Chapter 2
Environmental Forces

Abstract This chapter deals with different types of environmental loads on offshore structures. It also includes code information regarding the loads. Step-by-step method for load estimate on a cylindrical member and an example structure is detailed. The procedure for estimating wave loads is illustrated through examples. Solved numerical examples and exercise are given at the end for practice.

Keywords Wind forces · Wave forces · Aerodynamic admittance function · Current forces · Wave theories · Marine growth · Design requirements · Allowable stress method · Limit state method · Fabrication and erection loads

2.1 Introduction

Loads acting on offshore structures are classified into the following categories:

- Permanent loads or dead loads
- Operating loads or live loads
- Other environmental loads including earthquake loads
- Construction and installation loads
- Accidental loads

While the design of buildings onshore is influenced mainly by the permanent and operating loads, the design of offshore structures is dominated by environmental loads, especially waves, and the loads arising in the various stages of construction and installation. In civil engineering, earthquakes are normally regarded as accidental loads (see Eurocode 8), but in offshore engineering, they are treated as environmental loads.

Environmental loads are those caused by environmental phenomena. These include wind, waves, current, tides, earthquakes, temperature, ice, seabed movement, and marine growth. Their characteristic parameters, defining design load values, are determined in special studies on the basis of available data. According to

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US and Norwegian regulations (or codes of practice), the mean recurrence interval for the corresponding design event must be 100 years, while according to the British rules, it should be 50 years or greater. The different loads to be considered while designing the structure are wind loads, wave load, mass, damping, ice load, seismic load, current load, dead load, live load, impact load, etc.

2.2 Wind Force

Wind forces on offshore structures are caused by complex fluid-dynamics phenomenon, which is generally difficult to calculate with high accuracy. Most widely used engineering approaches to estimate wind forces on offshore structures are based on few observations as listed below:

- When stream of air flows with constant velocity (v), it will generate force on the flat plate of area (A).
- The plate will be placed orthogonal to the flow direction.
- This force will be proportional to (Av^2).
- The proportionality constant is independent of the area, which is verified by experimental studies.

Hence, the wind force on a plate orthogonal to the wind flow direction can be determined by the net wind pressure as given below:

$$p_{\mathrm{w}} = \frac{1}{2}\rho_{\mathrm{a}} C_{\mathrm{w}} v^2 \tag{2.1}$$

where ρ_{a} is mass density of air (1.25 kg/m^3), and C_{w} is wind pressure coefficient. It is important to note that the mass density of air increases due to the water spray (splash) up to a height of 20–20 m above MSL. Hence, the total wind-induced force on the plate is given by:

$$F_{\mathrm{w}} = p_{\mathrm{w}} A \tag{2.2}$$

If the plate has an angle (θ) with respect to the wind direction, then the appropriate projected area, normal to the flow direction, should be used in the above equation. The wind pressure coefficient C_{w} is determined under controlled stationary wind flow conditions in a wind tunnel. It depends on the Reynolds number; typical values of 0.7–1.2 are used for cylindrical members. Natural wind has two components: (i) mean wind component (which is static component) and (ii) fluctuating, gust component (which is a dynamic component). The gust component is generated by the turbulence of the flow field in all the three spatial directions. For offshore locations, mean wind speed is much greater than the gust component, which means that in most of the design cases, a static analysis will suffice. The wind velocity is given by:

$$v(t) = \bar{v} + v(t) \tag{2.3}$$

where $\bar{v}\sqrt{a^2 + b^2}$ is the mean wind velocity and $v(t)$ is the gust component. The spatial dependence of the mean component is only through the vertical coordinate, while $v(t)$ is homogeneous in both space and time. Wind force in the directions parallel (drag force) and normal to the wind direction (lift force) is given by:

$$\begin{aligned} F_{\mathrm{D}} &= \frac{1}{2}\rho C_{\mathrm{D}}\bar{v}_{\mathrm{z}}A \\ F_{\mathrm{L}} &= \frac{1}{2}\rho C_{\mathrm{L}}\bar{v}_{\mathrm{z}}A \end{aligned} \tag{2.4}$$

Wind spectrum above water surface is given by 1/7th power law, which is:

$$v_z = V_{10}\left[\frac{z}{10}\right]^{\frac{1}{7}} \tag{2.5}$$

where v_z is the wind speed at elevation of z m above MSL, V_{10} is the wind speed at 10 m above MSL, and 10 m is called the reference height. Power law is purely empirical and most widely used. It is tested with the actual field measurements and found to be in good agreement. As Eq. (2.5) gives mean wind component, the gust component can be obtained by multiplying a gust factor with the sustained wind speed. Average gust factor (F_g) is in the range of 1.35–1.45; variation of the gust factor along the height is negligible. The sustained wind speed, which is to be used in the design, is the *one minute average wind speed*, according to the US Weather Bureau. The product of sustained wind speed and the gust factor will give the *fastest mile velocity*. 200 year sustained wind velocity of 125 miles per hour is to be used for the design of offshore structures.

Wind produces a low-frequency excitation. The fluctuating component is modeled probabilistically. Drag force on the members is caused by the encountered waves and wind. Wave forces alone acting on the member will cause inertia and drag forces, while earthquake forces cause only inertia forces on the members. Hence, vibration of the structure induced by wind and waves is different from that caused by earthquakes. For the design of members under wind loads, most of the international codes prefer quasi-static analysis. Very slender and flexible structures are wind-prone; for members under wave action, de-amplification takes place in flexible structures due to compliancy. While considering wind as a dynamic process, the following parameters are important:

- Length of the record: The record can be continuous, intermittent or a selective whose values are usually above the threshold ones. For the record to be continuous, average values of the wind velocity is lesser than that of the intermittent because of the longer length of the record when compared with the former.
- Wind spectrum: It is used as input for the structural analysis, which defines the fluctuating wind component.
- Gust component: It is approximated by the *aerodynamic admittance function*.

Aerodynamic admittance function is an intelligent way to define the cross-spectrum in the analysis, indirectly. There are two reasons for using the aerodynamic admittance function: (i) to bypass the rigorous random analysis and (ii) possibility of an accurate measurement of this function through wind-tunnel experiments. In this manner, the spatial variations of wind velocity are handled intelligently in the design. Force due to wind is given by:

$$\begin{aligned} F_{\mathrm{w}}(t) &= \frac{1}{2}\rho_{\mathrm{a}} C_{\mathrm{w}} v^2 A \\ &= \frac{1}{2}\rho_{\mathrm{a}} C_{\mathrm{w}} A[\bar{v} + v(t)]^2 \\ &= \frac{1}{2}\rho_{\mathrm{a}} C_{\mathrm{w}} A\left[\bar{v}^2 + (v(t))^2 + 2\bar{v}v(t)\right] \end{aligned} \tag{2.6}$$

by neglecting higher powers of gust component,

$$\cong \bar{F}_{\mathrm{w}} + \rho_{\mathrm{a}} C_{\mathrm{w}} A \bar{v} v(t)$$

In the above equation, wind force is expressed as a sum of mean component and the gust component. Wind is considered as an ergodic process; the (one-sided) power spectral density of the wind process is then related to the wind spectrum as:

$$S_{\mathrm{F}}^{+}(\omega) = [\rho_{\mathrm{a}} C_{\mathrm{w}}\, A\, \bar{v}]^2 S_{\mathrm{U}}^{+}(\omega) \tag{2.7}$$

Substituting Eq. (2.2) in Eq. (2.7) and rearranging the terms, we get:

$$S_{\mathrm{F}}^{+}(\omega) = \frac{4[\bar{F}_{\mathrm{w}}]^2}{[\bar{v}]^2}\left[\chi\left\{\frac{\omega\sqrt{A}}{2\pi\bar{v}}\right\}\right]^2 S_{\mathrm{U}}^{+}(\omega) \tag{2.8}$$

In the above equation, force and the response spectra are connected by the *aerodynamic admittance function*, which varies as below:

$$\begin{aligned} &\text{for} \quad \frac{\omega\sqrt{A}}{2\pi\bar{v}} \Rightarrow 0, \quad \chi\left\{\frac{\omega\sqrt{A}}{2\pi\bar{v}}\right\} \Rightarrow 1 \\ &\text{for} \quad \frac{\omega\sqrt{A}}{2\pi\bar{v}} \Rightarrow \infty, \quad \chi\left\{\frac{\omega\sqrt{A}}{2\pi\bar{v}}\right\} \Rightarrow 0 \end{aligned} \tag{2.9}$$

Aerodynamic admittance function is proposed through an empirical relationship by Davenport (1977):

$$\chi(x) = \left\{\frac{1}{\left[1 + (2x)^{4/3}\right]}\right\} \tag{2.10}$$

Wind spectra for the design of offshore structures are listed below with the details. For the reference height of z = 10 m, wind spectra as applied to offshore structures are expressed in terms of circular frequency as given below:

$$S_u^+(\omega) = fG_u^+(f) \tag{2.11}$$

(i) *Davenport spectrum*

$$\frac{\omega S_u^+(\omega)}{\delta \overline{U}_p^2} = \frac{4\theta^2}{\left(1+\theta^2\right)^{4/3}} \tag{2.12}$$

(ii) *Harris spectrum*

$$\frac{\omega S_u^+(\omega)}{\delta \overline{U}_p^2} = \frac{4\theta}{\left(2+\theta^2\right)^{5/6}} \tag{2.13}$$

Derivable variable θ is given by:

$$\theta = \frac{\omega L_u}{2\pi \overline{U}_{10}} = \frac{\delta L_u}{\overline{U}_{10}}, \quad 0 < \theta < \infty \tag{2.14}$$

where L_u is integral length scale (=1,200 m for Davenport and 1,800 m for Harris spectrum), δ is surface drag coefficient referred to $\bar{U}_{10}$. For offshore locations, $\delta = 0.001$. It is important to note that none of these spectrum used in the analysis of wind speed is recorded offshore; they are based on onshore records. Hence, these applications to offshore locations are questionable. They have serious problem when used for low-frequency flexible structures. Alternatively, for large floating structures, following spectra are recommended by Dyrbye and Hassen (1997):

(a) *Kaimal spectrum*

$$\frac{\omega S_u^+(\omega)}{\sigma_u^2} = \frac{6.8\,\theta}{(1+10.2\,\theta)^{5/3}} \tag{2.15}$$

where σ_u^2 is the variance of $U(t)$ at reference height of 10 m?

