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An efficient nonlinear dynamic approach for calculating wave induced fatigue damage of offshore structures and its industrial applications for lifetime extension

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ABSTRACT

Because traditional frequency domain analysis can hardly properly handle nonlinear effects induced from many sources, based on a nonlinear time domain dynamic analysis approach, the current paper presents a general calculation procedure and comparison for wave induced fatigue damage of a typical jacket structure under two hydrodynamic coefficient specification systems. Compared with the previous version of Norsok Standard N-003 [Norsok Standard. Actions and action effects, N-003. Oslo: Norwegian Technology Standards Institution; 1999], the inertia coefficients adopted by the latest Norsok Standard [Norsok Standard. Actions and action effects, N-003. Oslo: Norwegian Technology Standards Institution; 2007] released in 2007 are significantly decreased while the drag coefficients are slightly increased. The current paper shows that, for fixed offshore structures such as jackets, this change may result in a significantly increased calculated fatigue life. This may indicate that offshore jacket structures currently in service may be allowed a significant life time extension with regard to fatigue. In addition, the influence of the bottom support conditions of the jacket structure on the fatigue life is also investigated. The investigation indicates that, in cases where the pile-soil and conductor soil stiffness data are not available and the structure is high enough, for a rough estimation of the fatigue life on the upper part of a jacket, the jacket's bottom support condition may be simply modelled as fully fixed. By comparing the fatigue life from dynamic analysis with that from static analysis without taking the inertia effects of the structure, equipment mass and other non-structural installations into account, it also shows the significance of the contribution from the structure's dynamic response. The nonlinear dynamic analysis method presented in the current paper has been successfully applied in several industry projects for the life time extension of offshore installations subject to wave and wind loads.

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1. Introduction

Offshore structures are designed to resist continual wave loading which may lead to significant fatigue damage on individual structural members, and other types of loads due to severe storms, corrosion, fire and explosion etc. Fatigue life is one of the major concerns for the offshore installations since the utilization of tubular members gives rise to significantly high stress concentrations in the joints. Most steel offshore support structures are three-dimensional frames fabricated from tubular steel members. This gives the best compromise in satisfying the requirements of low drag coefficient, high buoyancy and high strength to weight ratio [3]. The most common used offshore structure is a jacket structure, which comprises a prefabricated steel support structure (jacket) extended from the sea bed (connected with piles at the sea bed) to some height above the water surface level, and a steel deck (topside) on the top of the jacket. It is reported by Stacey and Sharp [4] that fatigue cracking has been a principal cause of damage to North Sea structures and in-service experience has witnessed several incidents of fatigue cracking requiring repair. In some cases, fatigue cracking has led to member severance, resulting in a consequent reduction in overall structural integrity. Wave-induced dynamic force is one of the most significant forces leading to fatigue of offshore tubular structures.

Different aspects of research on the dynamic response of fixed offshore structures have been reported. Based on the spectrum analysis theory, Gong, He and Jin [5] presented a fatigue calculation of a tubular jacket structure located in South China Sea. By investigating the transfer function of joint stresses, it is concluded that the first order mode provides a primary contribution to the dynamic response, and an appropriate selection of frequency and bandwidth has remarkable effect on the structural response. Etube, Brennan and Dover [6] presented a modelling of jack-up

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response for fatigue calculation. By analysing a mathematical model to obtain the transfer function of the dynamic response for a typical jack-up platform, they found out that the complex leg-soil interaction can be adequately modelled using springs and assuming a rigid foundation, The effect of translational spring is insignificant and the leg-soil connection can be satisfactorily treated as pinned. Moreover, the jack-up response is very sensitive to water depth and as a result, it is very difficult to produce any single load history that will be representative of service loading conditions for jack-ups operating in different water depth. Kawano and Venkataramana [7] have studied the response of a tuned mass damper installed and conclude that this damper is rather effective in decreasing the dynamic response due to wave loading. Wang [8] has studied the effectiveness of the lateral vibration control of Magnetorheological dampers due to wave loading. Patil and Jangid [9] have investigated wave-induced vibrations by adding viscoelastic, viscous and friction dampers into a jacket structure and found that viscoelastic dampers can modify the fundamental frequency of the jacket-damper system through changing the lateral stiffness of the jacket, while the viscosity and friction can only add damping to the structure.

It should be noted that frequency domain analysis cannot properly deal with the nonlinear load effects induced from Morison's equation, the variation of the water surface causing the intermittency of the wave loading and variation of buoyancy forces on members in the splash zone and large structural deformation into account; also because the power spectrum of the critical stress due to the dynamic wave loading may not be narrow banded. In the current study, for more accurate fatigue calculations, the time domain stress history is calculated for each wave load case with respect to direction, period and wave height. The number of cycles presented in the simulated time histories is then obtained by a rainflow counting algorithm. Miner's rule is therefore adopted to calculate the fatigue damage under each wave loading case. By adopting hydrodynamic coefficients specified by the original [1] and current [2] version Norsok Standard N-003 and applying the time domain analysis approach aforementioned on the fatigue calculation of a typical jacket structure, it is concluded that, for fixed offshore structures such as jackets, this change of hydrodynamic coefficients may result in a significantly increased calculated fatigue life. This may indicate that offshore jacket structures currently in service may be allowed a significant life time extension with regard to fatigue. In addition, the influence of the bottom support conditions of the jacket structure on fatigue life is also investigated. Analysis gives hints that, in case the data of pile-soil and conductor soil stiffness are not available, a rough estimation of fatigue life on the upper part of a jacket, and a jacket's bottom support condition may be simply modelled as fully fixed. By ignoring the inertia effects of the structure, equipment mass and other non-structural installations, and calculate the fatigue life based on a static analysis, the current paper also investigates how significant the dynamic response on the structure's fatigue life is compared with their static counterpart.

