevation of MIRSE record, $S_k(\omega_i)$ is the value of $S_k(\omega)$ in frequency of ω_i for the k-th wave spectrum (corresponding to k-th return period) and ε_{ik} shows the random phase.

In the Persian Gulf and for considered return periods, 1000 is assumed as an appropriate TDF_p for the records obtained from the random wave theory. For instance, a sample of MIRSE generated based on 2, 20 and 100-year return periods is shown in Fig. 7(a).

4.2. Modified Intensifying Constrained New-Wave model (MICNW)

In summary, constrained new-wave is a random elevation constrained to the most probable new-wave crest at a specified time. In addition to short time of analysis, constrained new-wave covers the absence of randomness in the new-wave profile by considering a random process. Therefore, this theory can be accounted as a proper method for determining the extreme response of the platforms under wave loading. For further details, the constrained new-wave theory and its application was described by Taylor et al. [17].

Similar to the methodology that is applied for generation of the MIRSE records, the production of Modified Intensifying Constrained New-Wave (MICNW) records can be expressed as follow:

$$\eta_{\mathrm{MICNW}}(t)=\begin{cases}\eta_{R_1}(t)+\rho_1(t)\left[\alpha_1-\sum_{n=1}^{N/2}a_{1n}\right]+\dfrac{-\dot{\rho}_1(t)}{\lambda_1^2}\left[\dot{\alpha}_1-\sum_{n=1}^{N/2}\omega_n b_{1n}\right] & t_0\le t<t_1\\ \eta_{R_2}(t)+\rho_2(t)\left[\alpha_2-\sum_{n=1}^{N/2}a_{2n}\right]+\dfrac{-\dot{\rho}_2(t)}{\lambda_2^2}\left[\dot{\alpha}_2-\sum_{n=1}^{N/2}\omega_n b_{2n}\right] & t_1\le t<t_2\\ \vdots & \\ \eta_{R_k}(t)+\rho_k(t)\left[\alpha_k-\sum_{n=1}^{N/2}a_{kn}\right]+\dfrac{-\dot{\rho}_k(t)}{\lambda_k^2}\left[\dot{\alpha}_k-\sum_{n=1}^{N/2}\omega_n b_{kn}\right] & t_{k-1}\le t<t_k\\ \vdots & \\ \eta_{R_m}(t)+\rho_m(t)\left[\alpha_m-\sum_{n=1}^{N/2}a_{mn}\right]+\dfrac{-\dot{\rho}_m(t)}{\lambda_m^2}\left[\dot{\alpha}_m-\sum_{n=1}^{N/2}\omega_n b_{mn}\right] & t_{m-1}\le t\le t_m\end{cases} \tag{7}$$

where η_{MICNW} is the surface elevation of MICNW, k represents the k-th profile, α_k is the crest elevation and $\dot{\alpha}_k$ is its slope. $\rho_k(t)$ and $\dot{\rho}_k(t)$ are the unit new-wave and its slope, respectively. λ_k^2 is obtained from the second spectral moment and variance of the wave energy spectrum ($\lambda_k^2=m_{2_k}/\sigma_k^2$). η_{Rk} is a random process that can be written as:

$$\eta_{R_k}(t)=\sum_{n=1}^{N/2}\left(a_{kn}\cos(\omega_n t)+b_{kn}\sin(\omega_n t)\right) \tag{8}$$

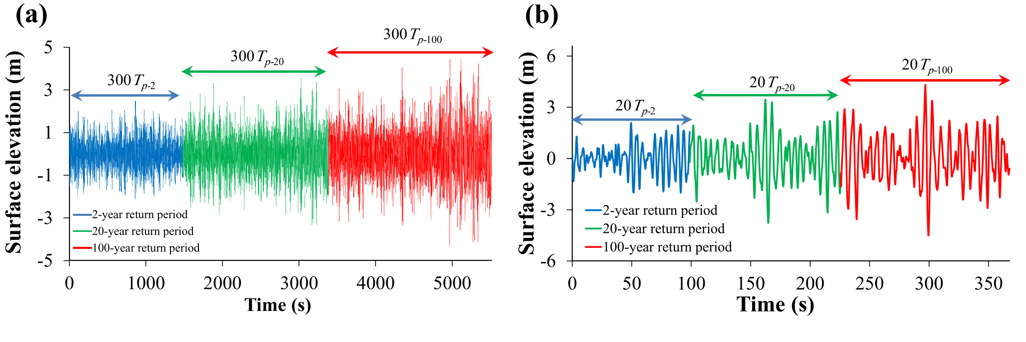

Fig. 7. Sample time history of (a) MIRSE and (b) MICNW.