(b) *API (2000) spectrum*

$$\frac{\omega S_u^+(\omega)}{\sigma_u(z)^2} = \frac{\left(\omega/\omega_p\right)}{\left[1+1.5\left(\omega/\omega_p\right)\right]^{5/3}} \tag{2.16}$$

where ω_p is peak frequency and σ_z^2 is the variance of $U(t)$, which is not assumed as independent.

$$0.01 \leq \frac{\omega_p z}{\overline{U}(z)} \leq 0.1 \tag{2.17}$$

Usually, a value of 0.025 is obtained in lieu of the values computed from the above equation. Standard deviation and speed are given by:

$$\sigma_u(z) = \begin{cases} 0.15\overline{U}(z)\left(\frac{Z_s}{Z}\right)^{0.125} : Z \leq Z_s \\ 0.15\overline{U}(z)\left(\frac{Z_s}{Z}\right)^{0.275} : Z > Z_s \end{cases} \tag{2.18}$$

where Z_s is the thickness of the surface layer, which is usually taken as 20 m.

2.3 Wave Forces

Wind-generated sea surface waves shall be represented as a combination of series of regular waves. Regular waves of different magnitude and wave lengths from different directions are combined to represent the sea surface elevation. Water particle kinematics of regular waves is expressed by the sea surface elevation by various wave theories (Srinivasan and Bhattacharyya 2012). Among all the theories, Airy's wave theory is commonly used because it assumes linearity between the kinematic quantities and the wave height, which makes the wave theory simple. Airy's theory assumes a sinusoidal wave form of wave height (H), which is small in comparison with the wave length (λ) and water depth (d) as given below:

$$\begin{aligned}
\eta(x,t) &= \frac{H}{2}\cos(kx - \omega t) \\
k &= \frac{2\pi}{\lambda} \\
\dot{u}(x,t) &= \frac{\omega H}{2}\frac{\cosh(ky)}{\sinh(kd)}\cos(kx - \omega t) \\
\dot{v}(x,t) &= \frac{\omega H}{2}\frac{\sinh(ky)}{\sinh(kd)}\sin(kx - \omega t) \\
\ddot{u}(x,t) &= \frac{\omega^2 H}{2}\frac{\cosh(ky)}{\sinh(kd)}\sin(kx - \omega t) \\
\ddot{v}(x,t) &= -\frac{\omega^2 H}{2}\frac{\sinh(ky)}{\sinh(kd)}\cos(kx - \omega t)
\end{aligned} \tag{2.19}$$

Airy's theory is valid up to mean sea level only. However, due to the variable submergence effect, the submerged length of the members will be continuously changing. This will attract additional forces due to their variable submergence at any given time. To compute the water particle kinematics up to the actual level of submergence, stretching modifications suggested by various researchers are used.

(a) Wheeler suggested the following modification in the horizontal water particle velocity and acceleration to include the actual level of submergence of the member:

$$\begin{aligned} \dot{u}(x,t) &= \frac{\omega H}{2}\frac{\cosh\left(ky\left[\frac{d}{d+\eta}\right]\right)}{\sinh(kd)}\cos(kx-\omega t) \\ \ddot{u}(x,t) &= \frac{\omega^2 H}{2}\frac{\cosh\left(ky\left[\frac{d}{d+\eta}\right]\right)}{\sinh(kd)}\sin(kx-\omega t) \end{aligned} \tag{2.20}$$

(b) Chakrabarti suggested the modification as given below:

$$\begin{aligned} \dot{u}(x,t) &= \frac{\omega H}{2}\frac{\cosh(ky)}{\sinh(k(d+\eta))}\cos(kx-\omega t) \\ \ddot{u}(x,t) &= \frac{\omega^2 H}{2}\frac{\cosh(ky)}{\sinh(k(d+\eta))}\sin(kx-\omega t) \end{aligned} \tag{2.21}$$

The sea state, in a short term, which is typically 3 h, is assumed as zero-mean, ergodic Gaussian process. This can be defined completely by a wave spectrum. For North Sea, Johnswap spectrum is recommended. For open sea conditions, Peirson–Moskowitz (P–M) spectrum is recommended. In a long term, variation of sea state is slower than the short-term fluctuations. It is often approximated by a series of stationary, non-zero-mean Gaussian process, which is specified by the significant wave height (H_s) and peak wave period (T_p). Following are a few relevant spectra, applicable in the design of offshore platforms.

2.4 Wave Theories

Wave theories serve to calculate the particle velocities, accelerations, and the dynamic pressure as functions of the surface elevation of the waves. For long-crested regular waves, the flow can be considered two-dimensional and are characterized by parameters such as wave height (H), period (T) and water depth (d), as shown in Fig. 2.1. $k = 2\pi/L$ denotes the wave number, $\omega = 2\pi/T$ denotes the wave circular frequency, and $f = 1/T$ denotes the cyclic frequency. Different wave theories available are as follows:

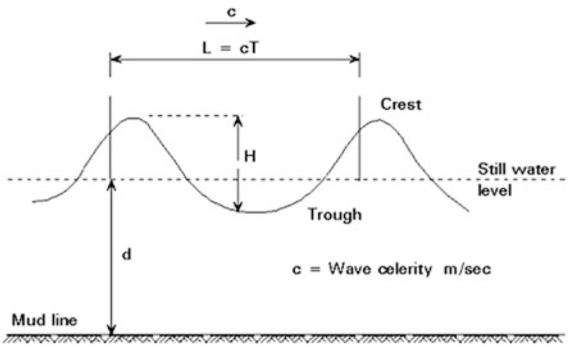


Fig. 2.1 Definition of wave parameters

- Linear or first-order or Airy theory
- Stokes fifth-order theory
- Solitary wave theory
- Cnoidal theory
- Dean's stream function theory
- Numerical theory by Chappelear

Figure 2.2 shows the chart for the selection of the most appropriate theory, based on the parameters, H, T, and d. For example, linear wave theory can be applied when $H/gT^2 < 0.01$ and $d/GT^2 > 0.05$, besides other ranges, as shown in the figure.

(a) *PM spectrum for wave loads*

$$S^{+}(\omega) = \frac{\alpha g^2}{\omega^5} \exp\left[-1.25\left(\frac{\omega}{\omega_0}\right)^{-4}\right] \tag{2.22}$$

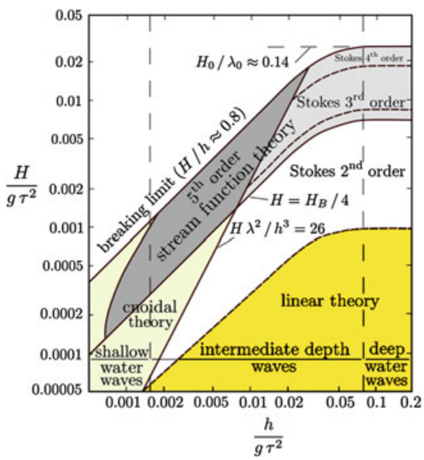


Fig. 2.2 Wave theory selection chart (Sarpakaya and Issacson 1981)

where α is Phillips constant $\cong 0.0081$.

(b) *Modified PM spectrum (2 parameters H_s, ω_0)*

$$S^{+}(\omega) = \frac{5}{16} H_{\mathrm{s}} \frac{\omega_0^4}{\omega^5} \exp\left[-1.25\left(\frac{\omega}{\omega_0}\right)^{-4}\right] \tag{2.23}$$

(c) *International Ship Structures Congress (ISSC) spectrum (2 parameters H_s, $\bar{\omega}$)*

$$\begin{aligned} S^{+}(\omega) &= 0.1107 H_{\mathrm{s}} \frac{\omega^{-4}}{\omega^5} \exp\left[-0.4427\left(\frac{\omega}{\bar{\omega}}\right)^{-4}\right] \\ \overline{\omega} &= \frac{M_1}{M_0} \end{aligned} \tag{2.24}$$

where M_1 and M_0 are spectral moments.

(d) *Johnswap spectrum (5 parameters $H_{\mathrm{s}}, \omega_{0o}, \gamma, \tau_a, \tau_b$)*

$$S^{+}(\omega) = \frac{\overline{\alpha} g^2}{\omega^5} \exp\left[-1.25\left(\frac{\omega}{\omega_0}\right)^{-4}\right] \gamma^{a(\omega)} \tag{2.25}$$

where γ is the peakness parameter.

$$a(\omega) = \exp\left[-\frac{(\omega-\omega_0)^2}{2\overline{\sigma}^2 \omega_0^2}\right] \tag{2.26}$$

where $\bar{\sigma}$ is spectral width parameter and is given by:

$$\overline{\sigma}_a = 0.07, \quad \omega \leq \omega_0 \tag{2.27}$$

$$\overline{\sigma}_b = 0.09, \quad \omega > \omega_0 \tag{2.28}$$

The modified Phillips constant is given by:

$$\overline{\alpha} = 3.25 \times 10^{-3} H_s^2 \omega_0^4 [1 - 0.287 \ln(\gamma)] \tag{2.29}$$

$$\gamma = 5 \text{ for } \frac{T_p}{\sqrt{H_s}} \leq 3.6 \tag{2.30}$$

$$= \exp\left[5.75 - 1.15 \frac{T_p}{\sqrt{H_s}}\right] \text{ for } \frac{T_p}{\sqrt{H_s}} > 3.6 \tag{2.31}$$

$$H_s = 4\sqrt{m_0} \tag{2.32}$$

where γ varies from 1 to 7.

