Chapter 2
Offshore Structures Versus Land-Based Structures

2.1 Introduction to Offshore Structures

2.1.1 Offshore Platforms

Offshore structures (Fig. 2.1) with their facilities are used to drill wells, to extract and process oil and natural gas, or to temporarily store product until it can be brought to shore for refining and marketing. In many cases, an offshore platform also contains facilities (e.g., living quarters) to house the work-force as well. They are defined by either their functions or their circumstances and configurations [1]. A brief review of the different types of platforms with their structural forms and uses, and the developments, are presented in a few literatures [2–4].

The functions of an offshore structure may be one of the following (even though multiple functions may be possible for a structure):

- Exploratory Drilling Structures: a Mobile Offshore Drilling Unit (MODU) configuration is largely determined by the variable deck payload and transit speed requirements.
- Production Structures: a production unit can have several functions, e.g., processing, drilling, workover, accommodation, oil storage and riser support.
- Storage Structures: used for storing the crude oil temporarily at the offshore site before its transportation to the shore for processing.

Depending on the circumstances and configuration, offshore platforms may be fixed to the ocean floor, which is typically referred to as fixed offshore platforms, including the conventional fixed jacket platforms (shown in Fig. 2.1) for a water depth of less than 400 m, jackups (shown in Fig. 2.34) being placed in relatively shallow waters (less than 150 m), compliant piled tower (shown in Fig. 2.1) that can be used for water depths up to 900 m, and an artificial island (Fig. 2.2). Offshore platforms may also be floating structures, including buoyant (e.g., semi-submersibles shown in Fig. 2.3, FPSO, and spars shown in Fig. 2.1) and

J. Jia, *Modern Earthquake Engineering*, DOI 10.1007/978-3-642-31854-2_2

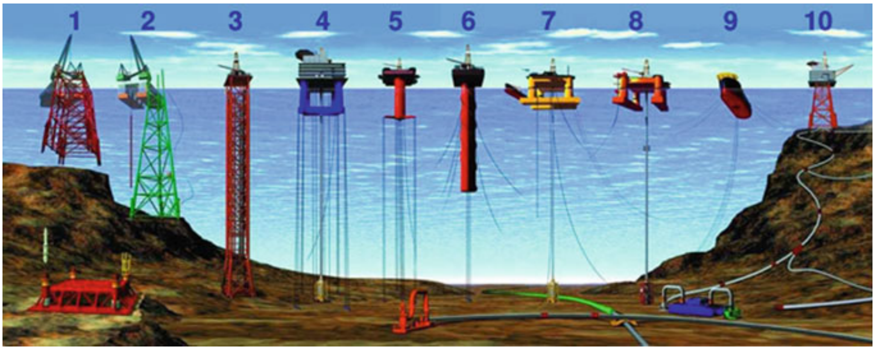


Fig. 2.1 Various types of offshore structures. *1* and *2* conventional fixed jacket platforms; *3* compliant piled tower; *4* and *5* vertically moored tension leg and mini-tension leg platform (TLP); *6* Single point anchor reservoir platform (SPAR); *7* and *8* semi-submersibles; *9* floating production, storage, and offloading facility (FPSO); *10* subsea completion and tie-back to host facility (courtesy of US National Oceanic and Atmospheric Administration)

Fig. 2.2 An artificial island (Northstar Island) for oil drilling in the Beaufort Sea

positively buoyant (e.g., tension leg platforms shown in Fig. 2.1). In addition, remote subsea wells, as shown in Fig. 2.4, may also be connected to a platform by flow lines and by umbilical connections. These subsea solutions may consist of one or more subsea wells, or of one or more manifold centers for multiple wells. It is noted that the function, water depth, and environmental loading are essential factors to influence the structural design concepts for offshore platforms.

For fixed offshore structures, they can be piled (Fig. 2.8) or gravity based (GBS, as shown in Fig. 2.9), or a compliant (number 3 in Fig. 2.1) or articulated structure. A significant advantage of fixed platforms is that they use conventional well systems that developed along with platforms, leading to an economical design until water depths increase to the point where the cost of the structure outweighs the savings from the well system. These platforms use the minimum possible amount of

Fig. 2.3 A floating drilling platform Aker H6e (courtesy of Aker Solutions)

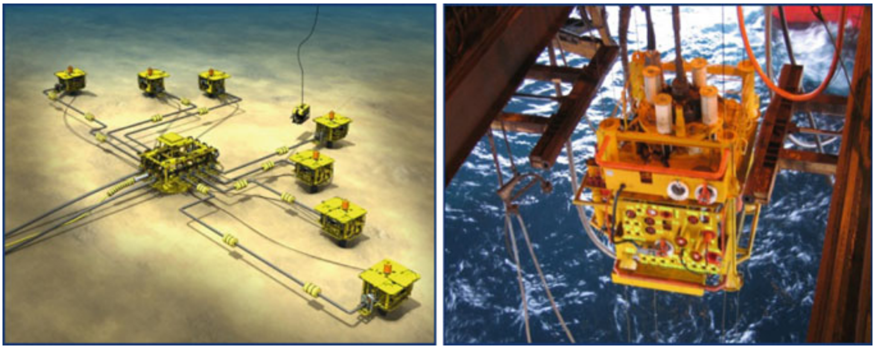


Fig. 2.4 Subsea wells (courtesy of Aker Solutions)

material and expose the least possible area to environmental loads. The most widely used fixed offshore structures are jacket structures as shown in Fig. 2.8. They are especially advantageous when operated at offshore sites with soft soil condition. The pile foundations supporting the jacket structures are put in place and connected to each leg to support the jacket. This leads to a transformation of the moment loading at mudline caused by lateral loading due to earthquake, or ocean wave and wind loading applied at superstructure, into axial forces in the jacket piles [5], as illustrated in Fig. 2.5. Therefore, jacket piles are usually insensitive to the lateral loads. Jacket piles can support a significant amount of load from substructure. Moreover, jacket structures can be constructed in sections and transported, making it more efficient for construction. Gravity based structures, which rely on the weight of the structure itself to resist the environmental loading, are also adopted, particularly in the North Sea. They are frequently installed at sites where driving piles