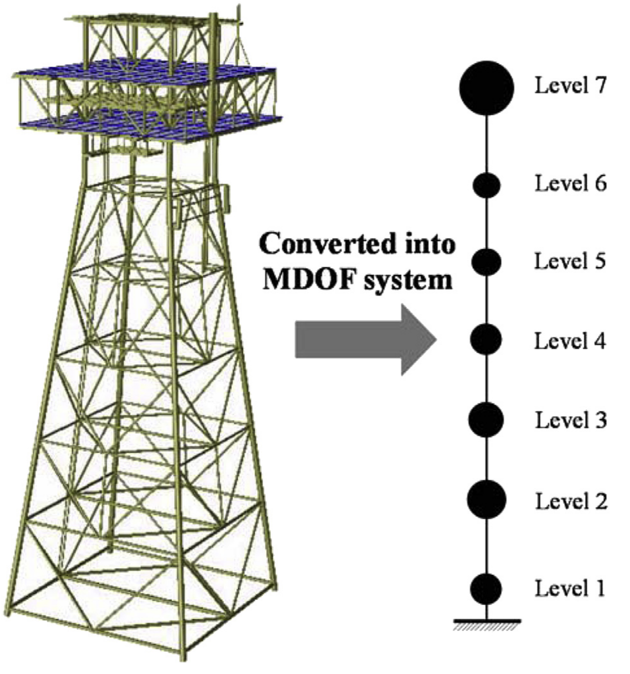

Fig. 8. Ressalat platform and its simplified MDOF system.

where a_{kn} and b_{kn} are independent Gaussian random variables that can be shown as:

$$\begin{aligned} a_{kn} &= rn_a\sqrt{S_n(\omega)d\omega} \\ b_{kn} &= rn_b\sqrt{S_n(\omega)d\omega} \end{aligned} \tag{9}$$

that rn_a and rn_b are standardized normally distributed random variables. As mentioned before, 20 is assumed as an adequate value for TDF_p in the constrained new-wave theory. Same as the former example, Fig. 7(b) shows a sample of MICNW for 3 different return periods (2, 20 and 100 years).

5. Practical case study

5.1. Structural model

Complete modeling of the whole realistic structure is a rigorous way for measuring the response with detailed information. But due to the complicated geometries and time consuming analyses of the platforms, many researchers have been used the simplified MDOF system instead [23–27]. This approximate model cannot evaluate the response of structural components, but it is acceptably well in

Table 2
Specifications of the Ressalat platform.

Ressalat platform	Level 1	Level 2	Level 3	Level 4	Level 5	Level 6	Level 7
Mass (ton)	105.89	129.20	116.25	105.30	91.93	63.13	1790.04
Stiffness (MN/m)	179.00	145.58	146.25	120.88	106.26	89.63	37.73
Volume (m^3)	134	134	117	113	103	22	0
Cross-sectional area (m^2)	227	238	213	209	191	35	0

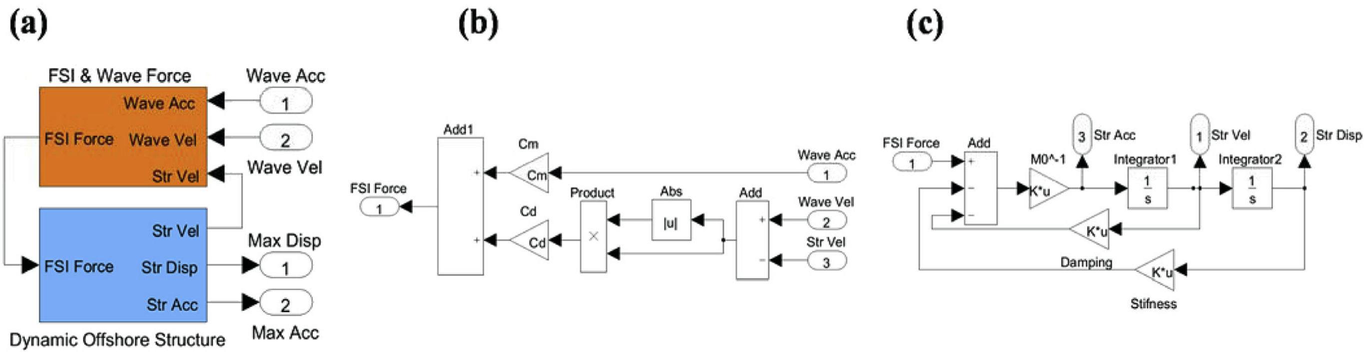

Fig. 9. MATLAB SimuLink blocks (a) SimuLink model (b) "FSI & Wave Force" block (c) "Dynamic Offshore Structure" block.

estimation of overall performance. Moreover, in this manner, the computational cost is significantly low in comparison with detailed model.

For explanation and evaluation of MEWA methodology, the Ressalat platform has been studied as a practical case study shown in Fig. 8. This structure is a four-legged platform that is located in 68.2 m depth of the Persian Gulf. This platform is modeled as a 7-degree of freedom model in which the periods of first three modes are 2.355, 0.502 and 0.243 s, respectively. It is also assumed that the information of the structure is completely deterministic; and the source of the uncertainties is only the wave data.