Main force components, rising from the wave loads, are grouped as follows: (i) *Froude–Krylov force*, which is caused by the pressure effects due to the undisturbed incident waves; (ii) *diffraction force*, which is caused by the pressure effects due to the presence of the structure in the fluid-flow domain; (iii) *hydrodynamic added mass and potential damping forces*, which is caused by the pressure effects due to the motion of the structural components in ideal fluid; and (iv) *viscous drag force*, which is caused by the pressure effects due to the relative velocity between the water particle and the structural component. For slender structures, *Froude–Krylov force and diffraction forces* are idealized by a single inertia term. Velocity and acceleration do not differ significantly from the values of the cylinder axis when $D/\lambda < 0.2$. When the waves act on the slender structures, the structure oscillates, which will set up waves radiating away from it. Reaction forces are then set up in the fluid, which will be proportional to the acceleration and velocity of the structure. Reaction force proportional to the acceleration of the structure will result in an added mass term, contributing to the inertia force. Reaction force proportional to the velocity results in the potential damping force. If the structure is compliant, the added mass forces associated with the relative acceleration between the fluid particles and the structures are included. Drag force is computed by replacing the water particle velocity with the relative velocity term. The total force acting normal to the axis of the member is given by:

$$
\begin{aligned}
q_n &= \rho\,\mathrm{d}V \cdot a_n + (C_m - 1)\rho\,\mathrm{d}V(a_n - \ddot{x}_n) + \frac{1}{2}\rho\,C_d\,\mathrm{d}A(v_n - \dot{x}_n)|v_n - \dot{x}_n| \\
&= C_m \rho \mathrm{d}V \cdot a_n - (C_m - 1)\rho \mathrm{d}V \ddot{x}_n + \frac{1}{2}\rho\,C_d\,\mathrm{d}A(v_n - \dot{x}_n)|v_n - \dot{x}_n|
\end{aligned}
\tag{2.33}
$$

where ρ is density of fluid, (C_d, C_m) are the drag and inertia coefficients, (v_n, a_n) are velocity and acceleration of the water particle normal to the axis of the member, $\dot{x}, \ddot{x}$ are the velocity and acceleration of the structure, and (dA, dV) are exposed area and displaced volume of water per unit length, respectively.

The above equation has two main issues: first, the relative motion formulation is valid only if the structure motion is of large amplitude; second, the relative velocity formation of the drag produces both excitation and damping forces. In the above equation, the most critical aspect is the evaluation of the drag and inertia coefficients, which is dependent on flow conditions, Keulegan–Carpenter number, and Reynolds number. The recommended value of drag coefficient is 0.6–1.2, while that of the inertia coefficient is 1.2–2.0, as seen in the literature (APR RP 2A). As in the case of bottom-supported structures (gravity platforms), when the diameter of the member is very large, incident waves are disturbed by the presence of the structure. In such cases, viscous force becomes less significant due to the smaller values of the ratio of wave height to member diameter ($H/D \ll 1$). In such cases, the above equations cannot be applied; it is recommended that the analyzer should use numerical methods to determine the forces on the members.

Offshore structures have large plane area. Larger topside is required for accommodating the equipment layout as discussed in the previous chapter. As the deck is supported by few column members, their spacing plays an important role in order to reduce the interference of the waves by their presence. For a large spacing of c/c distance of column members, there can be cancellation of forces. Let us consider an example of the tension leg platform (TLP). For a typical size of topside of 90 × 90 m, resting on four columns, phase angle (θ) is given by the following relationship:

$$\theta = \frac{2\pi \Delta x}{\lambda} \tag{2.34}$$

where Δx is the c/c distance between the column members (leg spacing) and λ is the wave length. For the spacing between the columns of 90 m and wave period of 10 s, the phase angle will be 1.2π, which can cause cancellation of forces on members. It is important to note that the spacing of the members is chosen in such a manner that the force cancellation effects at the dominant wave frequencies are expected to have close to the natural frequency of the platform. The forces on a submerged structure in waves appear from the pressure distribution on its surface. For a small structure, Morison equation is valid because the flow structure is complex. However, for large structures (relative to the wavelength), the flow remains essentially attached to the surface. It is therefore easier to compute this pressure field. If the computation of the scattered wave potential is waived and its effect is incorporated by a force coefficient, then this force is called the *Froude–Krylov* force. Thus, the calculation of the force is performed assuming that the structure does not distort the wave field in its vicinity. The force is computed by a pressure-area method using the incident wave pressure that is acting on the submerged surface of the structure. Then, a force coefficient is used to account for the wave diffraction.

For a few basic shapes of the structural forms, a closed form expression is obtained by the Froude–Krylov theory: (i) horizontal cylinder, (ii) horizontal half-cylinder, (iii) vertical cylinder, (iv) sphere, (v) hemisphere, and (vi) rectangular barge.

(a) Force on a horizontal cylinder is given by:

$$f_{\mathrm{H}} = r\ell \int_0^{2\pi} p \cos\theta \, \mathrm{d}\theta$$

(b) Force on a vertical cylinder:

Consider a vertical cylinder placed on the ocean bottom and extended above the still water level, as shown in Fig. 2.3:

Velocity potential is given by:

$$\varphi = \frac{gH}{2\omega} \frac{\cosh(ks)}{\cosh(kd)} \sinh(kx - \omega t) \tag{2.35}$$

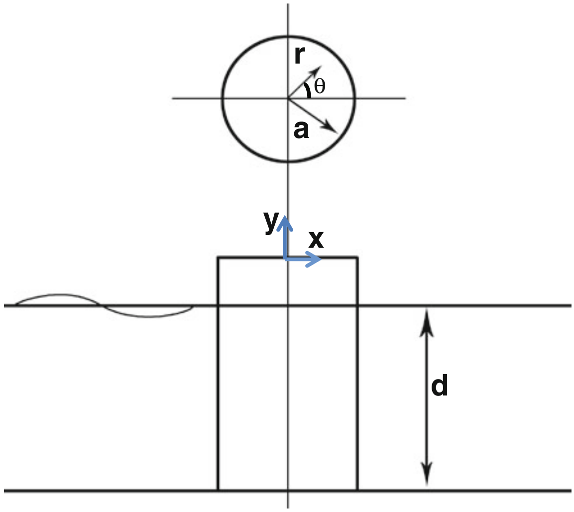


Fig. 2.3 Bottom-supported cylinder

Dynamic pressure is given by:

$$\begin{aligned} p &= \rho \frac{\partial \varphi}{\partial t} \\ &= \rho g \frac{H}{2} \frac{\cosh(ks)}{\cosh(kd)} \cos(kx - \omega t) \end{aligned} \tag{2.36}$$

Horizontal force per unit length is given by:

$$\begin{aligned} f_x &= \rho \int_0^{2\pi} \int_{-d}^{0} \frac{\partial \varphi_0}{\partial t} a \cos\theta \, \mathrm{d}\theta \, \mathrm{d}\ell \\ f_x &= \frac{\rho g a H}{2\cosh(kd)} \int_{-d}^{0} \cosh(ks) \mathrm{d}s \int_0^{2\pi} \cos[ka \cos\theta - \omega t] \cos\theta \, \mathrm{d}\theta \end{aligned} \tag{2.37}$$

This reduces to the following form, which accounts for the diffraction effect:

$$f_x = C_{\mathrm{H}} \frac{\pi \rho g H a}{k} J_1(ka) \tanh(kd) \sin \omega t \tag{2.38}$$

The above method of computing the forces by the incident wave alone is known as Froude–Krylov theory. It does not give the correct value of the force, as the phase value is accounted for in the equation. It is due to this fact a force coefficient is used in the expression as a multiplier. For a vertical cylinder, the horizontal force coefficient is taken as 2; for small values of ka; the value changes as ka increases.

Table 2.1 Forces on members of different geometric shapes using Froude–Krylov theory

Basic shape	Horizontal force	C_H	Vertical force	C_V	K_a range
Horizontal cylinder	$C_H \rho V \dot{u}_0$	2.0	$C_V \rho V \dot{v}_0$	2.0	0–1.0
Horizontal half-cylinder	$C_H \rho V [\dot{u}_0 + C_1 \omega v_0]$	2.0	$C_V \rho V [\dot{v}_0 + C_2 \omega u_0]$	1.1	0–1.0
Vertical cylinder	$C_H \rho V \frac{2J_1(ka)}{ka} \frac{\sinh\left[\frac{k\ell_1}{2}\right]}{\left[\frac{k\ell_1}{2}\right]} \dot{u}_0$	2.0	–	–	–
Rectangular block	$C_H \rho V \frac{\sinh\left[\frac{k\ell_3}{2}\right]}{\left[\frac{k\ell_3}{2}\right]} \frac{\sinh\left[\frac{k\ell_1}{2}\right]}{\left[\frac{k\ell_1}{2}\right]} \dot{u}_0$	1.5	$C_V \rho V \frac{\sinh\left[\frac{k\ell_3}{2}\right]}{\left[\frac{k\ell_3}{2}\right]} \frac{\sinh\left[\frac{k\ell_1}{2}\right]}{\left[\frac{k\ell_1}{2}\right]} \dot{v}_0$	6.0	0–5.0
Hemi sphere	$C_H \rho V [\dot{u}_0 + C_3 \omega v_0]$	1.5	$C_V \rho V [\dot{v}_0 + C_4 \omega u_0]$	1.1	0–0.8
Sphere	$C_H \rho V \dot{u}_0$	1.5	$C_V \rho V \dot{v}_0$	1.1	0–1.75

Table 2.1 shows the equations for forces using Froude–Krylov theory for different geometric shapes of members.

Where V = submerged volume of the structure; C_H and C_V are force coefficients in the horizontal and vertical directions, respectively; subscript zero indicates that the amplitude of the water particle velocity or acceleration is computed at the center of the geometric shape; and l_1 and l_3 are the length and underwater depth of the rectangular block, respectively. The numerical values of C_1–C_4 depend on the diffraction parameter ka and are given in Table 2.2. The forces in Table 2.1 are given in terms of the water particle acceleration and velocity at the center of the structure wherever possible. The force coefficients shown are applicable over a small range of diffraction parameter. If the values of ka are much different from the range given in the table, values of the force coefficients are to be used with caution.