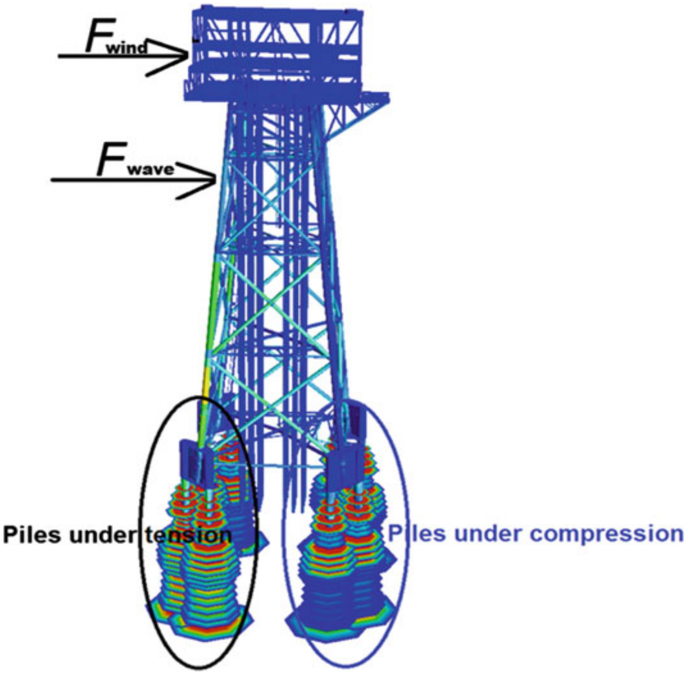


Fig. 2.5 Piles subjected to tensile and compressive axial loading due to the lateral wind and wave loading transferred from the *upper part* of the structure

becomes difficult. These structures have foundation elements that contribute significantly to the required weight and spread over a large area of the sea floor to prevent failure due to overturning moments caused by lateral loads. They are capable of supporting large topside loads during towing out, thus minimizing the hook-up work during installation. Moreover, their merits also include (1) construction onshore for transport; (2) towing to the site of installation; (3) quick installation by flooding; and (4) use of traditional methods and labor for installation [6]. GBS is typically constructed with reinforced concrete and consists of a large cellular base surrounding several unbraced columns that extend upward from the base to support the deck and equipment above the water surface. Gravity based platforms consist of production risers as well as oil supply and discharge lines contained in one of the columns; the corresponding piping system for exchange of water is installed in another column, and drilling takes place through the third column [6]. This particular type is referred to as a CONDEEP (concrete deep water) structure. Gravity based structures are particularly suitable to resist enormous horizontal loading such as ice loading, in which a jacket or a jackup structure would not offer the required global capacity to resist ice or ice ridge loading or is forced to limit operations (in case of a mobile jackup structure) to a short summer season, which may damage legs and/or unprotected conductors/risers (conductors/risers are typically installed inside the concrete shaft in a GBS). It should also be noted that the gravity based platform can also be constructed with steel instead of concrete, such as the Maureen Alpha steel gravity platform in the United Kingdom. Table 2.1 lists the water depths of typical GBS platforms operated in the North Sea, with the highest one being Troll A platform (Fig. 2.6) with a total height of 481 m. It is operated at a water depth of 330 m, and has a bottom dimension of 160 m × 60 m and a natural period of 4 s. Troll A is also the tallest structure that has ever been moved on Earth.

As the water depth increases, bottom supported compliant structures are more economical. As this type of structure is rather flexible, they move with the dynamic environmental loads such as winds, waves and currents to a limited extent, rather than resisting them as a GBS or a jacket structure does. Therefore, the compliant platforms resist lateral environmental loading by their relative movements instead of their weight as a GBS structure does. These types of structures are attached to the seabed by means of tension legs, guy lines, flexible members or articulated joints. The buoyancy force or the force of elasticity of the axially stressed legs generates the restoring forces. As the system is not soft, the fundamental natural frequency remains low. The designs are technically and economically feasible as they increase

Table 2.1 Water depths of representative gravity based platforms installed in the North Sea

Platform	Water depth (m)
Troll A	330
Gulfaks A	133
Gulfaks B	133
Gulfaks C	214
Oseberg North	100
Oseberg A	100
Ekofisk 1	70
Draugen	251
Heidrun	350
Statfjord A	145
Statfjord B	145
Statfjord C	145
Frigg TP 1	104
Frigg TP 2	103
Frigg MCP01	94
Frigg CDP1	98
Frigg TCP2	104
Brent B	140
Brent C	141
Brent D	142
Cormorant A	149
Dunlin A	153
Beryl A	118

Fig. 2.6 Troll A platform

the natural periods of the structure to such an extent that typical storm wave periods are far below the natural periods of the structures. Examples of bottom supported compliant platforms are compliant towers (3 in Fig. 2.1), guyed towers (Fig. 2.7), buoyant towers, flexible towers, articulated towers, and hybrid compliant platforms. Given the discussion above, it would be assumed that the load distribution and transfer mechanism is quite different between the fixed and the compliant structures [7]: in a fixed structure the static and dynamic forces are almost all transmitted to the seafloor; while for the compliant structures, the horizontal dynamic loads are counteracted by the inertia forces. This reduces the internal forces in the structure as well as the support reactions. The vertical dynamic loads are transferred to the seafloor, much as with a fixed offshore structure (Figs. 2.8 and 2.9).

As the water depth further increases and the sites are sometimes located far off the continental shelf, floating structures are more used because they are economically attractive for deep water sites with a reduced structural weight compared to conventional fixed and bottom supported platforms. Floating structures resist loads by undergoing large excursions when subjected to environmental loads and thereby reducing the forces on the structures. However, as large motions are expected, geometric nonlinearity is an

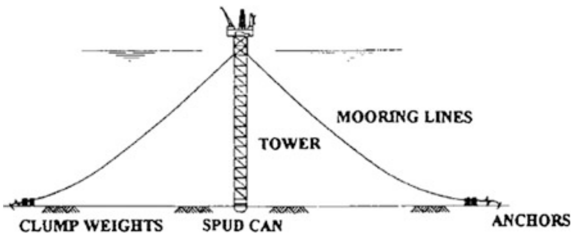


Fig. 2.7 Illustration of a guyed tower

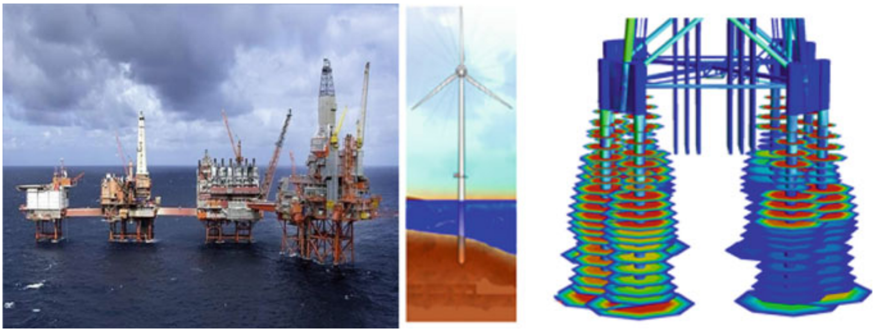


Fig. 2.8 Fixed jacket structures (*left*), pile foundations of a fixed monopile (*middle*) and of a jacket structure (*right*)

