TECHNICAL NOTE

DESIGN AND ANALYSIS OF FIXED OFFSHORE STRUCTURE – AN OVERVIEW

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Abstract: Construction of offshore structures is more complex than onshore structures in terms of structural response and its loading system. The high dependency on the environment with high level of uncertainties is the main factor that contributes to the complexity of design and construction process. Better understanding on the type of structures and load exposed to it are required to maintain the high integrity of the operation life. An overview of the design and analysis of offshore structure in the case of fixed platform will be discussed in this paper. It comprises of the fundamental principle of wave dynamic, the dominant load acting on the structure, method used to quantify the responses and probability of failure that should be considered at the design stage. It is mainly to provide understanding and guideline for the purpose of the design phase. Description in details can be obtained through further reading based on references that have been cited in the content.

Keywords: *Probability of Failure, Morison's equation, offshore structure, probabilistic analysis, wave kinematics*

1.0 Introduction

The oil and gas industry is rising rapidly in the worldwide since 50 years ago. The offshore structure used for exploration and production of oil and gas have been extended from shallow to deep water. The common design life for the offshore structure is between 15-25 years, depending on the capacity of the reservoir. The offshore structure can be classified into two types; fixed and floating (Chakrabarti, 2005). The analysis and design of fixed offshore structure have three main phases which are feasibility studies, preliminary design and final design (Chakrabarti, 2005). All the analysis and design procedure must be referred to the recommendation by the American Petroleum Institute (2007) or the International Organization for Standardization (2007).

Generally, the installation of the offshore structure depends on the water depth such as fixed structure usually used in shallow water. When the water depth increase, the compliant structure which is a bounded-structure are more practical for water depth between 450m to 900m (refer Figure 1). For deeper water, the floating structure such as spar and semi-submersible are more suitable.

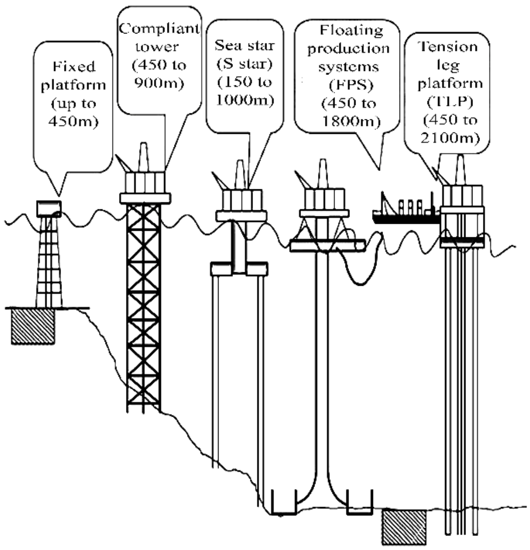


Figure 1: Types of offshore platform (El-Reedy, 2012)

During the operation, the offshore structure is exposed to several types of environmental loads such as wave, wind and current. In order to use the Morison's equation for computing the wave load, the appropriate wave theory is required to determine the water particle kinematics such as linear wave theory, Stokes theory, Cnoidal theory, etc. (Chakrabarti, 2005; Wilson, 2003; Chandrasekaran, 2015).

In reality, the dominant load is commonly due to the wind-generated random wave that can be achieved by using probabilistic method (Najafian & Burrows, 1994). Therefore, the capability to predict the extreme offshore response during the service life will be a very significant to the designer.

2.0 Ocean Waves

Ocean waves are very descriptive as seen in nature. It is often represented by its surface profile and also the motion beneath its surface. Ocean waves on the surface are primarily generated by winds, it is the fundamental feature of the ocean hydrodynamics. Such surface gravity waves are describable by either deterministic or probabilistic approach (Wilson 2003). Estimation of short-term wave features is often addressed by the deterministic approach. Probabilistic approach contemplates on the statistical uncertainty of random waves. It gives a better representation of long-term wave's features on wave spectra.

Deterministic is further categorized as analytic and numeric as illustrated in Figure 2. The analytical description is based on the approximation of power series of the velocity potential. It comprises of linear theory and nonlinear theories. Numerical description solves Laplace equation by using finite difference, finite element or boundary integral with adequate boundary conditions. Both of the descriptions, however are incomplete solutions to the wave boundary problem whereas at some point, abbreviated solution may be necessary (Sorensen, 2005).