Table 2.2 Numerical values of C_1–C_4

ka	C_1	C_2	C_3	C_4
0.1	0.037	15.019	0.042	12.754
0.2	0.075	7.537	0.085	6.409
0.3	0.112	5.056	0.127	4.308
0.4	0.140	3.825	0.169	3.268
0.5	0.186	3.093	0.210	2.652
0.6	0.223	2.612	0.252	2.249
0.7	0.259	2.273	0.292	1.966
0.8	0.295	2.024	0.332	1.760
0.9	0.330	1.834	0.372	1.603
1.0	0.365	1.685	0.411	1.482
1.5	0.529	1.273	0.591	1.156
2.0	0.673	1.105	0.745	1.034
2.5	0.792	1.031	0.867	0.989
3.0	0.886	0.999	0.957	0.977
3.5	0.955	0.989	1.015	0.978
4.0	1.000	0.087	1.945	0.985

2.5 Current Forces

The presence of current in water produces the following distinct effects: Current velocity should be added vectorially to the horizontal water particle velocity before computing the drag force, because drag force depends on the square of the water particle velocity. Current decreases slowly with the increase in depth, but even a small magnitude of current velocity can cause significant drag force. This effect is insignificant and generally neglected. Current makes the structure itself to generate waves, which in turn creates diffraction forces. However, these values are negligible for realistic value of current acting on the normal-sized members. The presence of current is alternatively accounted by increasing the wave height to 10–15 % and neglects the presence of current per se.

2.6 Earthquake Loads

Offshore platforms which do not have stiff connection with the seabed are indirectly influenced by earthquakes; those which are bottom-supported are affected by earthquakes directly. Compliant structures that are position-restrained by tethers will be subjected to dynamic tether tension variations under the presence of earthquake forces. This will give rise to the dynamic tether tension variations, which in turn shall affect the response of the platform under lateral loads. Earthquakes give rise to the horizontal and vertical motions for a typical duration of 15–30 s. Earthquake acceleration exhibits random characteristics due to (i) the nature of the mechanism causing earthquakes; (ii) wave propagation; (iii) reflection; and (iv) deflection. Effects of earthquake forces give rise to horizontal and vertical motions with durations of 15–30 s. Earthquake loads exhibit random characteristics due to the nature of the mechanism causing earthquake, wave propagation, reflection, and deflection. Earthquakes can result in inertia forces due to the acceleration and damping forces due to the motion of the water particles.

In case of the analysis of compliant structures like TLPs, earthquake forces are handled in an indirect manner. Water waves generated due to the ground motion are neglected. Stiffness of TLP tether is modeled as axial tension members; slackening of tethers is neglected. The dynamic tether tension variation, caused by the horizontal motion of the earthquakes, is used to update the stiffness matrix of the TLP using the following equation (Chandrasekaran and Gaurav 2008):

$$\Delta T = \frac{AE}{\ell}\left[x(t) - x_g(t)\right] \tag{2.39}$$

where $x(t)$ is the instantaneous response vector of TLP and $x_g(t)$ is the ground displacement vector, which is given by:

$$x_{g}(t)=\begin{Bmatrix} x_{1g}(t) \\ 0 \\ x_{3g}(t) \\ 0 \\ 0 \\ 0 \end{Bmatrix} \tag{2.40}$$

where x_{1g} and x_{3g} are the horizontal and vertical ground displacements, respectively. Ground motions can be generated using Kanai-Tajimi ground acceleration spectrum (K-T spectrum), which is given by:

$$\begin{aligned} S_{\ddot{x}_g\ddot{x}_g}(\omega) &= \left[\frac{\omega_g^4+4\xi_g^2\omega_g^2\omega^2}{\left(\omega_g^2-\omega^2\right)^2+4\xi_g^2\omega_g^2\omega^2}\right]S_0 \\ S_0 &= \frac{2\xi_g\sigma_g^2}{\pi\omega_g(1+4\xi_g^2)} \end{aligned} \tag{2.41}$$

where S_0 is the intensity of earthquake, ω_g is the natural frequency of the ground, ξ_g is the damping of the ground, and σ_g^2 is the variance of the ground acceleration. These are the three parameters on which K-T spectrum depends on, which need to be chosen for any analytical studies on TLP under seismic action. The above three parameters should be estimated from the representative earthquake records by established estimation processes (Chandrasekaran et al. 2006). For example, an earthquake occurred in GoM, approximately at 250 miles WSW of Anna Maria, Florida on September 10, 2006 at 14:56:07 (coordinated universal time). The signal was epicentered 26.34N, 86.57W. Incidentally, MARS TLP was operating in the Mississippi Canyon Block, which is also located in GoM. The three parameters S_0, ω_g, and ξ_g are chosen such that the real earthquake is simulated for analysis purposes (Chandrasekaran and Gaurav 2008). Studies showed that the dynamic tether tension variations caused by the earthquake forces are in the order of about 65 % more than that of the normal values. Even structures with rigid degrees of freedom like heave are excited, which may result in the loss of functionality of the platform.

2.7 Ice and Snow Loads

Ice loads are dominant in offshore structures in the Arctic regions. Prediction of ice loads is associated with a significant degree of uncertainty, as there are various ice conditions that exist in the service life of an offshore platform. They are level ice, broken ice, ice ridges, and icebergs. Offshore structures show different types of failure under ice loads, namely creep, cracking, buckling, spalling, and crushing. Ice loads exhibit random variations in both space and time. They are classified into: (i) total or global loads and (ii) local loads or pressure. Global loads affect the overall motion and stability of the platform, while local loads affect the members at connections. In the level ice condition, frequency of interaction between the structure and ice is important; number of interactions per unit time is important to quantify the ice loads on offshore platforms. Total ice force can result in a periodic loading and can cause dynamic amplification in flexible/slender structures. Current codes include equations for the extreme static ice loads, which depend on the geometric shape of the structure. Studies show that ice loads in a conical structure are lesser than that of the cylindrical structure (Sanderson 1988). This is because a well-designed cone shape can change the ice-failure mode from crushing to bending. Estimating (predicting) ice forces on offshore platforms has a lot of uncertainties. Ice forces often control the design of the platform in operational conditions, in particular. The design ice loads use varying factors for level ice, first-year ridge ice, and multi-year ridge ice; the factored values are 2, 5, and 7, respectively.

There are four approaches for addressing ice forces on offshore platforms: (a) experimental studies on scaled models; (b) numerical studies; (c) field studies; and (d) data mining. Experimental studies use scaling laws to determine the ice loads and ice–structure interaction. This method claims many advantages due to the capability of testing many types of structural shapes in large testing facilities. However, such tests are expensive apart from a strong disagreement of the model ice not being accurately scaled as of the sea ice. As the ice failure is dependent on the geometric shape significantly, ice-failure behavior cannot be accurately studied. This may result in overprediction of ice loads. Numerical modeling uses high-end software to model ice forces for different interaction scenarios, which makes it very cost-effective and instructive. However, limited validation of results with that of the experiments is reported in the literature. The more practical way to estimate ice loads is from data mining. Previous platforms can be visited to determine the ice loads through field measurements. This will give a real picture of the ice loads. In the frequency domain approach, excitation caused by ice loads is modeled as sinusoidal pseudo-excitation, and the response is characterized by the transfer function. Ice force spectrum on a narrow conical structure is given by:

$$S^{+}(f) = \frac{A\bar{F}_0^2\bar{T}^{(-\delta)}}{f^{\gamma}}\exp\left[-\frac{B}{\bar{T}^{(\alpha)}f^{\beta}}\right] \tag{2.42}$$

where A (=10) and B (=5.47) are constants; $\bar{F}_0$ is the force amplitude on the structure; $\bar{T} = L_b/v$ is the period of ice; L_b is ice-breaking length, which is typically 4–10 times of thickness of ice; v is the velocity; and $\alpha, \beta, \gamma, \delta$ are constants whose values are typically 0.64, 0.64, 3.5, and 2.5, respectively. Force amplitude on the structure is given by:

$$\bar{F}_0 = C\,\sigma_f h^2 \left(\frac{D}{L_\mathrm{c}}\right)^{0.34} \tag{2.43}$$

where C is the constant; σ_f is bending strength of ice (0.7 MPa); h is the ice thickness; D is the diameter of the ice cone, and L_c is the characteristic length of ice, which is given by the following equation:

$$L_c = \left[\frac{Eh^3}{12g\rho_w}\right]^{0.25} \tag{2.44}$$

where E is Young's modulus of ice (=0.5 GPa) and ρ_w is density of water.

2.8 Marine Growth

Marine growth or biofouling is the ubiquitous attachments of soft and hard bio-particles on the surface of a submerged structure. It ranges from seaweeds to hard shelled barnacles. Its growth on the surface of the structure increases its diameter and affects its roughness. Its main effect is to increase the wave forces on the members by increasing not only exposed areas and volumes, but also the drag coefficient due to higher surface roughness. In addition, it increases the unit mass of the member, resulting in higher gravity loads and in lower member frequencies. Depending upon the geographic location, the thickness of marine growth can reach 0.3 m or more. It is accounted for in the design through appropriate increases in the diameters and masses of the submerged members.

2.9 Mass

Mass is contributed by the structural mass and hydrodynamic added mass of the structure. For a slender structure, mass of the displaced volume of the structure will be significant and should be considered in the analysis. Added mass depends on the submerged volume of the platform, which also varies with respect to period of vibration. This is due to the variation in buoyancy, which in turn changes the tether tension variation that affects the natural frequency of motion. Based on the equipment layout plan and the chosen structural form, one can compute the mass of the platform readily. It is also important to establish the fact that a desired proportion between center of buoyancy and center of mass is maintained to ensure stability under free-floating conditions. This is important to enable smooth construction process in case of floating.