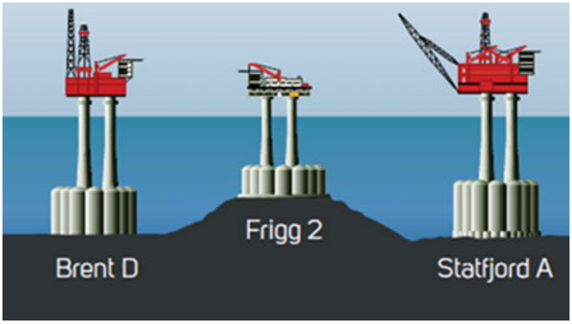


Fig. 2.9 Fixed GBS offshore structures with gravity based foundation design (courtesy of Aker Solutions)

important consideration in the analysis of floating structures. Floating structures are required to be moored in place. Subject to the environment loading, the floater remains within a specified circle of operation from a desired mean location, which is generally achieved by mooring lines or a dynamic positioning system [8]. Typical floating offshore structures include the semi-submersibles shown in Fig. 2.3, FPSO, and spars shown in Fig. 2.1, tension leg platforms (which are vertically restrained while horizontally compliant, permitting surge, sway and yaw motions) shown in Fig. 2.10 and tension buoyant towers. The selection process for the floater type generally depends on the following factors: well pattern, export methodology, service life and geographical region, gas to oil ratio, topside weight, well count and water depth. As floating structures have low stiffness, their natural periods of motions are higher than that of the wave loading, thus avoiding the resonance with wave loading. For example, typical natural periods of motions for a TLP are more than 100 s for surge and sway, more than 80 s for yaw, and 3–5 s for heave, pitch and roll.

For platforms operating in deep waters, since the distance between platforms and ports increases, deck spaces of the topside need to be designed with variable load capacities. For example, for drilling platforms, deck space is very important because more space is required for third-party equipment for well completion and

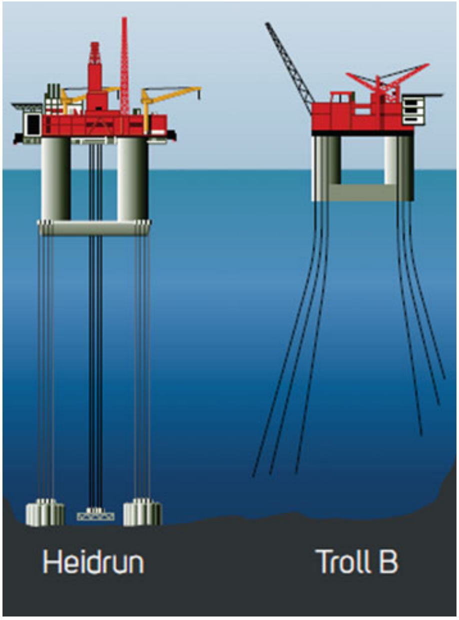


Fig. 2.10 A tension leg drilling platform (*left*) and a semi-submersible platform (*right*) (courtesy of Aker Solutions)

well testing activities, in addition to the space for drilling equipment. This typically requires that the topside be designed with variable load capacities between 10,000 and 20,000 t [6].

In addition to functionality, environmental loading and water depth, dynamic behavior and performance of each type of offshore structures also strongly influence the selection of structural form and design concept. As shown in Fig. 2.11, the natural periods corresponding to the global bending and global torsional vibrations for fixed offshore structures and the period of axial leg tension vibrations of TLPs should be below the period of ocean wave and wind loading, thus avoiding resonance conditions, which would otherwise cause excessive dynamic responses. On

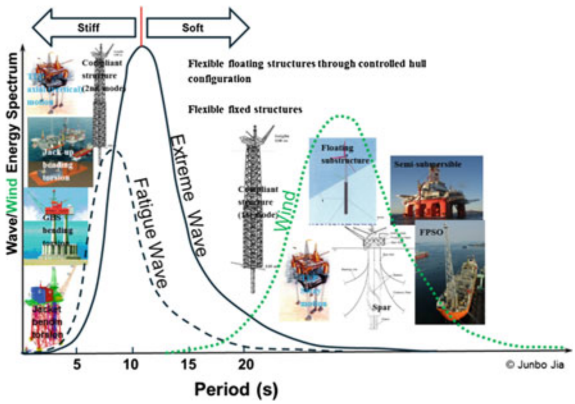


Fig. 2.11 Accounting of dynamics in the design of offshore structures (from a lecture “Dynamic Analysis and Design of Offshore Structures” by the author at OMAE 2016, Pusan)

the other hand, to avoid resonance motion responses of floating structures due to wave loading (normally the dominating loads for offshore structural designs), they generally have natural periods of motions higher than the period of ocean wave loadings, even though they may potentially reach a resonance condition due to dynamic loading caused by wind turbulence.

Typically, the design of fixed offshore structures has to consider the impact from earthquakes. The design of TLP also needs to consider the influence from the seismic loading transferred to tension leg(s) and the response of connected floating structures.

2.1.2 Offshore Wind Turbine Substructures and Foundations

Offshore sites provide a reliable source of strong winds due to the cooling and heating effect of water and land. Moreover, wind turbulence offshore is generally lower than that of inland sites. Therefore, due to the higher wind speed and lower wind turbulence offshore than over adjoining land, in recent years, offshore wind farms have rapidly developed across the world, dominated by developers in Europe and East Asia. In the meantime, the United States is also catching up by completing its first offshore wind farm with five steel jacket substructures (each carry a 6 MW wind turbine), installed at the Block Island Wind Farm site in December 2015. So far, more than 80 % of the operated and currently planned offshore wind farms are located in Europe.

A wind turbine-supporting structure system includes both rotor-nacelle assembly and support structure. The former includes rotor (blades and hub) and nacelle assembly (all components above tower except the rotor, including driven train, bed plate, yaw system, and nacelle enclosure, etc.). The latter include a tower (connecting the substructure to rotor-nacelle assembly), substructure (for fixed substructure, it extends upwards from the seabed and connects the foundation to the tower; for floating substructure, it is the part below the tower), and foundation (transferring various types of load acting on the structure into the seabed soil).

The substructures and foundations for offshore wind turbines (OWTs), as shown in Fig. 2.12, can be either fixed to the seabed, bottom supported compliant structures, or floating structures.