2. Description of the adopted wave spectrum

The wave environment comprises sea states, which are random processes described by a random wave model using a wave spectrum. The model may be visualised as the summation of a large number of periodic wavelets; each of these wavelets has its own direction, amplitude and frequency.

A typical wave energy expression in a frequency domain has the form as expressed in Eq. (1):

$$S_{PM}(\omega) = \frac{A}{\omega^5} \mathrm{e}^{-\frac{D}{\omega^4}}. \qquad (1)$$

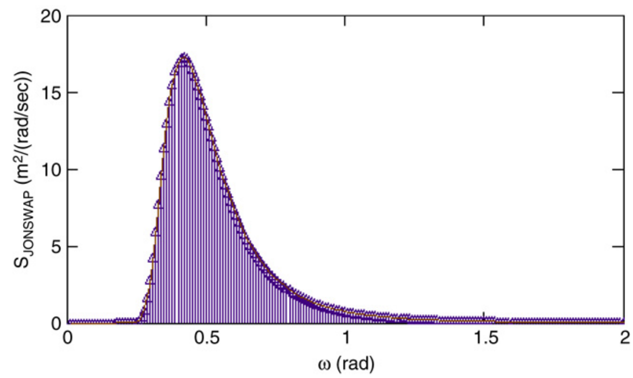

Fig. 1. Illustration of JONSWAP spectra and the sampling frequency with the $N = 64$ samples (Δ) per rad.

A and D are two coefficients determined by wave characteristics. ω is the angular frequency of the wave. Pierson–Moskowits (PM) spectrum can be used for the fully developed wave condition, i.e. the fetch and the duration are large and there is no disturbance from other areas. This is due to the fact that after a certain period of wind blowing, the sea elevation becomes statistically stable. It is mainly developed for the description of waves in North Atlantic Ocean. It can be expressed using the significant wave height $\overline{H_{1/3}}$ and zero crossing mean period $\overline{T_0}$ as independent parameters and input into the typical wave energy expression (Eq. (1)) as expressed in Eqs. (2) and (3):

$$A = 123.95\overline{H_{1/3}}^2/\overline{T_0}^4 \qquad (2)$$

$$D = 495.8/\overline{T_0}^4. \qquad (3)$$

In shallow waters with limited fetch and for extreme wave conditions, JONSWAP spectrum developed by the Joint North Sea Wave Project can be adopted. Compare to PM spectrum, JONASWAP spectrum is narrow banded and extensively adopted by offshore industry. It is expressed by enhancing the peak of the PM spectrum as shown in Eq. (4) and Fig. 1,

$$S_{\mathrm{JONSWAP}}(\omega) = \frac{A}{\omega^5} \mathrm{e}^{-\frac{D}{\omega^4}} \cdot \gamma^\delta \qquad (4)$$

where $A = a \cdot g^2, D = 1.25 \cdot \omega_m^2, \delta = \mathrm{e}^{-\frac{(\omega-\omega_m)^2}{2\cdot\sigma^2\cdot\omega_m^2}}, g = 9.8\ \mathrm{m/s^2}$.

A, D and γ are functions of $\overline{H_{1/3}}$ and mean zero crossing period of waves. a represents the level of high frequency tail, g is the acceleration of gravity. The JONSWAP spectrum has five free parameters: a, ω_m, γ, σ_a, and σ_b. a is taken as 1 in the current study. ω_m is the peak angular frequency of the wave spectrum. The γ value indicates the enhancement of the spectrum peak, normally taken as 2 by standards in offshore designs. σ represents the narrowness of the peak, and has a different value for frequencies lower (σ_a) and higher (σ_b) than the peak frequency ω_m as expressed in Eq. (5):

$$\sigma = \begin{cases} \sigma_b = 0.09, & \omega > \omega_m \\ \sigma_a = 0.07, & \omega < \omega_m. \end{cases} \qquad (5)$$

By discretising the wave energy spectrum into N number of components $S(\omega_n) \cdot \Delta\omega$ between $\omega_n - \Delta\omega/2$ and $\omega_n + \Delta\omega/2$, the relation between the wave energy spectrum and the amplitude of wave components can be approximated by Eq. (6):

$$a_n = \sqrt{2S(\omega_n)\Delta\omega} \qquad (6)$$

where $n = 1, 2, \ldots, N$.

Provided that $\Delta\omega$ is constant, the wave elevation will be repeated with a period of $2\pi/\Delta\omega$. In order to increase this period,

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