2.10 Damping

For steel offshore structures, structural damping is usually considered to vary from 0.2 to 0.5 % of that of the critical damping (Adams and Baltrop 1991). For concrete structures, it can be of the order 0.5–1.5 %. Hydrodynamic damping originates from the radiation damping and viscous damping effects. Radiation damping is determined using potential theory. It exhibits a strong dependence on frequency and submergence effects. Literature shows that the drag damping is lower for structures with large diameter column members ($\sim$0.1 %). Damping ratio for offshore structures (wet structures), including the effects of added mass, can be expressed as a ratio of that of the dry structures, as given below:

$$\xi_{\text{wet}} = \xi_{\text{wet}} \frac{\left(m^*_{\text{dry}}\right)\left(\omega^*_{\text{dry}}\right)}{\left(m^*_{\text{wet}}\right)\left(\omega^*_{\text{wet}}\right)} \tag{2.45}$$

where m^* and ω^* are generalized mass and frequency, respectively (Naess and Moan 2013). Literature shows that the total damping ratio is about 2 % for the first three modes of bottom-supported structures.

2.11 Dead Load

Dead load is the weight of the overall platform in air, which includes piling, superstructure, jacket, stiffeners, piping, conductors, corrosion anodes, deck, railing, grout, and other appurtenances. Dead load excludes the following: weight of the drilling equipment placed on the platform including the derrick, draw works, mud pumps, mud tanks, etc.; weight of production or treatment equipment located on the platform including separators, compressors, piping manifolds, and storage tanks; weight of drilling supplies that cause variable loads during drilling such as drilling mud, water, fuel, casing, etc.; weight of treatment supplies employed during production such as fluid in the separator, storage in the tanks; drilling load, which is approximate combination of derrick load, pipe storage, rotary table load, etc.

2.12 Live Load

Live loads are acting in addition to the equipment loads. They include load caused by impacts of vessels and boats on the platform. Dynamic amplification factor is applied to such loads to compute the enhanced live loads. Live loads are generally designated as factor times of the applied static load. These factors are assigned by the designer depending on the type of platform. Table 2.3 gives the live load factors that are used in the platform design.

Table 2.3 Typical live load values used in platform design (Graff 1995)

Description	Uniform load on decks (kN/m^2)	Concentrated load on deck (kN/m^2)	Concentrated load on beams
Walkway, stair	4.79	4.38	4.45 kN/m^2
Areas >40 m^2	3.11		
Areas for light use	11.9	10.95	267 kN

Table 2.4 Impact factor for live loads

Structural item	Load direction	
	Horizontal	Vertical
Rated load in craned	20 %	100 %
Drilling hook loads	–	–
Supports of light machinery	–	20 %
Supports of rotating machinery	50 %	50 %
Boat landings	890 kN	890 kN

2.13 Impact Load

For structural components which experience impact under live loads, the stipulated live loads in Table 2.3 should be increased by an impact factor, as given in Table 2.4. Deck floor loads can be taken as 11.95 kN/m^2 in the drilling rig area, 71.85 kN/m^2 in the derrick area, and 47.9 kN/m^2 for pipe racks, power plants, and living and accommodation areas.

2.14 General Design Requirements

Design methodology of offshore platforms differs with different types of offshore structures. For example, vertical deformation will be lesser in case of bottom-supported structures such as jacket platform and GBS. Such platforms are highly rigid and tend to attract more forces. Hence, the design criteria should be to limit the stresses in the members. Displacement of the members under the applied loads will be insignificant. On the contrary, compliant structures are more flexible, as they all displaced more under wave action. They also create more disturbances in the waves. Hence, the design criteria will be to control displacement instead of limiting the stresses in the members. Orientation of the platform is another important aspect in the design. Preferred orientation is that members are oriented to have less projected area to the encountered wave direction. This induces lesser response on the members. Predominant wave direction for the chosen site is made available to the designer based on which the platform orientation is decided (Chandrasekaran and Bhattacharyya 2012). Following are the list of data required for the design of offshore structures:

- Land topographical survey of sufficient area covering the chosen site for platform installation
- Hydrographical survey of the proposed location (hydrographic charts are used for this purpose)
- Information regarding silting at the site
- Wind rose diagram showing information on wind velocities, duration, predominant direction round the year
- Cyclonic tracking data showing details of the past cyclonic storm such as wind velocities, direction, peak velocity period, etc., are indicated.
- Oceanographic data including general tide data, tide table, wave data, local current, seabed characteristics, temperature, rainfall, and humidity
- Seismicity level and values of acceleration
- Structural data of existing similar structures, preferably in the close vicinity
- Soil investigation report

2.15 Steel Structures

The analysis of an offshore structure is an extensive task, embracing consideration of the different stages, i.e., execution, installation, and in-service stages, during its life. Many disciplines such as structural, geotechnical, naval architecture, and metallurgy are involved. The analytical models used in offshore engineering are in some respects similar to those adopted for other types of steel structures. Only the salient features of offshore models are presented here. The same model is used throughout the analysis with only minor adjustments to suit the specific conditions, e.g., at supports in particular, relating to each analysis. Stick models (beam elements assembled in frames) are used extensively for tubular structures (jackets, bridges, and flare booms) and lattice trusses (modules and decks). Each member is (normally) rigidly fixed at its ends to other elements in the model. If more accuracy is required, particularly for the assessment of natural vibration modes, local flexibility of the connections may be represented by a joint stiffness matrix. In addition to its geometrical and material properties, each member is characterized by hydrodynamic coefficients, e.g., relating to drag, inertia, and marine growth, to allow wave forces to be automatically generated. Integrated decks and hulls of floating platforms, involving large bulkheads, are described by plate elements. The characteristics assumed for the plate elements depend on the principal state of stress to which they are subjected. Membrane stresses are taken when the element is subjected merely to axial load and shear. Plate stresses are adopted when bending and lateral pressure is to be taken into account. After developing a preliminary model for analysis, member stresses are checked for preliminary sizing under different environmental loads.

The verification of an element consists of comparing its characteristic resistance(s) to a design force or stress. It includes (i) a strength check where the characteristic resistance is related to the yield strength of the element and (ii) a stability check for

Table 2.5 Coefficient for resistance to stresses

Condition	Axial	Strong axis bending	Weak axis bending
Normal	0.60	0.66	0.75
Extreme	0.80	0.88	1.00

elements in compression where the characteristic resistance relates to the buckling limit of the element. An element (member or plate) is checked at typical sections (at least both ends and midspan) against resistance and buckling. This verification also includes the effect of water pressure for deepwater structures. Tubular joints are checked against punching under various load patterns. These checks may indicate the need for local reinforcement of the chord using over-thickness or internal ring-stiffeners. Elements should also be verified against fatigue, corrosion, temperature, or durability wherever relevant.

2.16 Allowable Stress Method

This method is presently specified by American codes (API, AISC). The loads remain unfactored, and a unique coefficient is applied to the characteristic resistance to obtain an allowable stress as shown in Table 2.5.

'Normal' and 'extreme,' respectively, represent the most severe conditions under which (a) the plant is to operate without shutdown and (b) the platform is to endure over its lifetime.

2.17 Limit State Method

This method is enforced by European and Norwegian authorities and has now been adopted by American Petroleum Institute (API) as it offers a more uniform reliability. Partial factors are applied to the loads and to the characteristic resistance of the element as given in Table 2.6. They reflect the amount of confidence placed in

Table 2.6 Load factors

Limit state	Load categories				
	P	L	D	E	A
ULS (normal)	1.3	1.3	1.0	0.7	0.0
ULS (extreme)	1.0	1.0	1.0	1.3	0.0
FLS	0.0	0.0	0.0	1.0	0.0
PLS (accidental)	1.0	1.0	1.0	1.0	1.0
PLS (post-damage)	1.0	1.0	1.0	1.0	0.0
SLS	1.0	1.0	1.0	1.0	0.0

the design value of each parameter and the degree of risk accepted under a limit state as discussed below:

- Ultimate limit state (ULS), which corresponds to an ultimate event considering the structural resistance with appropriate reserve.
- Fatigue limit state (FLS), which relates to the possibility of failure under cyclic loading.
- Progressive collapses limit state (PLS), which reflects the ability of the structure to resist collapse under accidental or abnormal conditions.
- Service limit state (SLS), which corresponds to the criteria for normal use or durability (often specified by the plant operator).

where the following explanations are applicable:

P represents permanent loads (structural weight, dry equipment, ballast, and hydrostatic pressure)
L represents live loads (storage, personnel, and liquid)
D represents deformations (out-of-level supports and subsidence)
E represents environmental loads (wave, current, wind, and earthquake)
A represents accidental load (dropped object, ship impact, blast, and fire). The material partial factors for steel are normally taken equal to 1.15 for ULS and 1.00 for PLS and SLS design. Guidance for classifying typical conditions into typical limit states is given in Table 2.7.

The analysis of the offshore platform is an iterative process, which requires progressive adjustment of the member sizes with respect to the forces they transmit, until a safe and economical design is achieved. It is therefore of utmost importance to start the main analysis from a model which is close to the final optimized one. The simple rules given below provide an easy way of selecting realistic sizes for the main elements of offshore structures in moderate water depth (up to 80 m) where dynamic effects are negligible.

Jacket Pile Sizes

- Calculate the vertical resultant (dead weight, live loads, and buoyancy), the overall shear, and the overturning moment (environmental forces) at the mudline.
- Assuming that the jacket behaves as a rigid body, derive the maximum axial and shear force at the top of the pile.
- Select a pile diameter in accordance with the expected leg diameter and the capacity of pile-driving equipment.
- Derive the penetration from the shaft friction and tip bearing diagrams.
- Assuming an equivalent soil subgrade modulus and full fixity at the base of the jacket, calculate the maximum moment in the pile and derive its wall thickness.