The types of fixed offshore structures can be gravity foundation, monopile, tripod-pile, tripile, jacket(braced frame)-pile, and suction bucket, as shown in Fig. 2.13. The basic types of floating structure concepts used for offshore wind turbines are TLP (Fig. 2.14), spar (Fig. 2.15), and semi-submersibles (Fig. 2.16). The world's first floating wind farm Hywind Scotland is planned to be completed by 2017. It lies 25 km off the northeast coast of Scotland near Peterhead and consists of five 6 MW spar type floating turbines operating in water depths of between 95 and 120 m. The concept designs using bottom supported compliant

Fig. 2.12 Wind turbines with their substructures and electrical substation located in the North Sea (license under CC BY-SA 3.0, by StekrueBe)

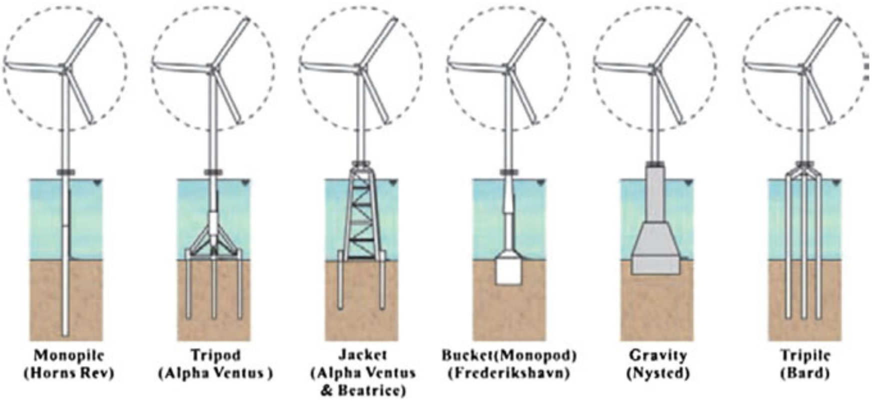


Fig. 2.13 The types of existing substructures and foundations for fixed offshore wind turbines [16]

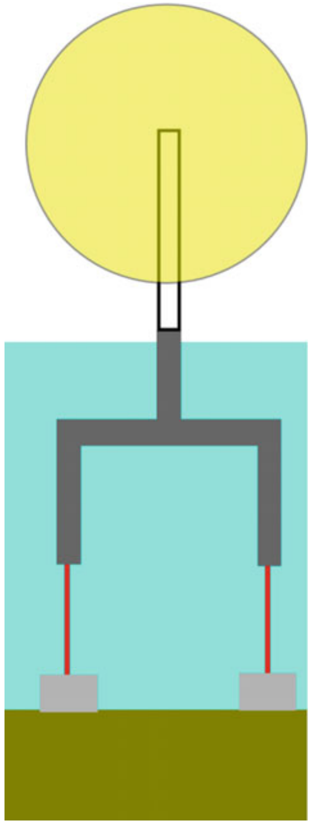


Fig. 2.14 Schematic illustration of a TLP used for an OWT substructure

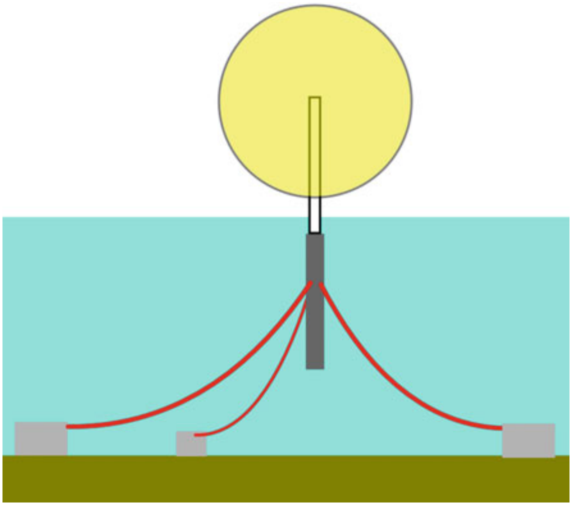


Fig. 2.15 Schematic illustration of a single floating cylindrical spar buoy moored by catenary cables

Fig. 2.16 WindFloat, which is a semi-submersible structure for an OWT operating at a rated capacity of 2 MW approximately 5 km offshore of Aguçadoura, Portugal (under license CC BY-SA 3.0 by untrakdrover)

structures for OWTs have not received much popularity, because the low frequency wind turbulence may interfere with the natural frequency of the compliant structure, potentially causing resonance. Moreover, the interaction between a compliant structure and the wind turbine can exert significant forces on the top of the relatively flexible structure, leading to significant deflections of the compliant structure.

Monopile currently represents the most common substructure application for OWT in shallow waters. It is a relatively simple design in which the tower to supporting turbines, made up of steel pipe, is supported by the monopole, either directly or through a transition piece. The vertical loads are transferred to the seabed by shaft friction and tip resistance. The vertical bearing capacity is therefore largely determined by the diameter of the mono pile (typically between 2 and 6 m), which attracts the lateral hydrodynamic loads due to wind, waves and current. These horizontal loads dominated by bending moments will be further transferred to the soil. Monopile is so far a cost competitive solution and suitable for shallow waters up to 30 m. One critical issue for the design of monopiles is the consideration of cyclic behavior of piles with large diameter. Investigations have shown that the horizontal deflections of large diameter monopiles are underestimated for extreme loads [5]. On the other hand, experiences from operating offshore wind farms supported by monopiles indicate that the foundation stiffness for small operational loads is significantly underestimated. The installation method involves lifting or floating the structure into position using equipment such as floating crane vessels, drilling jack-up units, and specially constructed installation vessels before driving the piles into the seabed.

Jacket designs are more and more utilized in the applications of OWT. As an example, jacket structures were chosen as substructures by the Moray Offshore Wind Farm project (1500 MW total) in the UK, with up to 8 MW for each offshore wind turbine. The obvious benefits (compare to a monopile) are the reduction in hydrodynamic loading and weight, and suitability for increased water depth (30 m+). Jacket structures are less dependent on soil conditions compared to monopile and tripod, and are more suitable for sites with soft soil conditions.

Fig. 2.17 A close look at a tripod used as a substructure for OWT (courtesy of Aker Solutions)

They are easily capable of supporting OWT with 6 MW or more. The smaller diameter of the piles (typically less than 2.5 m) also means a reduction in required driving power and less noise during piling. In addition, better precision for pile orientation and positioning can be ensured. By considering the financial limits, OWT using jacket works well for water depths up to 60 m.

The main part of the tripod, shown in Fig. 2.17, consists of a tubular pole, but the lower part consists of braces and legs. The tubular steel foundation piles are driven through the sleeves in the three legs. As a tripod has a large base surface, it performs well in resistance to overturning moments. Moreover, it also has more redundancy due to three installed piles rather than one (monopile) [9].

Gravity based foundations consist of a slender steel or concrete substructure mounted onto a single large reinforced concrete or a ballast-filled steel shell. To maintain stability, tensile loads between the bottom of the support structure and the seabed are resisted by self-weight of the foundation. The gravity based foundations require a flat base and scour protection. A significant advantage of gravity based foundations is associated with their transportation, as they can be fabricated and partially assembled in local yards, transported and completely installed at sea, depending on the fabrication yard capacity, the available draft during their transportation, and the availability of ballast materials.