In here, structural behavior of the jacket is studied in the linear range. The lumped masses and stiffnesses are obtained according to the properties of the real structure in a way that the natural periods to be similar to the detailed model. Related values are given in Table 2. The damping matrix of the structure has been considered as the Rayleigh damping assuming 2% damping ratio for the first and second modes.

For this model, the equation of motion of an offshore structure subjected to wave loading can be expressed as:

$$M_0\ddot{X} + C\dot{X} + KX = F_I + F_D \tag{10}$$

where M_0, C and K are the mass, damping and stiffness matrixes, respectively. Also, X, $\dot{X}$ and $\ddot{X}$ are displacement, velocity and acceleration of the structure, respectively. F_I and F_D are inertia and drag forces determined from the Morison equation and can be written as:

$$\begin{aligned} F_I &= \rho C_m V\dot{U} \quad \text{: Inertia force term} \\ F_D &= \frac{1}{2}\rho C_d A'\left(U - \dot{X}\right)\left|U - \dot{X}\right| \quad \text{: Drag force term} \end{aligned} \tag{11}$$

in which U and $\dot{U}$are the wave particle velocity and acceleration and ρ is the water density that assumed to be 1024 kg/m^3. C_m and C_d are inertia and drag coefficients considered 1.2 and 1.05 according to API

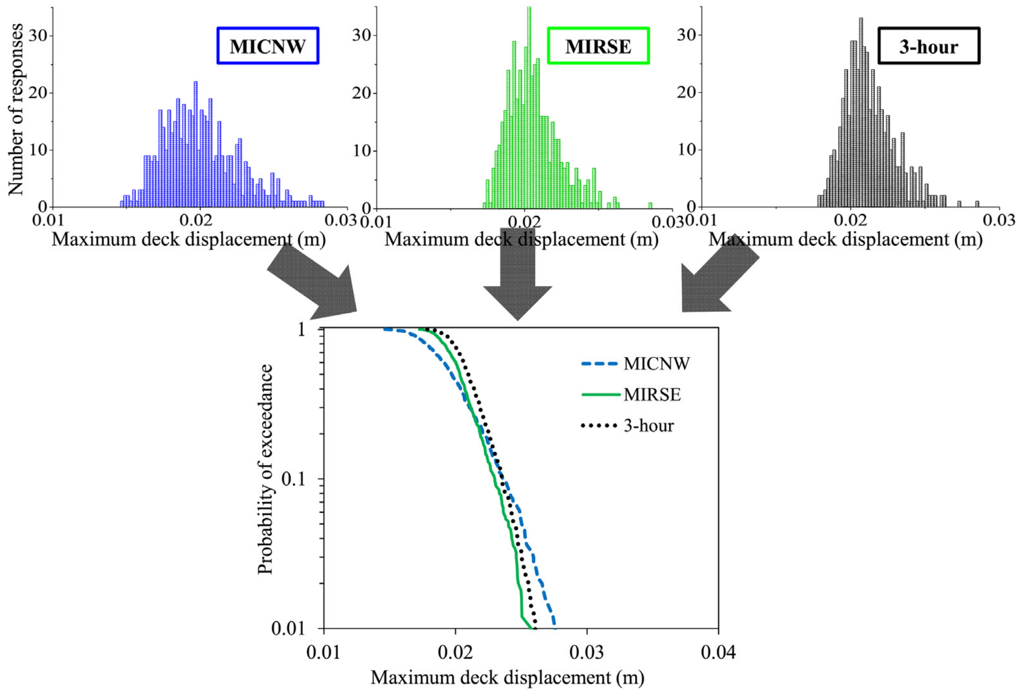

Fig. 10. Extreme deck displacement distribution and its cumulative probability distribution for MICNW, MIRSE and 3-h records (2-year return period).