Table 2.7 Conditions specified for various limit states

Conditions	Loadings				Design criterion
	P/L	E	D	A	
Construction	P				ULS, TSLS
Load-out	P	Reduced wind	Support displacement		ULS
Transport	P	Transport wind and wave			ULS
Tow-out (accidental)	P			Flooded compartment	PLS
Launch	P				ULS
Lifting	P				ULS
In-place (normal)	P + L	Wind, wave and snow	Actual		ULS, SLS
In-place (extreme)	P + L	Wind and 100 year wave	Actual		ULS, SLS
In-place (exceptional)	P + L	Wind and 10,000 year wave	Actual		PLS
Earthquake	P + L	10^{-2} quake			ULS
Rare earthquake	P + L	10^{-4} quake			PLS
Explosion	P + L			Blast	PLS
Fire	P + L			Fire	PLS
Dropped object	P + L			Drill collar	PLS
Boat collision	P + L			Boat impact	PLS
Damaged structure	P + reduced L	Reduced wave and wind			PLS

Deck Leg Sizes

- Adapt the diameter of the leg to that of the pile.
- Determine the effective length from the degree of fixity of the leg into the deck (depending upon the height of the cellar deck).
- Calculate the moment caused by wind loads on topsides and derive the appropriate thickness.

Jacket Bracings

- Select the diameter in order to obtain a span/diameter ratio between 30 and 40.
- Calculate the axial force in the brace from the overall shear and the local bending caused by the wave assuming partial or total end restraint.
- Derive the thickness such that the diameter/thickness ratio lies between 20 and 70 and eliminate any hydrostatic buckle tendency.

Deck Framing

- Select spacing between stiffeners (typically 500–800 mm).
- Derive the plate thickness from formulae accounting for local plastification under the wheel footprint of the design forklift truck.
- Determine by straight beam formulae the sizes of the main girders under 'blanket' live loads and/or the respective weight of the heaviest equipment.

The static in-place analysis is the basic and generally the simplest of all analyses. The structure is modeled as it stands during its operational life and subjected to pseudo-static loads. This analysis is always carried at the very early stage of the project, often from a simplified model, to size the main elements of the structure. The main model should account for eccentricities and local reinforcements at the joints. For example, a typical model for North Sea jacket may feature over 800 nodes and 4,000 members. The contribution of appurtenances, such as risers, J-tubes, caissons, conductors, boat-fenders, etc., to the overall stiffness of the structure is normally neglected. They are therefore analyzed separately and their reactions applied as loads at the interfaces with the main structure. Since their behavior is nonlinear, foundations are often analyzed separately from the structural model. They are represented by an equivalent load-dependent secant stiffness matrix; coefficients are determined by an iterative process where the forces and displacements at the common boundaries of structural and foundation models are equated. This matrix may need to be adjusted to the mean reaction corresponding to each loading condition. The static in-place analysis is performed under different conditions where the loads are approximated by their pseudo-static equivalent. The basic loads relevant to a given condition are multiplied by the appropriate load factors and combined to produce the most severe effect in each individual element of the structure. A dynamic analysis is normally mandatory for every offshore structure, but can be restricted to the main modes in the case of stiff structures.

2.18 Fabrication and Installation Loads

These loads are temporary and arise during fabrication and installation of the platform or its components. During fabrication, various structural components generate lifting forces, while in the installation, phase forces are generated during platform load-out, transportation to the site, launching and upending, as well as during lifts related to installation. According to the Det Norske Veritas (DNV) rules, the return period for computing design environmental conditions for installation and fabrication loads is three times as that of the duration of the corresponding phase. API-RP2A, on the other hand, leaves this design return period up to the owner, while the BS6235 rules recommend a minimum recurrence interval of 10 years for the design environmental loads associated with transportation of the structure to the offshore site.

2.19 Lifting Force

Lifting forces are functions of the weight of the structural component being lifted, the number and location of lifting eyes used for the lift, the angle between each sling and the vertical axis, and the conditions under which the lift is performed, as shown in Fig. 2.4. All members and connections of a lifted component must be designed for the forces resulting from static equilibrium of the lifted weight and the sling tensions. Moreover, API-RP2A recommends that in order to compensate for any side movements, lifting eyes and the connections to the supporting structural members should be designed for the combined action of the static sling load and a horizontal force equal to 5 % this load, applied perpendicular to the padeye at the center of the pinhole. All these design forces are applied as static loads if the lifts are performed in the fabrication yard. If, however, the lifting derrick or the structure

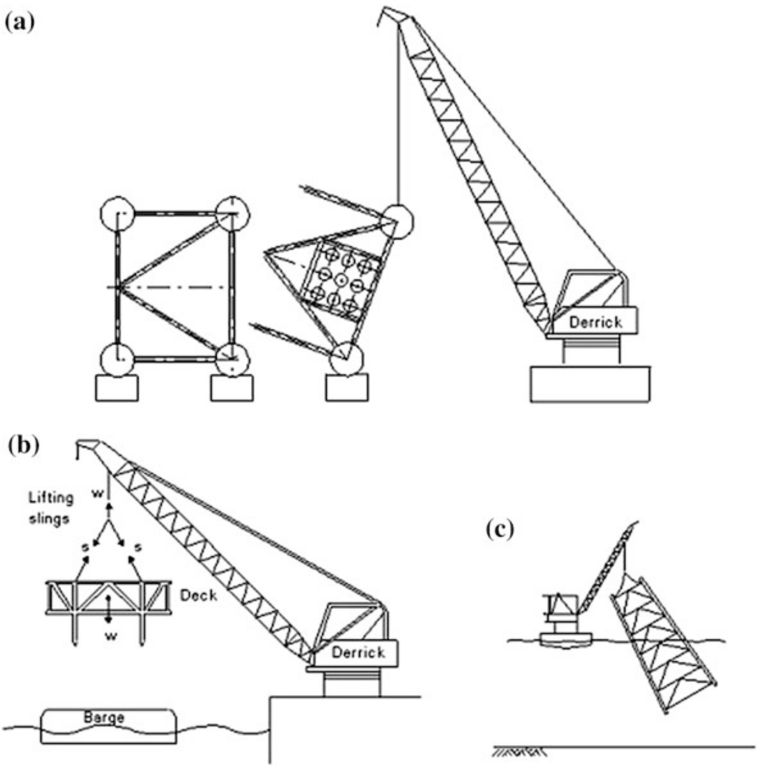


Fig. 2.4 Lifts under different conditions. **a** Derrick and structure on land. **b** Derrick on land, structure on floating barge. **c** Derrick and structure in the sea

to be lifted is on a floating vessel, then dynamic load factors should be applied to the static lifting forces. A factor of 2 is applied for members and connections and 1.35 for all other secondary members. For load-out at sheltered locations, the corresponding minimum load factors for the two groups of structural components are 1.5 and 1.15, respectively.

2.20 Load-Out Force

These are forces generated when the jacket is loaded from the fabrication yard onto the barge. If the load-out is carried out by direct lift, then, unless the lifting arrangement is different from that to be used for installation, lifting forces need not be computed. This is because lifting in the open sea creates a more severe loading condition, which requires higher dynamic load factors. If load-out is done by skidding the structure onto the barge, a number of static loading conditions must be considered, with the jacket supported on its side. Such loading conditions arise from the different positions of the jacket during the load-out phases as shown in Fig. 2.5. Since movement of the jacket is slow, all loading conditions can be taken as static.

Typical values of friction coefficients for the calculation of skidding forces are as follows: (i) steel on steel without lubrication (0.25); (ii) steel on steel with lubrication (0.15); (iii) steel on Teflon (0.10); and (iv) Teflon on Teflon (0.08). A typical ballast and displacement values are indicated in the figure.

2.21 Transportation Forces

These forces are generated when platform components (jacket, deck) are transported offshore on barges or self-floating. They depend upon the weight, geometry, and support conditions of the structure (by barge or by buoyancy) and also on the environmental conditions (waves, winds, and currents) that are encountered during transportation. The types of motion that a floating structure may experience are shown schematically in Fig. 2.6.

In order to minimize the associated risks and secure safe transport from the fabrication yard to the platform site, it is important to plan the operation carefully by considering the following (API-RP2A):

- Previous experience along the tow route
- Exposure time and reliability of predicted 'weather windows'
- Accessibility of safe havens
- Seasonal weather system
- Appropriate return period for determining design wind, wave, and current conditions, taking into account the characteristics of the tow such as size, structure, sensitivity, and cost.