Bucket foundations consist of a sub-structure column connected to an inverted steel bucket through flange-reinforced shear panels. The length of the skirt is normally in the same order as the bucket diameter, where the volume of soil inside the foundation may act as a permanent gravity base foundation. Bucket foundation is therefore similar to gravity base foundations in shape and size but differ in the method of installation and primary mode of stability. The installation of a bucket foundation is typically through pushing due to the weight of the foundation and its associated mass and/or creating a negative pressure inside the foundation to generate a downward pressure. These operations enable the foundation to penetrate

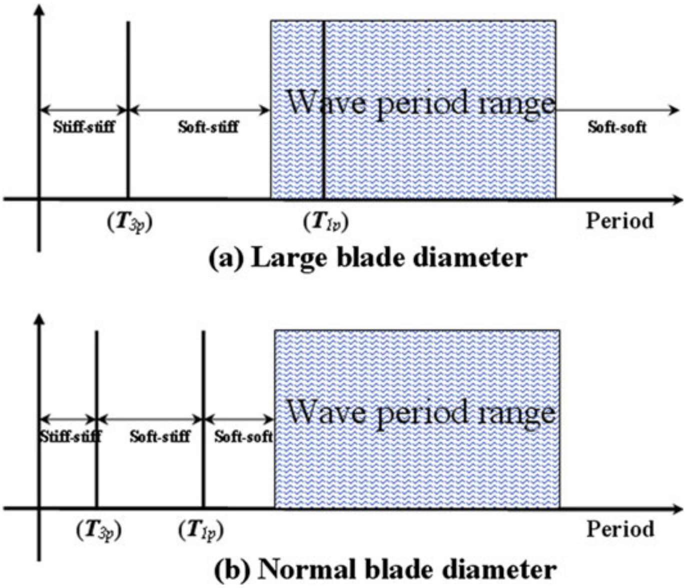


Fig. 2.18 Period interval for stiff-stiff, soft-stiff and soft-soft design of OWT substructures

into the seabed and to finally reach a desirable penetration depth. To create the negative pressure, water and air inside the foundation are pumped out through top of bucket when the rim of bucket seals with the seabed.

The dynamic characteristics of offshore wind turbine structures are slightly different from those of traditional offshore platform structures. An offshore wind turbine system comprises five physical components: rotor, transmission, generator, support structure, and control system. Each of these influences the dynamic behavior of the complete turbine system. It is noticed that the two most significant excitations are due to the rotation of blades and wave induced forces. In order to avoid resonance due to the excitations from wind turbine blade rotation and wave loadings, the support structure should be designed with a natural period as far as possible from the periods of both blade and wave excitations. This gives three possible natural period ranges for designing the support structure as shown in Fig. 2.18: a natural period larger than the first blade excitation period T_{1p} (soft-soft design), the one between the first (T_{1p}) and the second blade excitation period T_{3p} (soft-stiff design), and the one below the second blade excitation period T_{3p} (stiff-stiff design).

Traditionally, the soft-soft design is preferred because this usually leads to an economical design due to the need for less material and construction cost. In the offshore wind energy industry, the general trend is that the scale of turbines is becoming larger and larger. This would result in an increase of the blade's diameter, e.g., a 170 m diameter for a 7 MW wind turbine. The first and second excitation periods are also significantly increased for those large OWTs. This motivates the engineer to shift from a soft-soft to a soft-stiff or even to a stiff-stiff design. Moreover, the variable tip speed of the turbine also becomes a design alternative, which adds additional restrictions on the natural period range of the structures. In addition, it is also possible to convert the existing/abandoned offshore rigs into substructures for OWT systems, which can avoid/delay enormous decommissioning costs for energy companies as well as avoid cost and pollution for constructing new substructures for OWTs. Most of the existing fixed offshore platform structures have natural periods below 3.5 s; after removing part of the heavy topside modules at the top of the platforms, the natural periods will further decrease. This period range is then relevant to the soft-stiff or even soft-soft design. For developing this concept, one also needs to account for the cost with respect to maintenance and power grid integration.

Except artificial damping devices, an optimized control system and the associated control algorithm for both generator torque and pitch angle of wind turbine blades (either for a collective pitch control for all blades or an individual pitch control for each blade) can also mitigate dynamic loadings applied on an OWT tower. Such a control supplies additional damping to generally lightly-damped tower, and may even help to minimize both wind and wave induced platform motions. It normally involves a modeling of an extra pitch demand responsible for counter-balancing the vibrations and motions of the tower. This load reduction will allow for a more cost-effective structural design.

2.2 Accounting of Dynamics in the Concept Design of Structures

2.2.1 Dynamics Versus Statics

Over history, the safety and serviceability of structures have basically been measured on the basis of their static behavior, which required adequate stiffness and strength. This was perhaps because the necessary knowledge of dynamics was less accessible to engineers than their static counterpart. Nowadays, it is common knowledge that all bodies possessing stiffness and mass are capable of exhibiting dynamic behavior.

The major difference between dynamic and static responses is that dynamics involves the inertia forces associated with the accelerations at different parts of a structure throughout its motion. If one ignores the inertia force, the predicted responses can be erroneous. As an example, let's consider a bottom fixed cantilevered tower subjected to sea wave loadings as shown in Fig. 2.19 [10]. In addition to the static bending moment due to wave loadings applied on the structure, as shown in Fig. 2.19b, the stiffness and mass of the structure will react to the wave loadings and generate internal forces on both the top mass block (Q_i) and the tower (q_i), shown in Fig. 2.19c. Rather than a single function of mass, the amplitudes of the inertia forces are related to a ratio between stiffness and mass (eigenfrequency), mass, as well as damping, thus resulting in additional dynamic bending action (Fig. 2.19d).

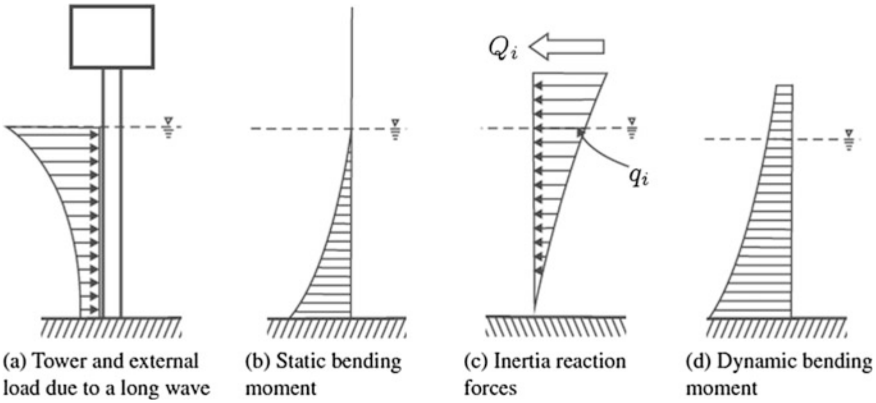
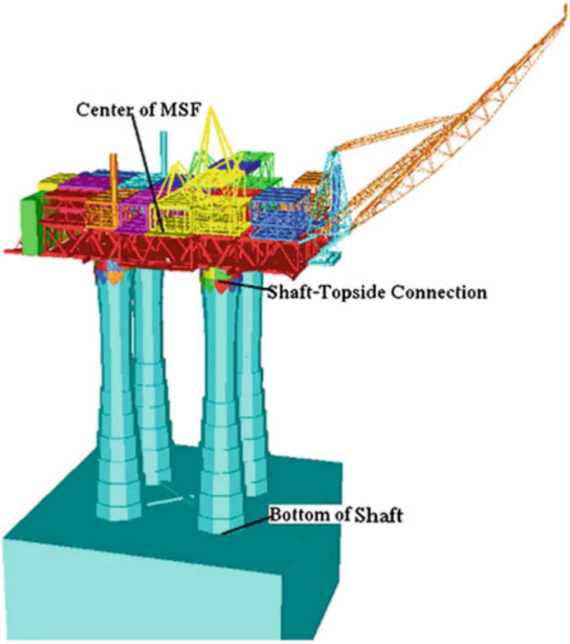