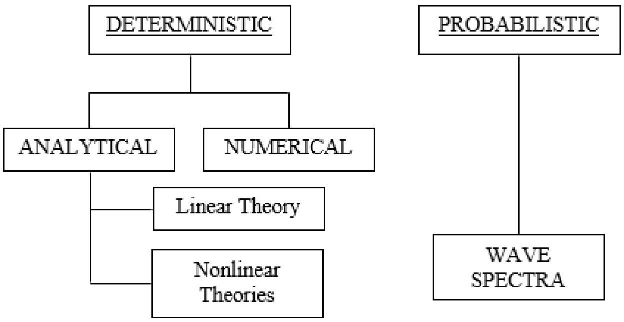


Figure 2: Methods for describing surface gravity waves (Wilson, 2003)

2.1 Water Wave Theories

Wave theory is developed by solving a boundary value problem (BVP) which consist of a differential equation and appropriate boundary conditions through an approximation. For a simple wave theory, a simple solution to the differential equations are present, however as it become more evident, there is no general solution existed to solve the complete BVP.

There are two approximation theories; the one that is developed with the wave height as a perturbation parameter such as in deep water and the one that is developed as a function of water depth as in shallow water (Chakrabarti, 1987). Water wave theories are developed by assuming that water is incompressible and continuity flow is assumed. Thus the conservation of volume can be expressed as the fundamental differential of wave motion.

$$\frac{\partial u}{\partial x} + \frac{\partial v}{\partial y} + \frac{\partial w}{\partial z} = 0 \quad (1)$$

where u, v, and w are three components of fluid particle velocity in the partial derivative with respect to arguments.

In any water wave theory, potential function Φ is hard to determine as it must satisfy Laplace (Eq. 2) and the other three boundary conditions; bottom boundary conditions, free surface kinematics and free surface dynamics conditions. On the other hand, the solution to a complete BVP is also restricted by the nonlinear free surface boundary which keep on changing.

$$\frac{\partial^2 \Phi}{\partial x^2} + \frac{\partial^2 \Phi}{\partial y^2} = 0 \quad (2)$$

In general, all water wave theories begin with the assumption that waves are regular in character (Chakrabarti, 2005). However, there are actually great variety types of water waves, yet no unique solution apparently existed to describe all of them (Le Méhauté, 1976). Hence, a number of water wave theories have been developed ranging from linear theory to nonlinear theories.

2.1.1 Linear Wave Theory

Linear wave theory is a first-order theory which estimates a sensible wave kinematics for small amplitude waves. It assumes that wave height is smaller than the wave length or water depth. It affords a simple basis for probabilistic analysis of forming a wave spectral description. In this theory, the instantaneous free surface is described by the superposition of a series of regular waves, while each harmonic component differs by amplitude frequency, phase, direction and speed.

Surface elevation $\eta(x,t)$ of the small-amplitude, a waves at instantaneous time, t and horizontal position, x is given by:

$$\eta(x,t) = \frac{H}{2} \cos(kx - \omega t) \quad (3)$$

where the wave height, $H = 2a$ in which a represent wave amplitude, wave number, $k = 2\pi/L$ in which L is the wave length and angular frequency $\omega = 2\pi/T$ in which T is the wave period.

For a finite depth, the horizontal velocity $u(x,t)$ and acceleration $\dot{u}(x,t)$ at given position from the mean water level (MWL) in specified water depth, d are given by:

$$u(x,t) = \frac{\pi H}{T} \frac{\cos k(d+z)}{\cos kd} \cos(kx - \omega t) \tag{4}$$

$$\dot{u}(x,t) = \frac{2\pi^2 H}{T^2} \frac{\cos k(d+z)}{\cos kd} \sin(kx - \omega t) \tag{5}$$

The dispersion relationship in term of wave number, k and angular frequency, ω is given by:

$$\omega^2 = gk \tanh(kd) \tag{6}$$

where g is the gravitational acceleration.

On the other hand, the following expression are valid for deep water condition:

$$u(x,t) = \frac{\pi H}{T} \exp(kz) \cos(kx - \omega t) \tag{7}$$

$$\dot{u}(x,t) = \frac{2\pi^2 H}{T^2} \exp(kz) \cos(kx - \omega t) \tag{8}$$

$$\omega^2 = gk \tag{9}$$

Simplification may be made on the dispersion relation for a shallow or deep water depth. Since the hyperbolic function $\tanh(kd)$ can takes on simpler approximation form, the deep and shallow water criterion can be established. The wavelength can be computed as the following (Chakrabarti, 1987).

Table 1: Water depth criterion and wavelength

Approximation	Criterion	Wavelength
Deep water	$\frac{d}{L} > \frac{1}{2}$	$L_0 = gT^2/2\pi$ (10)
Shallow water	$\frac{d}{L} < \frac{1}{20}$	$L = T\sqrt{gd}$ (11)
Intermediate water	$\frac{1}{20} < \frac{d}{L} < \frac{1}{2}$	$L = L_0[\tanh(2\pi d/L_0)]^{1/2}$ (12)

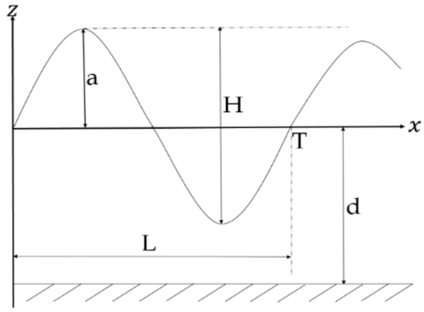


Figure 3: Definition diagram of linear wave theory

2.1.2 Modified Linear Wave Theory

The foregoing linear solution, however, cannot predict sensible kinematics above the MWL. The main difficulty in the study of water wave is indeed the boundary namely the free surface. Hence, a number of engineering approximation for estimating wave kinematics above the MWL have been introduced in the form of empirical modification.