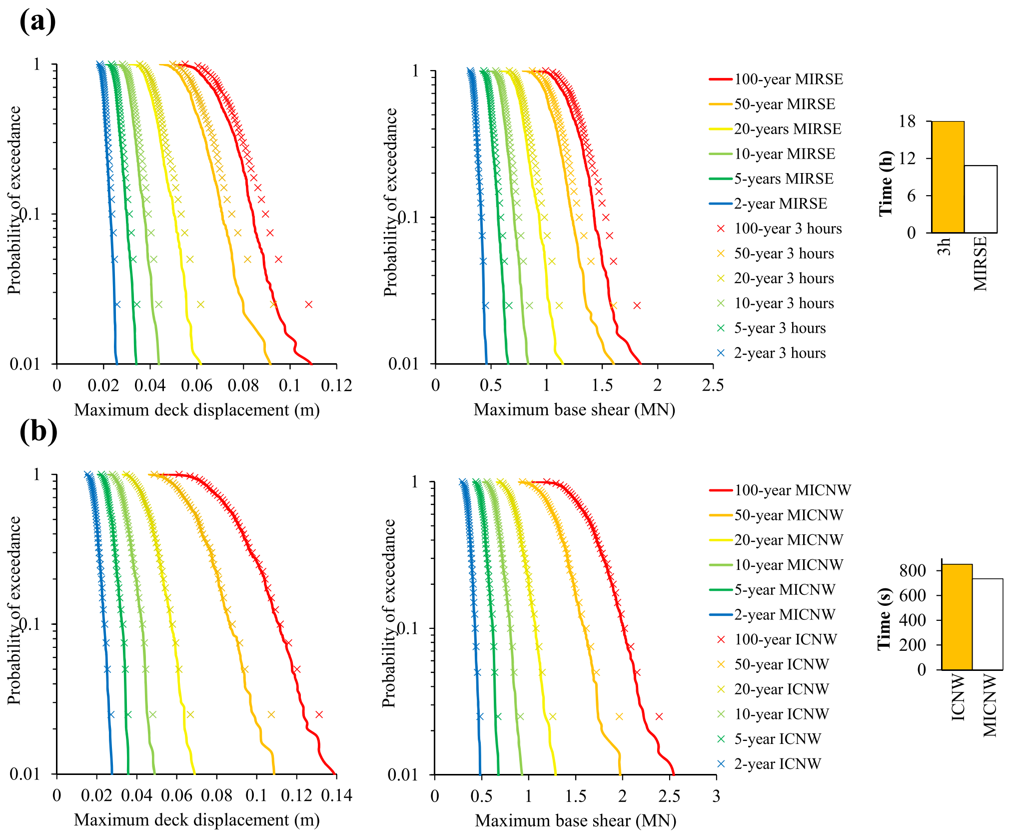

Fig. 11. Comparison of results and total time of analysis (a) MIRSE and 3-h simulation and (b) MICNW and ICNW.

[2], respectively. V and $A'$are the volume and cross-sectional area of the members that is presented in Table 2. The inertia force component is a linear term related to the wave particle acceleration while the drag force component is a nonlinear term depended on both structure and wave particle velocities which causes complexity in the FSI relation.

The mass matrix is summation of the structural mass M and added mass M_a which represented as:

$$M_0 = M + M_a \tag{12}$$

that

$$M_a = \rho(C_m - 1)V\ddot{X} \tag{13}$$

Table 3
Stochastic characteristics of maximum deck displacement under various wave excitations.

3-h simulation			MIRSE			MICNW			Return period
CV (%)	Median (m)	Mean (m)	CV (%)	Median (m)	Mean (m)	CV (%)	Median (m)	Mean (m)	(*n-year*)
7.79	0.021	0.021	8.39	0.020	0.021	13.02	0.020	0.020	2
8.97	0.027	0.027	9.55	0.026	0.026	11.87	0.027	0.028	5
10.39	0.033	0.034	10.85	0.032	0.033	13.07	0.035	0.036	10
12.71	0.044	0.045	12.99	0.042	0.043	15.41	0.047	0.047	20
13.62	0.062	0.064	13.81	0.059	0.061	17.40	0.070	0.071	50
13.73	0.075	0.075	13.86	0.071	0.072	17.56	0.089	0.091	100

Table 4
Stochastic characteristics of maximum base shear under various wave excitations.

3-h simulation			MIRSE			MICNW			Return period (*n-year*)
CV (%)	Median (MN)	Mean (MN)	CV (%)	Median (MN)	Mean (MN)	CV (%)	Median (MN)	Mean (MN)	
8.69	0.371	0.364	8.74	0.359	0.364	11.15	0.365	0.370	2
9.50	0.506	0.515	9.87	0.490	0.498	10.64	0.527	0.532	5
10.38	0.637	0.648	10.80	0.611	0.623	11.84	0.676	0.688	10
11.89	0.811	0.829	12.35	0.780	0.794	13.70	0.895	0.905	20
12.64	1.100	1.116	12.73	1.048	1.066	15.49	1.313	1.325	50
12.07	1.284	1.366	12.86	1.231	1.247	15.55	1.667	1.683	100

In this article, MATLAB SimuLink is applied for solving the nonlinear equation of motion and demonstrated in Fig. 9. Two blocks are shown in Fig. 9(a). "FSI & Wave Force" determines the inertia force and dissolves nonlinear term of the drag force using wave particle acceleration and relative velocity, Fig. 9(b). "Dynamic Offshore Structure" imposes the wave and FSI forces to the structure and solves the dynamic equation of motion, Fig. 9(c). The output of the program is a set of useful results such as time series of displacements and accelerations in all mass levels. In this approach, the nonlinearity of the drag component of Morison's wave loading is thoroughly considered which has a key role in the probabilistic analysis of the response.