Fig. 2.19 Wave induced static versus instantaneous dynamic forces and moments in a bottom-fixed cantilevered tower [10]

Fig. 2.20 A GBS with a heavy topside supported by four concrete shafts (legs)



As another example, consider a gravity-based structure (GBS), shown in Fig. 2.20, that is subjected to the ground motions recorded during El Centro earthquake, which have a high energy content at the vibration period above 0.2 s (below 5 Hz in Fourier amplitude shown in Fig. 2.21, which will be explained in Sect. 5.3.1). The dynamic responses of the platform are investigated by varying the thickness of four shafts from half of the reference thickness, to the reference

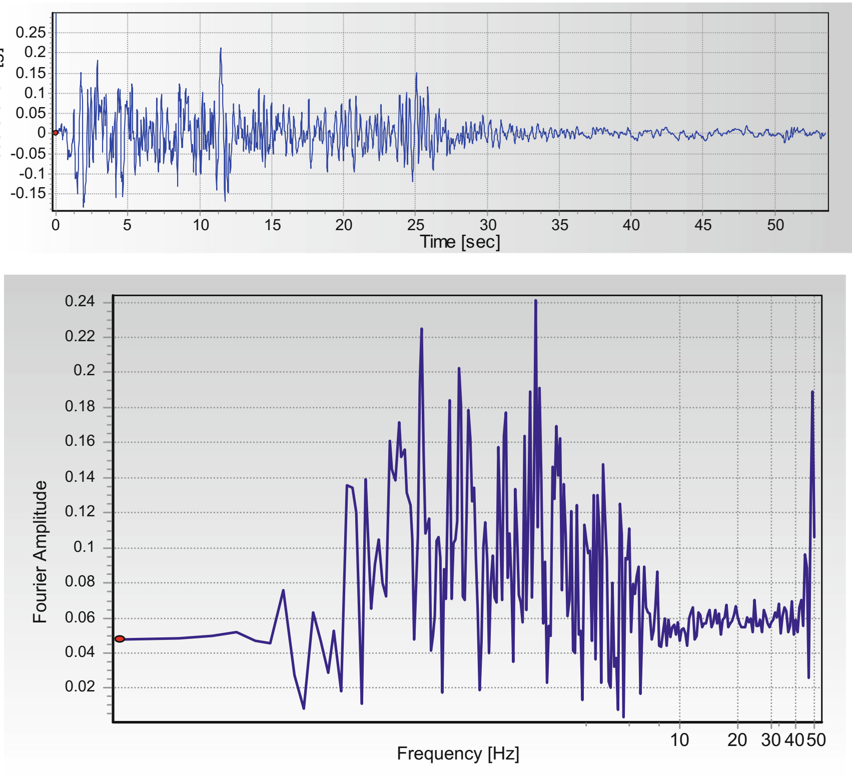
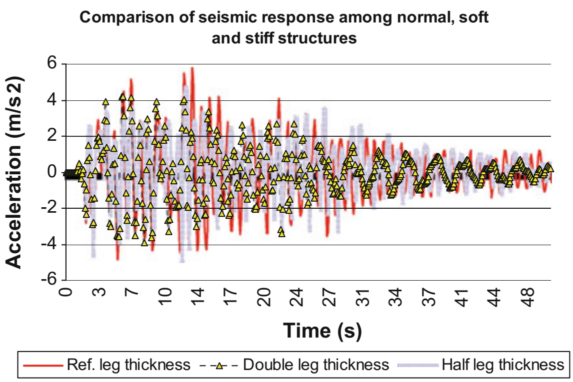


Fig. 2.21 Ground motions EW component (*upper*) recorded during El Centro earthquake and its Fourier amplitude (*lower*)

Fig. 2.22 Acceleration at the shaft-topside connection with various leg/shaft stiffness (peak acceleration: 4.7 m/s^2 for double leg thickness, 5.8 m/s^2 for reference leg thickness, 4.2 m/s^2 for double leg thickness)



thickness, to twice the reference thickness. It is obvious that the GBS becomes stiff by increasing the shafts' thickness. If a static analysis is performed, under the same seismic excitations the stiffer structure would have lower responses. However, the seismic responses involving dynamic effects may not obey this rule. Figure 2.22 shows the acceleration at the shaft-topside connection. It is clearly shown that the peak acceleration for the reference shaft thickness case is higher than that of the half-thickness case. However, the trend of peak acceleration response variation with the change of stiffness cannot be identified, as the peak acceleration for the double shaft thickness (the stiffest one) is lower than that for other cases with lower stiffness. This indicates the effects of inertia, which are more complex than their static counterpart. As will be discussed in Chap. 5 and Sect. 15.5, the response variation trend can be identified by relating the seismic responses to the dynamic characteristics of both structures and excitations.

Even for dynamic insensitive structures with low periods of resonance compared to that of the dynamic loading, dynamics does include the inertia effects due to loading that varies with time, even if this load variation may be quite slow. The inertia effects could lead to the fatigue failure of the materials at stress conditions well below the breaking strength of the materials. They may also be responsible for the discomfort of human beings. Figure 2.23 shows an offshore jacket structure subjected to two consequent sea waves; the jacket has a resonance period of 2.5 s.