Vertical stretching is the simplest above all. Water particle kinematics below the MWL is calculated from the linear wave theory. From the mean water level, water particle kinematics is stretched vertically by the following relationship:

$$u(x,z,t) = u(x,0,t) \quad z > 0 \tag{13}$$

Wheeler stretching (Wheeler, 1969) is an extension of linear wave theory through linear filtering technique. Wave kinematic profile is mapped from the sea bed to the instantaneous free surface by modifying the depth decay function. The reference elevation is represented by:

$$z_s = (z + d)\frac{d}{\eta + d} - d \tag{14}$$

Linear extrapolation as implied in its name, assumes the vertical partial derivative of wave kinematics to be constant above the MWL and equal to liner wave theory below MWL (Rodenbusch & Forristall, 2013).

$$u(x,z,t) = u(x,0,t) + z\frac{du}{dz}(x,0,t) \quad z > 0 \tag{15}$$

Delta stretching (Rodenbusch & Forristall, 2013) demonstrate interpolation between wheeler stretching and linear extrapolation to reduce errors, and it is only applicable to the crest wave. The stretched elevation is represented as:

$$z_\Delta = (z + d_\Delta)\frac{d_\Delta + \eta_\Delta}{d_\Delta + \eta} - d_\Delta \quad \text{for} \quad z > -d_\Delta \quad \eta > 0 \tag{16}$$
$$\text{otherwise,} \;\; z_\Delta = z$$

where Δ is the delta stretch parameter and d_Δ is the depth above the stretched kinematics profile. If $z_\Delta < 0$, wheeler stretching is used while if $z_\Delta > 0$, linear extrapolation is used. Pure stretching is yielded if $\Delta = 0$ and $d_\Delta = d$, while pure extrapolation if $\Delta = 1$ and $d_\Delta = d$.

However, kinematics predicted from these empirical models are found to vary from one another (Mohd Zaki *et al.*, 2013). Wheeler stretching was found to underestimates the kinematics under the wave crest while linear extrapolation prominently overestimates it (Gudmestad, 1993; Mohd Zaki *et al.*, 2014; Couch & Conte, 1997). The accuracy of each empirical model depends on the wave field characteristics particularly the wave spectrum bandwidth (Zhang *et al.* 1991).

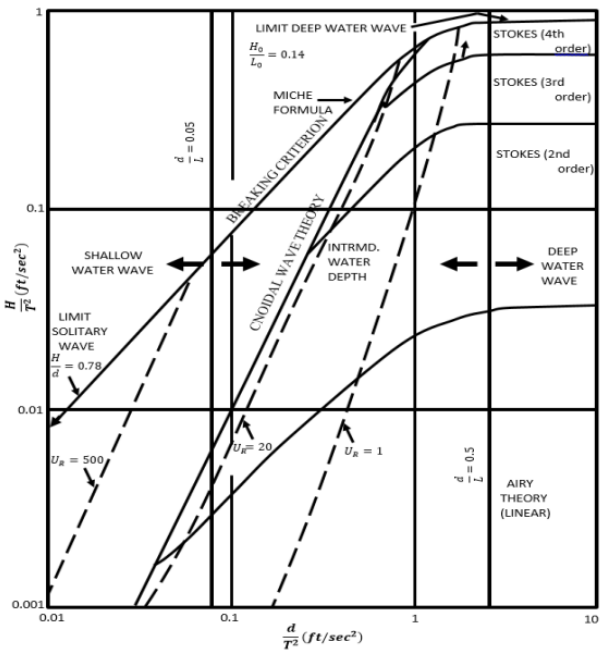


Figure 4: The limits of validity of wave theories (Le Méhauté 1976)

2.1.3 Higher Order Wave Theories

Finite amplitude wave often deviates from a pure sinusoidal. Thus, a simple treatment would not be adequate. The retention of nonlinear terms required more complicated theory. Thus, no unique solution existed for all depth conditions. Some of wave theories that are commonly used in offshore engineering are such as Stokes' Second Order to higher order theory, Cnoidal theory and Stream Function. The limit of validity of these wave theories is illustrated in Figure 4.