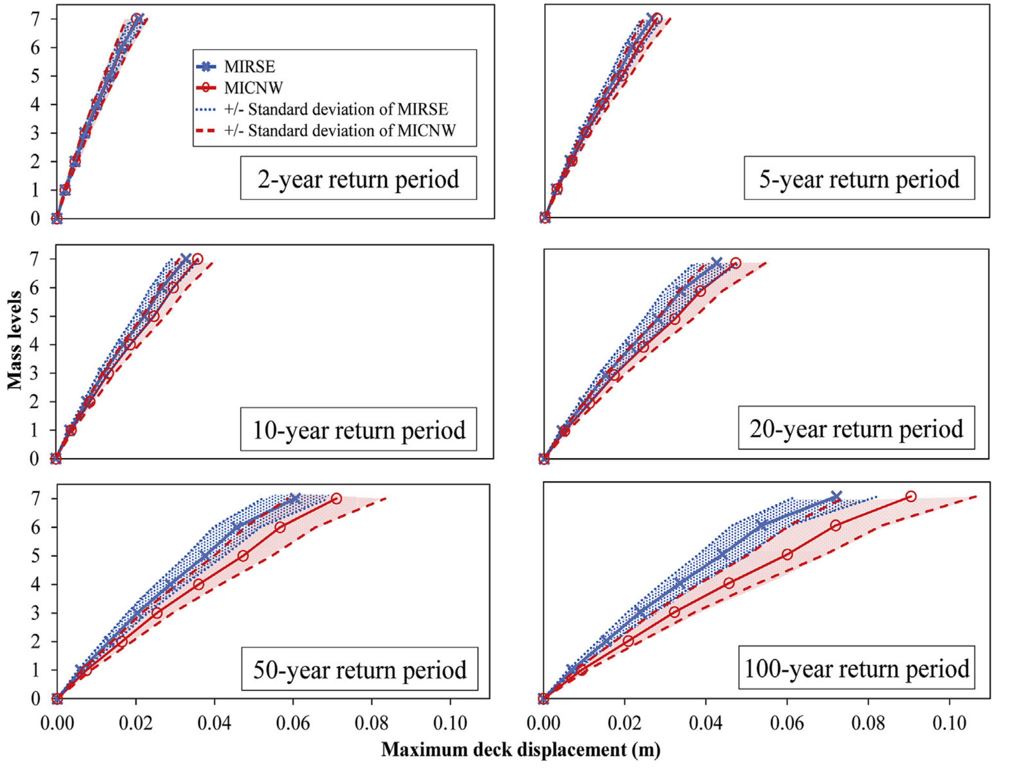

Fig. 12. 50% exceedance of maximum displacements in all mass levels for 3-h, MIRSE and MICNW series.

5.2. Stochastic approach

In this study, by using the MIWTFs, the platform response was evaluated for different return periods of wave loading in the Persian Gulf sea states (Table 1). Due to the random nature of the sea surface, probabilistic investigation of the response is a key issue in the design procedure [28]. Accordingly, for considering the extreme response statistics of MEWA records, 500 profile of MIRSE and MICNW series have been generated. Moreover, for each considered return periods, 500 3-h time history simulations are performed to investigate the performance of the MEWA records. In Fig. 10, histogram of maximum deck displacement based on 2-year return period extreme wave excitation and its probability of exceedance are plotted for MICNW, MIRSE and 3-h records. As seen in this figure, close agreement between results of MIRSE and 3-h records is achieved indicating 1000 T_p, is a reliable interval in this sea state. Moreover, the results of MICNW are totally matched with the results of MIRSE and 3-h records, but MICNW results show wider distribution.

For various return periods, probability of exceedance of maximum deck displacements and base shears are also obtained and shown in Fig. 11. As seen, the more return period increased, the wider ranges of the responses produced. In this case study, with utilizing MEWA method, time duration of the generated MIRSE is about one half of the 3-h simulation while the differences between results are practically acceptable. It is also very informative to visually compare on the same figure the results of ICNW (EWA records) versus MICNW and 3-h simulation versus MIRSE. In Fig. 11(a), the curves show that 3-h simulation and MIRSE results have good agreements even at the tail regions except for extreme waves with 50-year and 100-year return periods in which their peak spectral period is equal to 6.77 and 7.10 s, respectively. As shown in equation (11), the wave forces will, due to the non-linear drag term, cause excitation in both wave period and one-third of the wave period [29]. In here, structural period (2.355 s) is close to one-third of the peak spectral period of these two cases and therefore the distribution of the structural response is very sensitive to the excitation record, especially at the tail regions. This issue is important in the interpretation of the MEWA results.