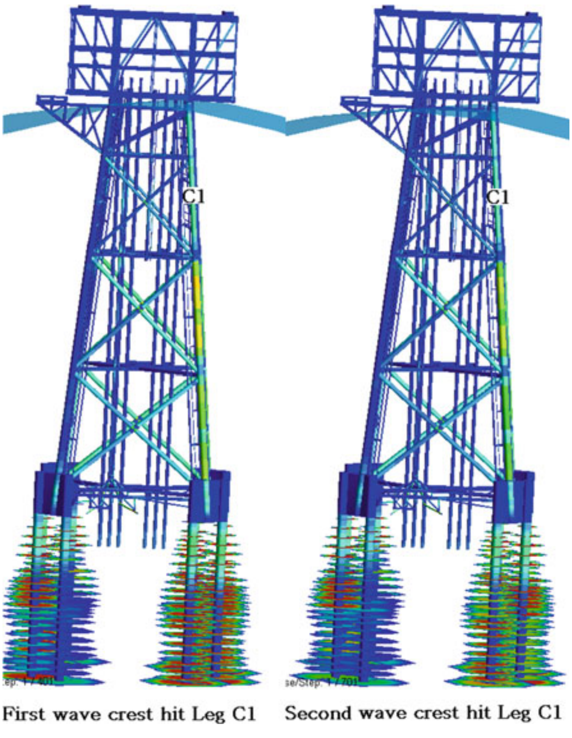


Fig. 2.23 An offshore jacket structure subjected to a wave with a wave height of 31.5 m and a wave peak period 15.6 s (courtesy of Aker Solutions)

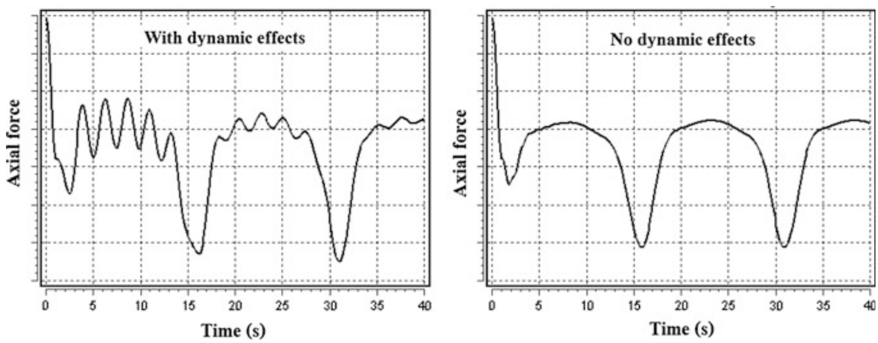


Fig. 2.24 Axial force time history on the *lower part* of leg C1 of the offshore jacket with and without dynamic inertia effects (the exact magnitude of axial forces are omitted to protect the interests of the relevant parties)

Figure 2.24 compares the calculated axial force time history at a leg C1 with and without accounting for the dynamic inertia effects. When the dynamic effects are ignored (right figure), the axial forces history entirely follows the variation of the wave and has a period of wave loading (15.6 s) well above the structure's resonance period (2.5 s). However, when the dynamic effects are accounted for, fluctuations (left figure) of the axial force can be clearly observed as a background noise with the resonance period of the structure (2.5 s). Depending on the magnitude of this background noise, it may influence the integrity of the structure with regard to fatigue damage.

From another angle, the dynamic loading often has a different orientation than the static one. For example, the static loading of a structure under the gravity of the Earth is strictly toward the Earth. However, when the structure is subjected to dynamic loading due to, for example, wind, earthquake or sea waves, the direction of resultant loadings change from downward to the one that is more toward a horizontal orientation, this can result in an entirely different pattern regarding the load level and load path, and this obviously influences the structural design. Therefore, structural engineers are required to have a complete picture of load path and level, and structures designed must have corresponding load resisting systems that form a continuous load path between different parts of the structures and the foundation. The structure shown in Fig. 2.23 represents a typical configuration of the jacket structure and a clear path for load transferring, i.e., the gravity and acceleration loads from topside, the wave load applied on the upper part of the jacket, and the jacket gravity and acceleration loads are all transferred through legs and braces down to the pile foundation at the bottom.

Before concluding this section, it is of great importance to emphasize that dynamics is a rather more complex process than its static counterpart. The natural frequency of a structure can change when a change in its stiffness, mass or damping occurs. What makes dynamics even more complicated is that, strictly speaking, regular harmonic loadings and responses, with a sine or cosine form at a single frequency, do not represent environmental loadings (such as earthquake loadings) and the associated responses in the real world, even if they can be a good simplification when the dynamics at a single frequency is dominating. This implies that one should always assess whether the vibrations in various frequencies need to be accounted for or not.

2.2.2 Characteristics of Dynamic Responses

If we take an SDOF spring-mass-damper system under forced excitation as an example, which is illustrated in Fig. 2.25, the equation of motions for the system is expressed as:

$$m\ddot{x}(t) + c\dot{x}(t) + kx(t) = F(t) \tag{2.1}$$

By exerting an external harmonic force ($F(t) = F_0 \sin(\Omega t)$) with an amplitude of F_0 and an angular frequency of Ω shown in Fig. 2.25, or displacement excitations in a harmonic form on the spring-mass-damper system, an SDOF spring-mass-damper system under forced harmonic excitation is constructed. The governing linear differential equation of motions for this system in case of harmonic force excitations can then be written as:

$$m\ddot{x}(t) + c\dot{x}(t) + kx(t) = F_0 \sin(\Omega t) \tag{2.2}$$

Dividing both sides of the equation above by m, this equation is rewritten as:

$$\ddot{x}(t) + \frac{c}{m}\dot{x}(t) + \omega_n^2 x(t) = (F_0/m)\sin(\Omega t) \tag{2.3}$$

It is noted that the viscous damping is very important in an oscillating system because it helps to efficiently limit the excursion of the system in a resonance situation. As a reference, we first define the critical damping c_c, which is the lowest damping value that gives no oscillation responses, i.e., the system does not vibrate at all and decays to the equilibrium position within the shortest time. This represents the dividing line between oscillatory and non-oscillatory motions:

$$c_c = 2\sqrt{km} = 2m\omega_n \tag{2.4}$$

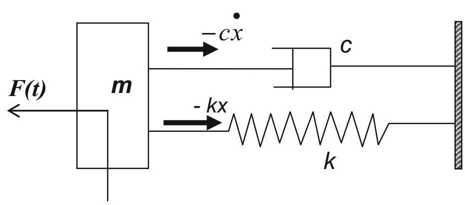


Fig. 2.25 An SDOF spring-mass-damper system under an external force $F(t)$

The actual damping ratio can be specified as a percentage of critical damping:

$$\zeta = \frac{c}{c_c} \tag{2.5}$$

By realizing that $c = 2\omega_n m\zeta$, the equation of motions for the system finally gives:

$$\ddot{x}(t) + 2\omega_n\zeta\dot{x}(t) + \omega_n^2 x(t) = (F_0/m)\sin(\Omega t) \tag{2.6}$$

As the equation above is a second-order non-homogeneous equation, the general solution for it is the sum of the two parts: the complementary solution $x_c(t)$ to the homogeneous (free vibrations) equation and the particular solution $x_p(t)$ to the non-homogeneous equation:

$$x(t) = x_c(t) + x_p(t) \tag{2.7}$$

The complementary solution exhibits transient vibrations at the system's natural frequency and only depends on the initial condition and the system's natural frequency, i.e., it represents free vibrations and does not contain any enforced responses:

$$x_c(t) = Xe^{-\zeta\omega_n t}\sin\left(\sqrt{1-\zeta^2}\omega_n t + \phi\right) \tag{2.8}$$