2.2 Irregular Waves

Sea state is composed of wave components at varying height and periods that propagates in differing directions. Such an irregular waves exhibit random characteristics and only describable by either statistical or spectral methods. In a wave train analysis, a statistical wave record is developed by using a time-history of the sea surface at a single point while the variability of wave field is considered in terms of probability of individual waves.

For spectral analysis, Fourier transform theory is used to sum up the simple sine waves. It is often defined by spectrum and allow treatment of variability with respect of period and travelling direction. It is by right, the most mathematically appropriate approach (USACE, 2002).

2.2.1 Wave Spectra

Wave spectra are the description of the energy density of random ocean wave over a frequency range. Spectrum is developed from the properties of ocean wave and thus, is empirical. Pierson-Moskowitz spectrum (Pierson & Moskowitz 1964) and JONSWAP spectrum (Hasselmann *et al*., 1973) are two most common wave spectrum.

2.2.1.1 Pierson-Moskowitz (P-M) Spectrum

P-M spectrum is the simplest representation of energy distribution with only single independent parameter. The data records used were from British weather ship operating in the North Atlantic. This spectrum is constructed based on fully developed sea state. It is when the wind blew steadily for a long period of time over a large area would then come into equilibrium with the ocean wave. The fully developed sea state can be represented by:

$$S(\omega) = \frac{\alpha g^2}{\omega^5} exp\left[-\beta\left(\frac{\omega_0}{\omega}\right)^4\right] \tag{17}$$

where $\omega = 2\pi f$, f is the frequency (Hz), $\alpha = 8.1 \times 10^{-3}$, $\beta = 0.74$, $\omega_0 = g/U_{19.5}$ and $U_{19.5}$ is the wind speed at 19.5m height above the sea surface (Stewart 2008).

2.2.1.2 JONSWAP Spectrum.

It is the extension of P-M spectrum which held on five parameters, the modification is made based on Eq. 17. The data record used were collected for a relatively light wind conditions but at higher wind velocities.

$$S(\omega) = \frac{\alpha g^2}{\omega^5} exp\left[-\frac{5}{4}\left(\frac{\omega_0}{\omega}\right)^4\right]\gamma^r \tag{18}$$

$$r = exp\left[-\frac{\left(\omega - \omega_p\right)^2}{2\sigma^2\omega_p^2}\right]$$

where $\alpha = 0.076\left(U_{10}^2/F_g\right)^2$, $\omega_p = 22(g^2/U_{10}F)^{1/3}$, $\gamma = 3.3$, $\sigma = 0.07$ for $\omega \leq \omega_p$ or $\sigma = 0.09$ for $\omega > \omega_p$ and F is the fetch distance (Stewart 2008).

In general, JONSWAP spectrum is similar to PM-spectrum except that its peak is more prominent and its wave continues to grow with distance.

2.2.2 Linear Random Wave Theory

2.2.2.1 Unidirectional Sea Spectrum.

For a linear system, frequency domain which is based on single wave spectrum is practical. However, for a nonlinear system, design of offshore structure should be based on time domain tools. In a time-domain analysis, time history of an ocean wave is needed. It is computed from the foregoing spectrum model. The wave height is derived from the formula:

$$H(f_1) = 2\sqrt{2S(f_1)\Delta f} \tag{19}$$

where $S(f_1)$ is the mean amplitude of the spectral density within the frequency interval Δf.

Thus, the random phased is assigned by a pair of the random generator to retain time history randomness. The time history wave profile in unidirectional sea can be obtained by;

$$\eta(x,t) = \sum_{i=1}^{N} \frac{H_i}{2} \cos(k_i x - 2\pi f_i t + \varphi_i) \tag{20}$$

where the entire spectrum is distributed into N frequency interval Δf and φ is the random phase angle which lies within the range $0 < \varphi < 2\pi$.

2.2.2.2 Directional Sea Spectrum

In the other hand, the simulation of the directional sea is similar to the unidirectional waves. It differs in three-dimensional spectral density. The general form is:

$$\eta(x,t) = \sum_{i=1}^{N} \frac{H_i}{2} \cos(k_i(x\cos\theta_i + y\sin\theta_i) - \omega_i t + \varphi_i) \tag{21}$$

where the wave height includes the spreading angle increment $\Delta\theta$ such as:

$$H(\omega) = 2\sqrt{2S(\omega)D(\omega,\theta)\Delta\omega\Delta\theta} \tag{22}$$

3.0 Loads on Offshore Structure

Loads on offshore structure can be categorized into five component which are; permanent loads or dead loads, operating loads or live loads, environmental loads, construction and installation loads and accidental loads

Permanent and operating loads are the main criteria in the design of the onshore structure. However, for the offshore structure, the design is dominated by environmental loads, especially wave (Chandrasekaran, 2015; Nallayarasu, 1981). The environmental load can be steady, which arise from the wind and current or oscillating due to fluctuating of structure motion and waves (Chakrabarti; 2005).