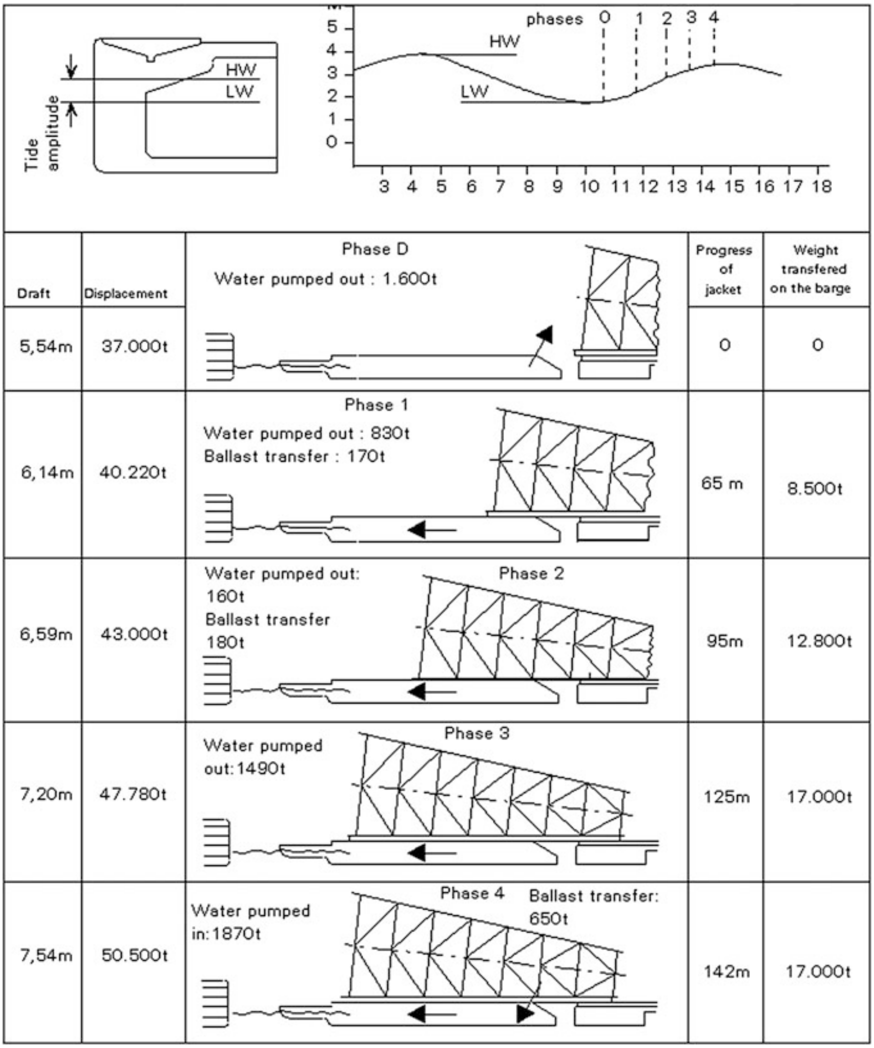


Fig. 2.5 Different phases of jacket load-out by skidding

The motion of the tow, i.e., the structure and supporting barge, generates transportation forces. They are determined from the design winds, waves, and currents. If the structure is self-floating, the loads are calculated directly. According to API-RP2A, towing analyses must be based on the results of model basin tests or appropriate analytical methods and must consider wind and wave directions parallel, perpendicular, and at 45° to the tow axis. Inertial loads shall be computed from a rigid body analysis of the tow by combining roll and pitch with heave motions, when the size of the tow, magnitude of the sea state, and experience make

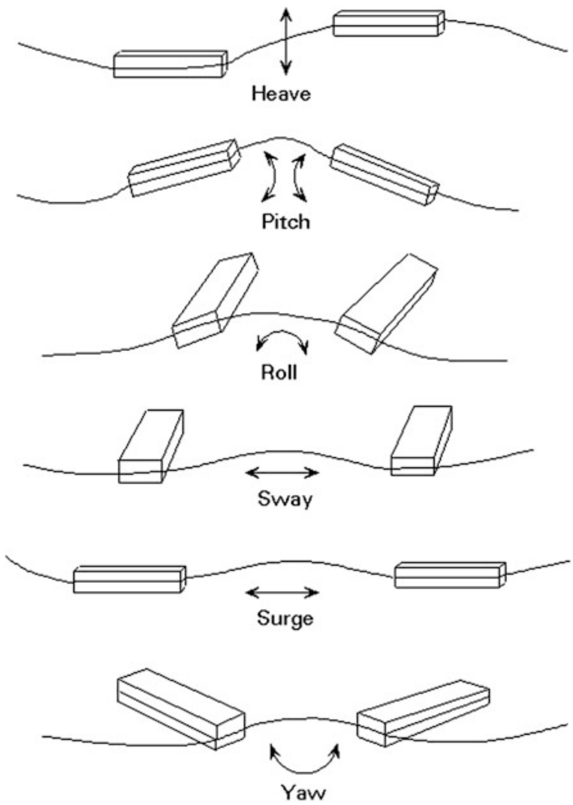


Fig. 2.6 Motion of floating objects during installation

such assumptions reasonable. For open sea conditions, typical values are 20° (for single amplitude roll motion) and 10° for single amplitude pitch motion. The period of roll or pitch is taken as 10 s, while heave acceleration is taken as 0.2 g. When transporting a large jacket by barge, stability against capsizing is a primary design consideration because of the high center of gravity of the jacket. Moreover, the relative stiffness of jacket and barge may need to be taken into account together with the wave slamming forces that could result during a heavy roll motion of the tow, as shown in Fig. 2.7. Structural analyses are carried out for designing the tie-down braces and the jacket members affected by the induced loads.

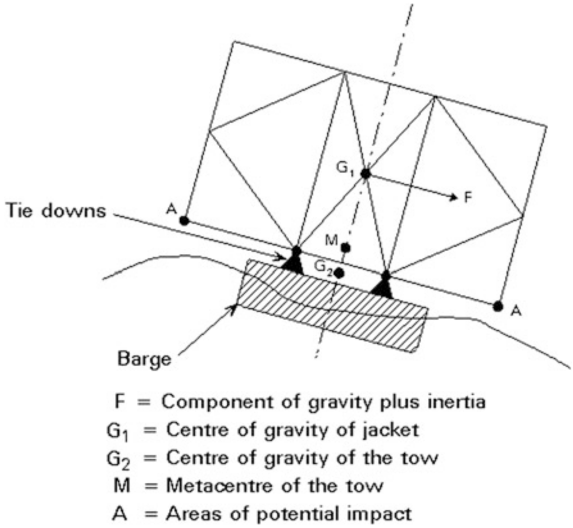


Fig. 2.7 View of launch barge and jacket undergoing motion

2.22 Launching and Upending Force

These forces are generated during the launch of a jacket from the barge into the sea and during the subsequent upending into its proper vertical position to rest on the seabed. A schematic view of the five stages the operation can be seen in Fig. 2.8.

Five stages in a launch-upending operation are as follows: (i) jacket slides along the skid beams; (ii) jacket rotates on the rocker arms; (iii) jacket rotates and slides simultaneously; (iv) detaches completely and comes to its floating equilibrium position; and (v) jacket is upended by a combination of controlled flooding and simultaneous lifting by a derrick barge. Both the static and dynamic loads for each

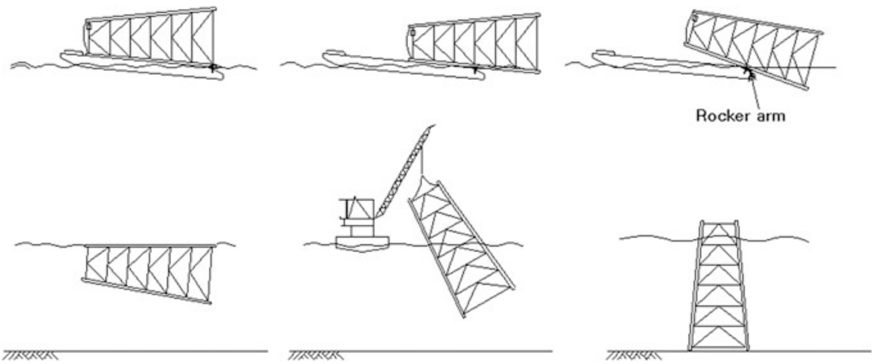


Fig. 2.8 Launching and upending

stage of the above under the action of wind, waves, and current need to be included in the analysis.

To start the launch, the barge must be ballasted to an appropriate draft and trim angle and subsequently the jacket must be pulled toward the stern by a winch. Sliding of the jacket starts as soon as the downward force (gravity component and winch pull) exceeds the friction force. As the jacket slides, its weight is supported on the two legs that are part of the launch trusses. The support length keeps decreasing and reaches a minimum, equal to the length of the rocker beams, when rotation starts. It is generally at this instant that the most severe launching forces develop as reactions to the weight of the jacket. During the last two stages, variable hydrostatic forces arise, which have to be considered at all members affected. Buoyancy calculations are required for every stage of the operation to ensure fully controlled, stable motion. Computer programs are available to perform the stress analyses required for launching and upending and also to portray the whole operation graphically.

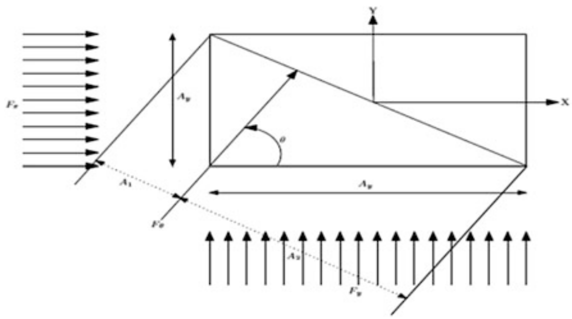
2.23 Accidental Load

According to the DNV rules, accidental loads are ill-defined with respect to intensity and frequency, which may occur as a result of an accident or exceptional circumstances. Examples of accidental loads are loads due to collision with vessels, fire or explosion, dropped objects, and unintended flooding of buoyancy tanks. Special measures are normally taken to reduce the risk from accidental loads. For example, protection of wellheads or other critical equipment from a dropped object can be provided by specially designed, impact resistant covers. An accidental load can be disregarded if its annual probability of occurrence is less than 10^{-4}. This number is the estimate of order of magnitude and is extremely difficult to compute.

Exercise

1. The design stages of various offshore platform are __________, ____________, and _______________.
2. The data collected during the FEED stage will be further verified to make sure the __________ and __________ of such data for further use.
3. Loads on offshore structures are _____________ and ____________.
4. Gravity loads are arising from __________ and ___________either permanent or temporary.
5. Seismic loads are arising from derived type ____________.
6. Gravity loads are ______________, _______________, ______________, _______________, and ____________.

7. Environmental loads are ____________, _____________, ____________, _____________, __________, and ___________.
8. The fluid loads are weight of fluid on the platform during ______________.
9. The wind speed at 10 m above _______________ is normally provided (V_0).
10. Wind speed obtained shall be ____________ to the height above for the calculation of wind speed.
11. The wind speed may be classified as ____________ and _______________.
12. ____________ are measured in an average less than 1 min in duration.
13. Wind loads shall be calculated as per ___________ guidelines.
14. Wind-driven waves are a major source of ______________on offshore platforms.
15. Sustained wind speeds measured for ______________ duration shall be used to compute global platform wind loads and gust wind which is measured for _________ duration shall be used to compute the wind loads to design individual members.
16. The wind pressure can be calculated as f_w = __________________.
17. The total force on the platform can be calculated as F_x = ______________.
18.