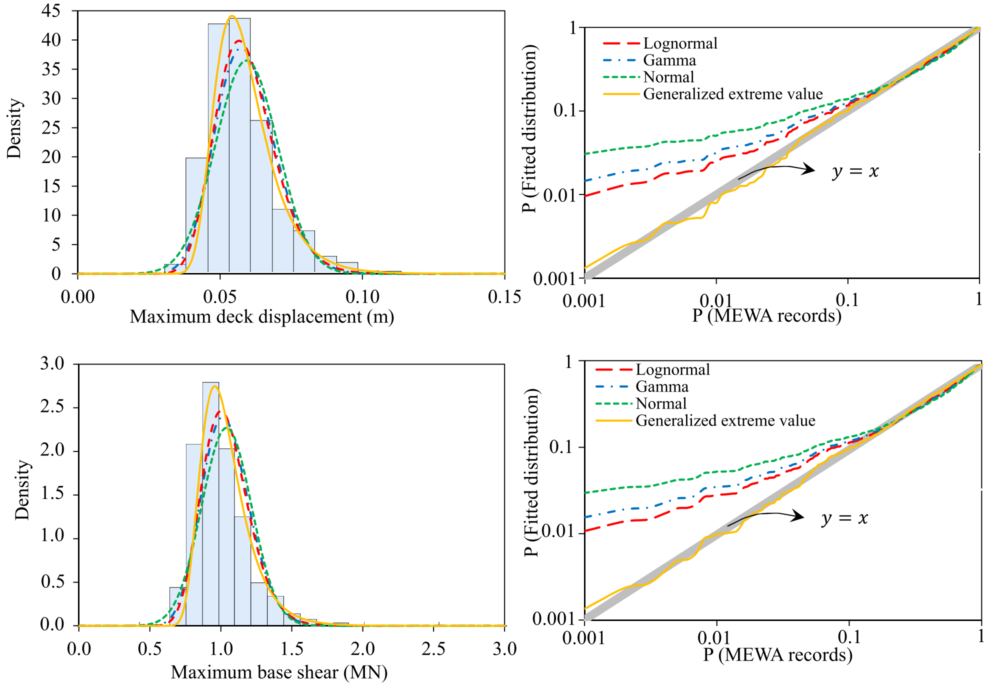

Fig. 13. Different distributions fitted to the sample data from the results of MIRSE records.

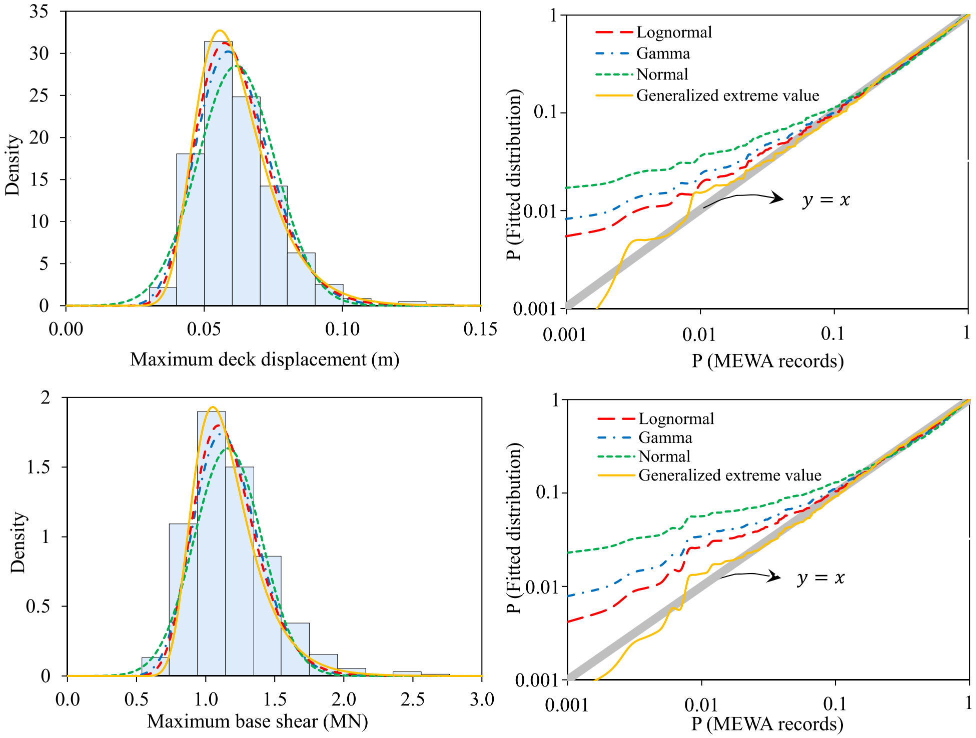

Fig. 14. Different distributions fitted to the sample data from the results of MICNW records.

In Fig. 11(b), the ICNW and MICNW results are also compared. The ICNW records are produced as same as MICNW but with a fixed time duration for all sea states. For achieving same accuracy in the fixed interval state, time durations of the ICNW must be considered equal to or greater than 20 Tp−100 in each excitation level. It can be obviously found out that MICNW results are completely matched with ICNW but the required time of analysis is decreased at least 16%.

The mean, median and CV values of maximum deck displacement and maximum base shear are listed in Tables 3 and 4, respectively. These values increase with increasing the return period due to nature of extreme waves. In the results of maximum deck displacement and maximum base shear, differences of these parameters for MIRSE and 3-h simulations are less than 9%. In all sea states, CV of MICNW results is relatively large indicating these results are more scatter. Mean and median of the MICNW results are also match with other results except for 50-year and 100-year return periods which may be related to the resonance effects.