3.1 Wind Load

The natural wind has two component; mean wind component which is a static and fluctuating component which is dynamic (Nallayarasu, 1981). However, for offshore location, mean wind is much greater than the fluctuating component. Thus, the wind load acting on the offshore structure can be determined using empirical formulas which depend on mean wind velocity and geometry of member of the structure exposed to the wind (Chakrabarti, 2005). Force will generate on the flat plate of the area (A) that is orthogonal to the flow direction of stream air with constant velocity (v). Hence, the wave-induced force can be computed by:

$$f = \frac{1}{2}\rho_a C_D A v^2 \tag{23}$$

where ρ_a is the air density (1.25kg/m^3) and C_D is the wind drag coefficient with a function of Reynolds number, Re. Table 2 listed the average values of C_D for normal wind approach recommended by API Guidelines (2007). While the mean wind velocity generally taken at an elevation of 10m from the water surface and 10m is called the reference height.

Table 2: Average value of C_D

Items	C_D
Beams	1.5
Sides of buildings	1.5
Cylindrical sections	0.5
Overall projected area of platform	1.0

3.2 Wave Load

Waves are considered as a dynamic load that depend on the geometry of the structure where the elements size is relative to the wavelength and based on the orientation to the wave propagation, the hydrodynamic condition and whether the structure is fixed or floating. Waves force can be determined by using two different methods; diffraction theory, and Morison's approach. Larger structural element (diffraction parameter, $\pi D/L >$ 0.15) experienced wave load under diffraction theory while Morison's equation can be applied to a load acting on smaller and slender structure element.

3.2.1 Morison's Equation

When the structure element (diameter cylinder) is small and slender compared to the wavelength, the incident wave is considered unaffected by the structure (Abu Husain *et al.*, 2013; Deo, 2007). In that state, equation is given by Morison *et al.* (1950) become relevant, given the resulting force on a body in an unsteady viscous flow which combined the effects of water particles velocity and acceleration on the structure;

$$F_i = F_D + F_I$$

$$= \frac{1}{2} C_D \rho D u|u| + C_m \rho \frac{\pi D^2}{4} \dot{u} \tag{24}$$

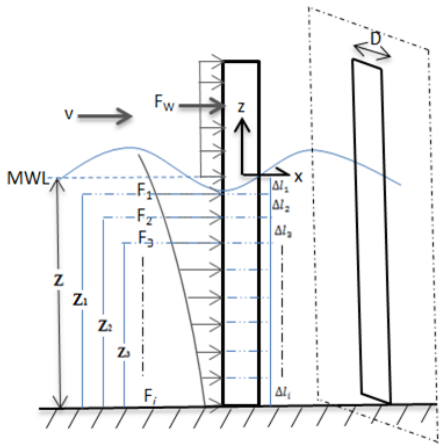


Figure 5: Distribution of forces on structure

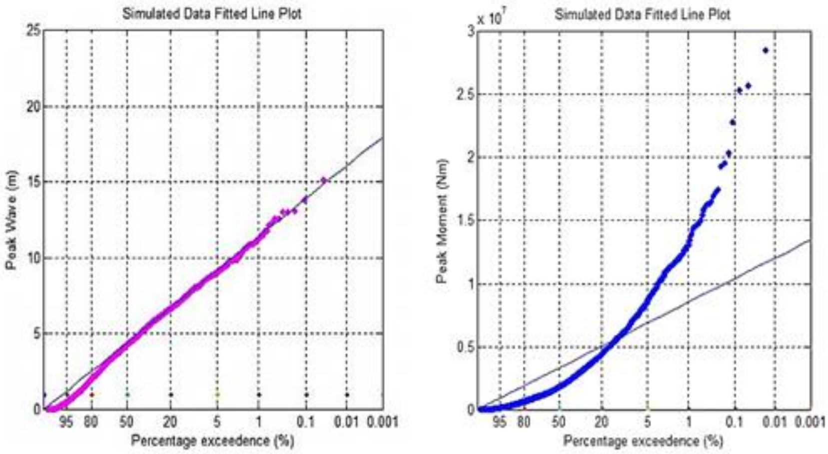
where F_i is Morison's force per meter length at member axis at given time at a given location, F_D and F_I indicates drag and inertia component respectively, D is the diameter of member, ρ is water density (1030kg/m^3), u is water particle velocity, $\dot{u}$ is water particle acceleration, C_D is drag coefficient, and C_m is mass (inertia) coefficient. The distribution of forces on the structure has been shown in Figure 5.