Calculate wind load on oblique directions F_θ =

19. In_____________________, a discrete set of design waves (maximum) and associated periods will be selected to generate loads on the structure.
20. In the spectral method, an energy spectrum of the sea state for the location will be taken and a ___________________ for the response will be generated.
21. Transfer function will be used to compute the ___________ in the structural members.
22. Sea-state energy spectra available for use are ________, ___________, and ___________.
23. Tides may be classified as___________, _____________, and _________________.
24. Combination of astronomical tide, wind tide, and pressure differential tide are called ____________.
25. The forces exerted by waves are most dominant in governing the jacket structures design especially the______________.

26. The wave loads exerted on the jacket is applied laterally on all members, and it generates ___________________ on the structure.
27. Period of wind-generated waves in the open sea can be in the order of ________________.
28. Waves are called _______________ and contain most part of _____________.
29. Relationship between significant wave height (H_s) and the maximum wave height (H_{max}) is _____________________.
30. Match the design wave height for various regions is tabulated below:

I. Bay of Bengal	(a) 11 m for 1 year and 24 m for 100 years
II. Gulf of Mexico	(b) 6 m for 1 year and 12 m for 100 years
III. South China Sea	(c) 5 m for 1 year and 12 m for 100 years
IV. Arabian Sea	(d) 14 m for 1 year and 22 m for 100 years
V. Gulf of Thailand	(e) 12 m for 1 year and 24 m for 100 years
VI. Persian Gulf	(f) 8 m for 1 year and 18 m for 100 years
VII. North Sea	(g) 8 m for 1 year and 18 m for 100 years

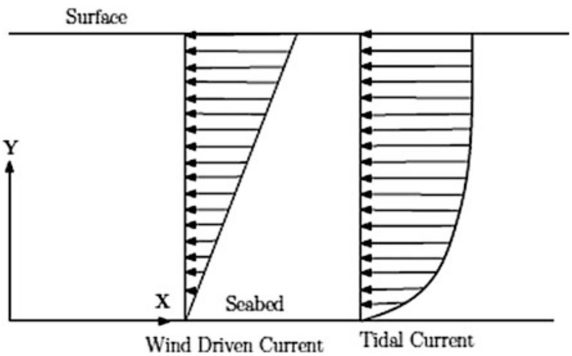
31. Draw the current profile of wave-driven currents and tidal currents.
32. Name some standard spectrum available in the literature.
33. Ocean currents are classified into few types based on their nature they are ________________, ____________________, and _________________________________.
34. Write down the expression for current variation along the depth and explain the terms involved in it.
35. Marine growth is an important part in increasing the loads on _________________.
36. Growth of marine algae increases the _______________ and ___________________which in turn increase the wave or current loading.
37. The thickness of marine growth generally ______________ with depth from the mean sea level, and it is maximum in the ______________.
38. Splash Zone is a region where the water levels ________________ between low to high.
39. Write down the expression for Morison equation and explain the terms appropriately.
40. Algebraic sum of wave and current loads is different from calculation of load by adding the horizontal water particle velocity with the current velocity and computing the loads. This is because of ____.
41. Interaction between the wave and current modifies the ______________________________.
42. Name some wave theories.
43. API-RP2A recommends to use a chart for such selection based on ____________ and _______________.
44. The reserve buoyancy is defined as buoyancy in _______________________.

45. The buoyancy force can be calculated using ________________method and ______________ method.
46. Sketch a jacket structure and mark the MSL, LAT, and HAT in the jacket structure.
47. Write down the empirical equation to estimate the force F_{ice}.
48. Platforms located in the vicinity of the river mouth will be subjected to ______________.
49. Write down the empirical equation to estimate the force F_{mud}.
50. Scour is removal of seafloor soils caused by ____________ and __________________.
51. Explain the force regime.
52. P-M spectral method describes the fully developed sea determined by _______________ parameter that is __________________.
53. In P-M spectrum, fetch and duration are considered ______________.
54. ______________________spectrum is on basis of the assumption that the spectrum is narrow banded, and individual wave height and wave period follow Rayleigh distribution.
55. ISSC spectrum suggested modification in form of the ____________________________.
56. _____________________ proposed modification in the P-M spectrum in terms of ________________________ and ____________________.
57. Write down the expression for Morison equation for force per unit length experienced by the structure due to its motion through the water.
58. Site-dependent databases are being developed to characterize the time varying fluid induced loads of ________, ___________, and ______________.
59. Explain the linear wave theory with the neat sketch and write the expression for velocity potential.
60. List the assumptions based on which the Morison equation is derived.

Answers

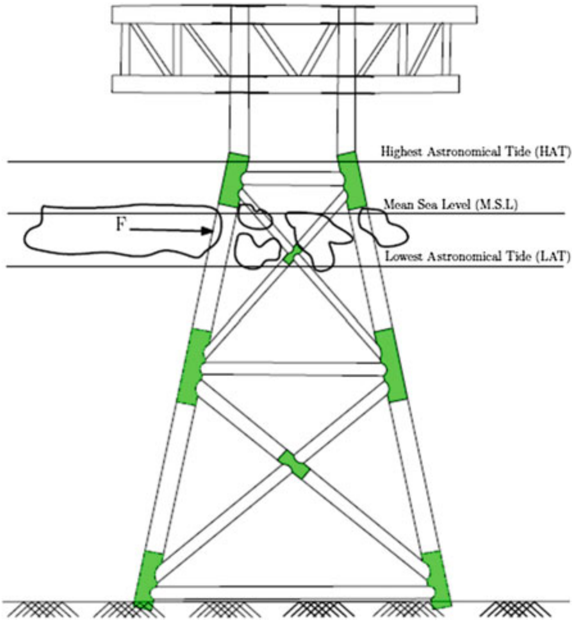
1. front end engineering design (FEED) or concept selection; basic design; detailed design
2. authenticity and reliability
3. gravity loads and environmental loads
4. dead weight of structure and facilities
5. gravity loads
6. structural dead loads; facility dead loads; fluid loads; live loads; and drilling loads
7. wind loads; wave loads; current loads; buoyancy loads; ice loads; and mud loads
8. operation

9. lowest astronomical tide (LAT)
10. extrapolated
11. gust and sustained wind velocity
12. Gusts
13. API-RP2A
14. environmental forces
15. 10 min average; 3 s
16. 1/2 ($\rho g V^2$)
17. $F_\theta F_x f_w A_x C_s$
18. $F_x \cos(\theta) + F_y \sin(\theta)$
19. design wave method
20. transfer function
21. stresses
22. PM spectra, Johnswap spectra, and ISSC spectra
23. astronomical tide, wind tide, and pressure differential tide
24. storm surge
25. foundation piles
26. overturning moment
27. 2–20 s
28. gravity waves and wave energy
29. $H_{max} = 1{:}86\ H_s$
30. I (f/g); II (e); III (a); IV (f/g); V (b); VI (c); VII (d)
31.



32. PM spectra; Johnswap spectra; and ISSC spectra
33. tidal current, wind-driven current, and current generated due to ocean circulation
34. $V_T = V_0 \left(\frac{y}{h}\right)^{1/7}$ V_T is the tidal current at any height from seabed, V_0 is the tidal current at the surface, y is the distance measure in meter from seabed, and h is the water depth
35. offshore structures
36. diameter and roughness of members

37. decrease and splash zone
38. fluctuate
39. Morison equation $F_T = F_D + F_I = \left(\frac{1}{2} C_D \rho_w D V|V| + \frac{\pi}{4} D^2 C_M \rho_w a\right)$
where F_T is the total force, ρ_w is the density of water, C_D and C_M are the drag and inertia coefficients, respectively, D is the diameter of the member including marine growth, V is the velocity, and a is the acceleration
40. nonlinear term in the drag equation
41. wave parameters and wave field
42. linear/Airy wave theory; Stokes wave theory (up to 5th order approximations); stream function wave theory (up to 22nd order approximations); cnoidal wave theory
43. d/gT^2 and H/gT^2
44. excess of its weight
45. marine method and rational method
46.



47. $f_{ice} = C\,f\,A$ where f_{ice} crushing strength of ice varies between 1.5 and 3.5 MPa; C_{ice} = ice force coefficient varies between 0.3 and 0.7; and A area struck by ice (diameter of member x ice sheet thickness)
48. mud load
49. Mud loads can be calculated using $F_{mud} = C_{mud}\ \tau\ D$ where C_{mud} = force coefficient varies from 7 to 9; τ = shear strength of soil 5–10 kPa; and D = Diameter of pile or member
50. currents and waves
51. 1. $D/L > 1$ condition approximate to pure reflection; $D/L > 0.2$ diffraction is increasingly important; $D/L < 0.2$ diffraction is negligible; $D/L_0 > 0.2$ inertia; $D/L_0 < 0.2$ drag dominant where D is the diameter of the structure; L is the wave length; and L_0 is the deepwater wavelength
52. one; wind speed
53. infinite
54. Bretschneider
55. Bretschneider spectrum
56. International Towing Tank Conference (ITTC); significant wave height and zero crossing frequency
57.

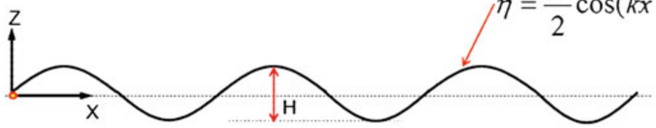
$$f = m\ddot{x} + C_A\rho\frac{\pi}{4}D^2\ddot{x} + \frac{1}{2}C_D\rho D|\dot{x}|\dot{x}$$

58. wind, wave, and current
59.

Linear Wave Theory

Airy wave theory is considered in the calculation of wave kinematics. Consider a progressive wave with water surface elevation depicted by cosine curve,

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



and the corresponding velocity potential is given by:

$$\phi = -\frac{H}{2}\frac{\omega}{k}\frac{\cosh k(h+z)}{\sinh kh}\sin(kx - \omega t)$$

60. Assumptions are as follows:

- Flow is assumed to be not disturbed by the presence of the structure.
- Force calculation is empirical calibrated by experimental results.
- Suitable coefficients need to be used depending on the shape of the body or structure.
- Validity range shall be checked before use and generally suitable for most jacket type structures where $D/L \ll 0.2$ where D is the diameter of the structural member and L is the wave length.

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