In Fig. 12, the 50% exceedance of maximum deck displacements in all mass levels together with one standard deviation are depicted for MIRSE and MICNW records. As seen in this figure, the scatter of the results depends on the height of the level, return period of the extreme wave loading and type of the

Table 5
Results of K–S test together with parameters of fitted distributions based on the results of MIRSE records.

Goodness-of-fit order (K–S test)	Distribution	Parameters	
		Maximum deck displacement	Maximum base shear
1	Generalized extreme value	$\zeta = -0.073$, $\sigma = 0.009$, $\mu = 0.068$	$\zeta = 0.070$, $\sigma = 0.137$, $\mu = 1.178$
2	Lognormal	$\mu = -2.634$, $\sigma = 0.134$	$\mu = 14.028$, $\sigma = 0.124$
3	Gamma	$\alpha = 54.734$, $\beta = 0.001$	$\alpha = 63.706$, $\beta = 0.020$
4	Normal	$\mu = 0.072$, $\sigma = 0.010$	$\mu = 1.247$, $\sigma = 0.160$

Table 6
Results of K–S test together with parameters of fitted distributions based on the results of MICNW records.

Goodness-of-fit order (K–S test)	Distribution	Parameters	
		Maximum deck displacement	Maximum base shear
1	Generalized extreme value	$\zeta = -0.090$, $\sigma = 0.014$, $\mu = 0.084$	$\zeta = -0.074$, $\sigma = 0.224$, $\mu = 1.571$
2	Lognormal	$\mu = -2.417$, $\sigma = 0.172$	$\mu = 14.325$, $\sigma = 0.151$
3	Gamma	$\alpha = 33.768$, $\beta = 0.003$	$\alpha = 43.577$, $\beta = 0.039$
4	Normal	$\mu = 0.090$, $\sigma = 0.016$	$\mu = 1.683$, $\sigma = 0.262$

records. Thus, it seems that while MICNW records pose low computational burden in comparison with MIRSE records, their results are more scatter. The displacement trend for each return period is almost linear for all type of records, but since there are greater lumped mass and lower stiffness in 7-th level, the rate of increase in this level is more than the others.

In the stochastic approach, a large number of numerical simulations are required to obtain a reliable distribution curve. On the other hand, if the distribution of the results can be estimated by a known Probability Distribution Function (PDF), required number of simulations will be decrease significantly but the type of PDF is usually unknown. It should be noted that this reduction is applicable when a similar offshore structure has been previously considered in the same extreme wave conditions. For instance, in the Persian Gulf the steel jacket platforms generally have a similar geometry; and therefore, it is expected their response under extreme waves have similar probability distribution function, too. The histogram of maximum deck displacement and base shear are shown in Figs. 13 and 14, respectively. To facilitate interpretation, width of the bins is made equal. Care should be taken in reading the number of bins which can affect the fitting process [30], so following relation is used [31]:

$$m = 1 + \log_2^n \tag{14}$$

where m and n are the number of bins and sample data, respectively. In this study, in order to examine the degree of fitting of different distributions to the sample data from the MEWA records, 4 well-known probability density functions are utilized: GEV, lognormal, gamma and normal distributions. Nonlinear least squares method is used to determine distribution parameters listed in Tables 5 and 6. For comparing the fitness of the distributions, Kolmogorov–Smirnov (K–S) goodness-of-fit test is also performed and corresponding results presented in these tables. It can be concluded that GEV distribution has better fit compare to others.

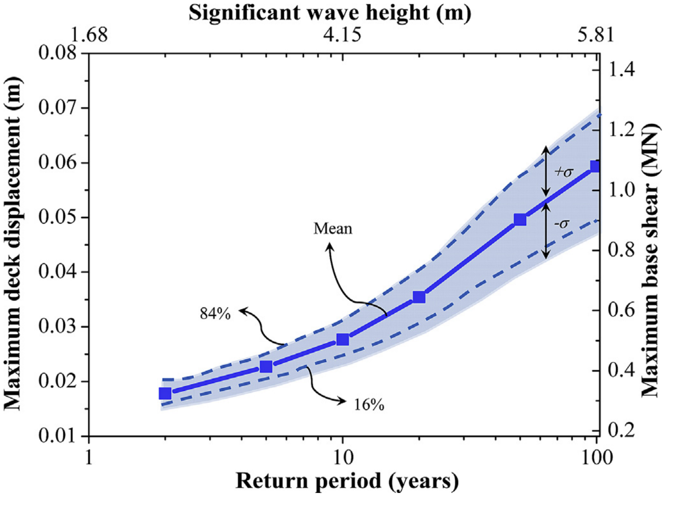

Fig. 15. Stochastic performance curve obtained from MEWA method.

In Figs. 13 and 14, the probability–probability plots (P–P plots) [32] are shown which compare cumulative distribution function of the MEWA results with cumulative distribution function of fitted distributions. P–P plots are used to consider how well the theoretical distributions model the results of MEWA records. It is clear that GEV distribution is appropriate for estimation of the MEWA results.