4.0 Response to Irregular Waves

4.1 Drag and Inertia Force

The drag force is caused by the viscous effects which relate to water particle velocity, u and the modulus of velocity ($|u|$) that due to the reverses direction of waves induced water particle after every half cycle. While inertia force is due to water particle acceleration, $\dot{u}$ and if it is dominant, the probability distribution is linearly followed a Gaussian whereas vice versa for the dominant drag component (Figure 6).

As referred to Eq. 24, drag force and inertia force are affected by the drag coefficient, C_D and mass (inertia) coefficient, C_m respectively. Both coefficients which known as hydrodynamic coefficient can be obtained from lab or field experiment as being done by Najafian *et al.* (2000), Konstantinidis *et al.* (2015) and Wolfram & Naghipour (1999). C_d and C_m are functions of size and shape of the structure. For fixed structure, it depends on Keulegan-Carpenter number, Reynold's number and cylinder roughness.



(a) (b)

Figure 6: (a) Gaussian distribution (b) non Gaussian distribution

The inertial force consists of two mechanism; a component due to the pressure waves induced by the unsteady flow and a component due to the added mass (C_a) (Konstantinidis *et al.,* 2015; Journée & Massie, 2001). For potential theoretical flow, $C_m = 1 + C_a$, where C_a depends on the geometry of the cylinder and for a circular shape, C_a=1.0 giving the theoretical value of 2.0.

Table 3: Value of inertia component (Journée & Massie 2001)

Force Component	Experimental coefficient	Theoretical value	Experimental value
Pressure waves induced	1	1	1
Added mass	C_a	1	Usually <1
Inertia	C_m	2	Usually 1 - 2

4.1.1 Keulegan-Carpenter and Reynold's Number

Keulegan-Carpenter number, KC determines the relative contribution of the inertia and drag forces by providing the ratio of maximum drag to maximum inertia and Reynold's number, *Re* is a function of inertia force over viscous force.

$$KC = \frac{u_m T}{D} \quad Re = \frac{u_m D}{v} \tag{25}$$

where u_m is maximum velocity in the wave cycle, T is wave period in sec, v is water particle velocity and D is diameter of cylinder.

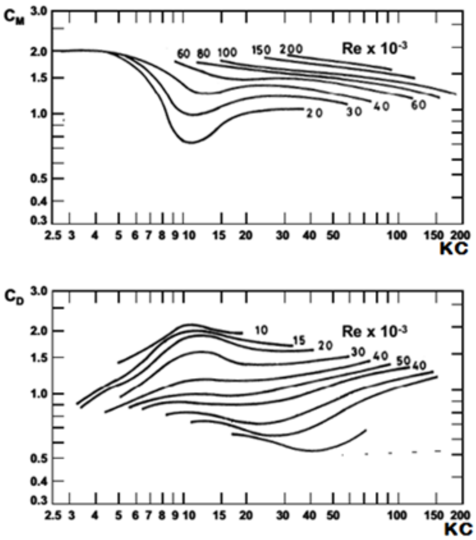


Figure 7: Hydrodynamic coefficient with respect to flow parameter (Chakrabarti 2005)

KC < 10 indicates the dominant of inertia force, while KC > 20 shows dominant of drag force and otherwise express significant dominant for both inertia and drag component. Once the value of KC is larger than 6 (KC > 6), then Morison's equation is sufficient to apply (Chakrabarti, 2005).

Figure **7** shows the relation between KC with inertia and drag coefficient. Both coefficient values lie in the range 0.8 to 2.0. The theoretical value of 2.0 for small KC and it gradually decreases with the increasing of KC value in the drag-inertia regime.

4.1.2 Roughness Factor

Relative surface roughness, *e* of structural member influences the forces on a small structure. It can be determined by the average size of the particles on the surface given by *K* normalized by the equivalent cross-sectional diameter of the structure member.

$$e = K/D \tag{26}$$

where,

e = 0.02 is consider to be very rough.

Roughness normally due to marine growth that change the flow from laminar to turbulent, giving lower Reynolds number and larger friction (Techet, 2004). Due to that, Deo (2007) have shown that C_m does not change much but contribute to a larger C_D from 2 to 3 times more than the initial value.

4.2 Base Shear and Overturning Moment

For the design purpose, it is important to calculate the global structural forces by indicates the maximum base shear, *BS* and maximum overturning moment, *OTM* (Nallayarasu, 1981; Chandrasekaren, 2012) since its considered to be the dominant response due to ocean wave (API 2007). Maximum *BS* is account as a maximum total lateral forces that acting at the centroid of each equally divided segment along it vertical cylinder member. Maximum *OTM* also used the lateral forces acting on the centroid of each segment. However, a further step is required by multiplying the force with the lever arm from mud-line before making a summation.

$$BS = \sum_{i=1}^{NS}[F_i * \Delta l_i] \tag{27}$$

$$OTM = \sum_{i=1}^{NS}[F_i * \Delta l_i * z_i] \tag{28}$$

where *NS* is the number of nodal force, F_i is Morison's force per unit length at node *i*, Δl_i is the length of member associated with node *i*, and z_i is the elevation of node *i* from seabed (refer to Figure 5).

A study carried by Abu Husain *et al*. (2014) and Lambert *et al*. (2013) have applied the maximum *BS* and *OTM* in order to obtain the probability of extreme response for offshore structure.

5.0 Probability of Failure (POF)

Structural system reliability focuses upon issues such as redundancy, robustness with respect to damage and rate of inspection (Azraai *et al*., 2016). Currently, analysis method is available for efficient estimation of the reliability of typical platforms under push over loadings. Structural reliability means simply the field of probabilistic analysis of structural behavior, serviceability and safety (Abu Husain *et al*., 2014).

The Structural Reliability Analysis (SRA) was performed after the push-over analysis to approximate the platform's reliability. An approximate reliability measure of the platform can be established through the determination of the return period of the environmental load which the structure can withstand with the (lowest) calculated RSR.

Probability of Failure (POF) (see Figure 8) is derived when the Load Distribution (base shear) is greater than the Resistance Distribution (RSR). Base shear and RSR derived from the push-over analysis is multiplied by a factor 'Bias' to obtain as accurate result as the mean values.

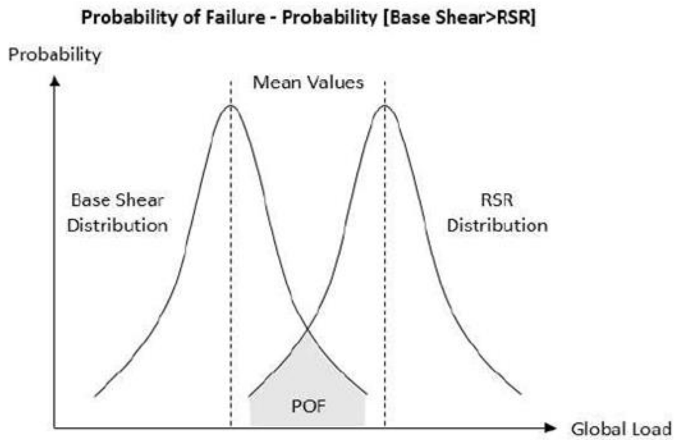


Figure 8: Probability of failure of base shear and RSR distributions

6.0 Summary

This paper has explained the general flow of analysis of offshore structures subject to environmental load. The overview on the analysis and design process of offshore structure can abet designer to understand the fundamental elements related to structure analysis. The emphasis is on the wave load including the selection of wave theory according to hydrodynamic condition, wave spectra to determine the significant wave height, the calculation of wave load using Morrison's equation especially for hydrodynamic transparent element, base shear, overturning moment and the fundamental concept of failure probability in defining the reliability of offshore structure.

7.0 Acknowledgements

This work was supported by the Universiti Teknologi Malaysia [grant no: Q.K130000.2540.09H39 / Q.K130000.2540.09H42] and the Ministry of Higher Education (Malaysia) [grant no: R.K130000.7940.4F584 / R.K130000.7840.4F583].

References

Abu Husain, M. K., Mohd Zaki, N. I., Mallahzadeh, H. and Najafian, G. (2014). *Short-term probability distribution of the extreme values of offshore structural response by an efficient time simulation technique*. Ships Offshore Structural Journal, 10, pp. 1–14.

American Petroleum Institute (2007). *Recommended practice for planning, designing and construction of fixed offshore platform working stress design*, API RP2A-WSD, 21st Edition.

Azraai S.N.A., Lim K.S., Yahaya N. & Noor N.M. (2016) Infill Materials of Epoxy Grout For Pipeline Rehabilitation and Repair, *Malaysian Journal of Civil Engineering 27(1):162-16.*

Chakrabarti, S. K. (1987). *Hydrodynamics of Offshore Structures*, WIT Press.

Chakrabarti, S. K. (2005). *Handbook of Offshore Engineering*. Vol. 1. Amsterdam: Elsevier.

Chandrasekaren, S. (2012). *Ocean Structure and Material: Introduction to design.* Lecture Note, Department of Ocean Engineering, Indian Institute of Technology, Madras.

Chandrasekaran, S. (2015). *Dynamic Analysis and Design of Offshore Structures*. Ocean Engineering & Oceanography. Vol. 5, India: Springer.

Couch, A. T. and Conte, J. P. (1997). *Field Verification of Linear and Nonlinear Hybrid Wave Models for Offshore Tower Response Prediction*, Journal Offshore Mech. Arct. Eng., vol. 119, no. 3, p. 158.

Deo, M. C. (2007). *Waves and Structures*, vol. 2377, p. 2007.