5.3. Performance curve

Fig. 15 is a reliable curve which can be used in either conventional design or assessment of offshore structures for practical engineers. Different EDPs can be used in this curve. Based on stochastic approach, in addition to mean of the MEWA results, mean plus/minus one standard deviation together with 16% and 84% fractiles are presented in this figure. These curves are obtained based on results of MIRSE records. As an example, for extreme wave excitation with 50-year return period (i.e. significant height equal to 5.33 m), mean, standard deviation, 16% and 84% fractiles of maximum deck displacement are obtained 0.048, 0.009, 0.041 and 0.051, respectively. Accordingly, the response of offshore structure can be estimated with considering the effects of inherent randomness in the response.

5.4. MEWA procedure

Consequently, the general methodology for the assessment procedure by MEWA method is presented in Fig. 16. As shown in this diagram, the first step is to select an appropriate wave theory. Then, the wave characteristics such as H_s and T_p should be identified based on extreme wave conditions of the

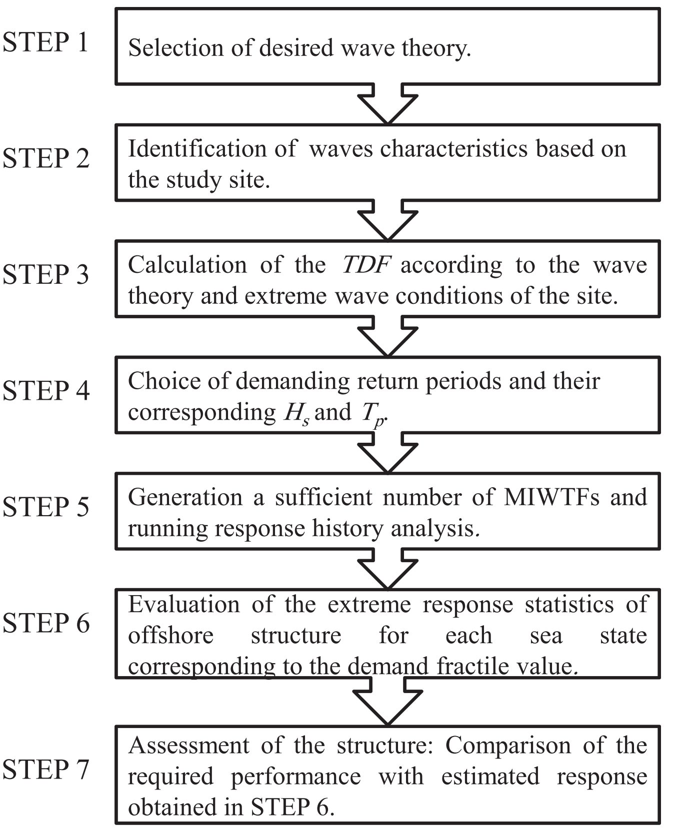

Fig. 16. Recommended procedure of the MEWA method for stochastic estimation of the response of offshore platform.

site. Afterward, *TDF*, a factor of T_p, is estimated to determine the time duration of each sea state. Next, the demanding return periods and their corresponding H_s and T_p are chosen. In the following, a sufficient number of MIWTFs is generated and exerted to the structure as a wave loading profile. Then, a probabilistic approach according to the extreme response of the platform should be carried out and reliable response corresponding to demand fractile value should be estimated. Finally, the desired curves for various parameters such as maximum displacement, base shear or any other EDPs can be drawn and compared with the required performance.

6. Conclusions

In this paper, concepts, design procedures and a practical application for Modified Endurance Wave Analysis (MEWA) were described step by step. The modification of Endurance Wave Analysis (EWA) can be summarized in three parts. First, due to random nature of sea waves, the deterministic method for assessment of offshore structures under extreme waves was replaced by probabilistic approach. In this way, for each sea state, 500 time history records are randomly generated, and the statistical distribution of response is determined. Second, time duration for each sea state was proposed as a factor of peak spectral period instead of a fixed value. This modification considers the effects of significant frequencies of the power spectral density and reduces total time of the records. This factor was introduced as Time Duration Factor (*TDF*) which was considered for random wave and constrained new-wave theories. Third, the increasing trend of the wave height was modified relying on demanding return periods. Thus, application of the modified records is more convenient and reliable for practical engineering in assessment of offshore structures under extreme waves.

A comparative study between the distribution of the responses of MEWA, EWA and 3-h simulation indicated that MEWA method can accurately estimate the structural performance, except in the resonance state. It is concluded that distribution of the results of 3-h and MIRSE records are very similar, but results of MICNW records are more scatter. It is also shown that GEV distribution provides a good fit with MEWA results (both results of MIRSE and MICNW records). The reduction in total time of analysis also brings forward MEWA as an optimal time saving method. In addition, a performance curve obtained from MEWA method illustrated that this approach can be utilized in performance-based design procedure. It should be noted that all the results and conclusions were satisfied in linear elastic state and the studied site. Further researches should be done for considering the effects of material nonlinearity in application of the MEWA method.

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