El-Reedy, M. A. (2012). *Chapter 1 - Introduction to Offshore Structures*, in *Offshore Structures*, Gulf Professional Publishing: Boston. p. 1-21.

Gudmestad, O. T. (1993). *Measured and predicted deep water wave kinematics in regular and irregular seas*, Marine Structure, vol. 6, no. 1, pp. 1–73, Jan.

Hasselmann, K., Barnett, T. P., Bouws, E., Carlson, H., Cartwright, D. E., Enke, K., Ewing, J. A., Gienapp, H., Hasselmann, D. E., Kruseman, P., Meerburg, A., Müller, P., Olbers, D. J., Richter, K., Sell, W. and Walden, H. (1973). *Measurements of wind-wave growth and swell decay during the Joint North Sea Wave Project (JONSWAP)*, Deutches Hydrographisches Institut, Jan.

International Organization for Standardization (2007). *Petroleum and Natural Gas Industries – Fixed Steel Offshore Structures*, ISO 19902.

Journée, J. M. J. and Massie, W. W. (2001). *Offshore Hydromechanics, chapter 12 wave forces on slender cylinders*, Offshore Hydromechanics, Delft Univ. Technol., no. January.

Konstantinidis, E., Dedes, A., and Bouris, D. (2015). *Drag and Inertia Coefficients for a Circular Cylinder in a Steady plus Low-Amplitude Oscillatory Flow*, no. October.

Le Méhauté, B. (1976). *An Introduction to Hydrodynamics and Water Waves*. Berlin, Heidelberg: Springer Berlin Heidelberg.

Lambert, L. A., Najafian, G., Cooper, J. E., Abu Husain, M. K. and Mohd Zaki, N. I. (2013). *Efficient Estimation of Offshore Structural Response Based on Treshold Upcrossing Rates*. pp. 1–7.

Mohd Zaki, N. I., Abu Husain, M. K. and Najafian, G. (2013). *Comparison of Extreme Offshore Structural Response from Two Alterna-tive Stretching Techniques*, Open Civ. Eng. J., vol. 7, no. 1, Dec.

Mohd Zaki, N. I., Abu Husain, M. K. and Najafian, G. (2014). *Extreme structural response values from various methods of simulating wave kinematics*, Ships Offshore Structure, pp. 1–16, Dec.

Morison, J. R., Johnson, J. W. and Schaaf, S. A. (1950). *The Force Exerted by Surface Waves on Piles*, J. Pet. Technol., vol. 2, no. 5, pp. 149–154.

Najafian, G. and Burrows, R. (1994). *Probabilistic modelling of quasi-static response of offshore structures subject to nonlinear wave loading: Two approximate approaches*. Applied Ocean Research, **16**(4): p. 205-221.

Najafian, G., Tickell, R. G., Burrows, R. and Bishop, J. R. (2000). *The UK Christchurch Bay compliant cylinder Project: Analysis and interpretation of Morison wave force and response data*, Appl. Ocean Res., vol. 22, no. 3, pp. 129–153.

Nallayarasu S. (1981). *Offshore structures: Analysis and Design*, vol. 41, no. 3, pp. 445–459.

Abu Husain, M. K., Mohd Zaki, N. I. and Najafian, G. (2013). *Derivation of Morison's Force Coefficients: An Application of Method of Moments*, no. September.

Pierson, W. J. and Moskowitz, L. (1964). *A proposed spectral form for fully developed wind seas based on the similarity theory of S. A. Kitaigorodskii*, J. Geophys. Res., vol. 69, no. 24, pp. 5181–5190, Dec.

Rodenbusch G. and Forristall, G. Z. (2013). *An Empirical Model for Random Directional Wave Kinematics near The Free Surface*, in Offshore Technology Conference.

Sorensen, R. M. (2005). *Basic Coastal Engineering*. Springer Science & Business Media.

Stewart, R. H. (2008). *Introduction to Physical Oceanography*, Department of Oceanography, Texas A&M University.

Techet, A. H. (2004). *Morrison's Equation*. Naval, vol. 34, pp. 191–197.

USACE (2002). U.S. Army Corps of Engineers, *Coastal Engineering Manual*, Washington, D.C.

Wheeler, J. D. (1969). *Methods for Calculating Forces Produced by Irregular Waves*, in Offshore Technology Conference.

Wilson, J. F. (2003). *Dynamics of offshore structures*. New York: John Wiley & Sons.

Wolfram J. and Naghipour, M. (1999). *On the estimation of Morison force coefficients and their predictive accuracy for very rough circular cylinders*, Appl. Ocean Res., vol. 21, no. 6, pp. 311–328.

Zhang, J., Randall, R. E. and Spell C. (1991). *On Wave Kinematics Approximate Methods*, in Offshore Technology Conference.