It is noticed that this aspect of the vibration dies out due to the presence of damping, leaving only the particular solution exhibiting steady-state harmonic oscillation at excitation frequency Ω. This particular solution is also called the steady-state solution that depends on the excitation amplitude F_0, the excitation frequency Ω as well as the natural frequency of the system, and it persists motions for ever:

$$x_p(t) = E\sin(\Omega t) + F\cos(\Omega t) \tag{2.9}$$

By substituting the equation above and its first and second derivatives into Eq. (2.6), one obtains the coefficients E and F as:

$$E = \frac{F_0}{k}\frac{1-(\Omega/\omega_n)^2}{\left[1-(\Omega/\omega_n)^2\right]^2 + [2\zeta(\Omega/\omega_n)]^2} \tag{2.10}$$

$$F = \frac{F_0}{k}\frac{-2\zeta\Omega/\omega_n}{\left[1-(\Omega/\omega_n)^2\right]^2 + [2\zeta(\Omega/\omega_n)]^2} \tag{2.11}$$

By inserting the expression for coefficient E and F into Eq. (2.9) and rearranging it, one can rewrite the steady-state solution as:

$$x_p(t) = \frac{F_0}{km} \frac{\sin(\Omega t - \varphi)}{\sqrt{\left[1 - (\Omega/\omega_n)^2\right]^2 + [2\zeta(\Omega/\omega_n)]^2}} \tag{2.12}$$

where φ is the phase between the external input force and the response output, with the most noticeable feature being a shift (particularly for underdamped systems) at resonance. It can be calculated as:

$$\varphi = \tan^{-1}\left(\frac{2\zeta(\Omega/\omega_n)}{1 - (\Omega/\omega_n)^2}\right) \tag{2.13}$$

It is clearly shown that the steady-state solutions are mainly associated with the excitation force and the natural frequency. Figure 2.26 shows an example of the dynamic responses due to the contribution from both transient and steady-state responses, with a Ω/ω_n ratio of 0.8, a damping value of 0.05 (φ = 0.21), and ϕ = 0.1. Phases between the two types of response can be clearly observed.

When mass m in Fig. 2.25 is subjected to harmonic excitations, the magnitude and phase of the displacement responses strongly depend on the frequency of the excitations, resulting in three types of steady-state responses, namely quasi-static, resonance, and inertia dominant responses, which are illustrated in Fig. 2.27.

When the frequencies of excitations Ω are well below the natural frequencies of the structure ω_n, both the inertia and damping term are small, and the responses are controlled by the stiffness. The displacement of the mass follows the time varying force almost instantaneously. Subject to environmental loading such as wind or ocean wave loading, the majority of land-based structures and fixed offshore structures are designed to reach this condition, as shown in Fig. 2.28. However, earthquake loading is likely to have a dominant frequency higher than the natural frequency of structures.

When the excitation frequencies are close to the natural frequency of the system, the inertia term becomes larger. More importantly, the external forces are almost overcome (controlled) by the viscous damping forces. Resonance then occurs by

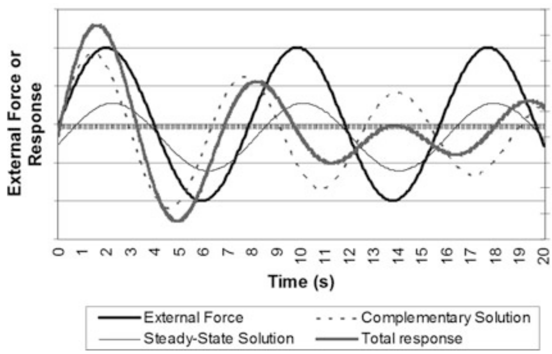
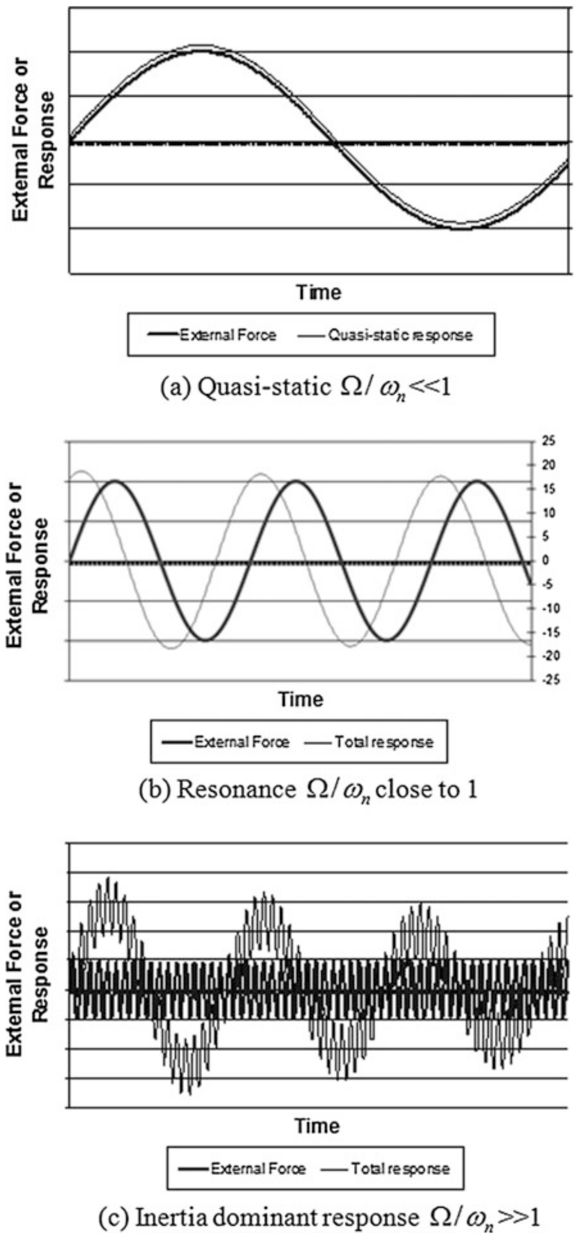


Fig. 2.26 Transient and steady-state responses due to external harmonic force excitations applied on a system with ω_n = 1.0, Ω = 0.8, ζ = 0.05, and ϕ = 0.1

Fig. 2.27 Damped responses due to harmonic excitations with the characteristics of **a** quasi-static ($\Omega/\omega_n \ll 1$); **b** resonance (Ω/ω_n close to 1); and **c** inertia dominant responses ($\Omega/\omega_n \gg 1$), for a system with ω_n = 1.0 and viscous damping ratio ζ = 0.03 [11]



producing responses that are much larger than those from quasi-static responses, as shown in the circle in the upper figure of Fig. 2.28, and there is a dramatic change of phase angle, i.e., by neglecting the damping, the displacement is 90° out of phase with the force, while the velocity is in phase with the excitation force. In a typical

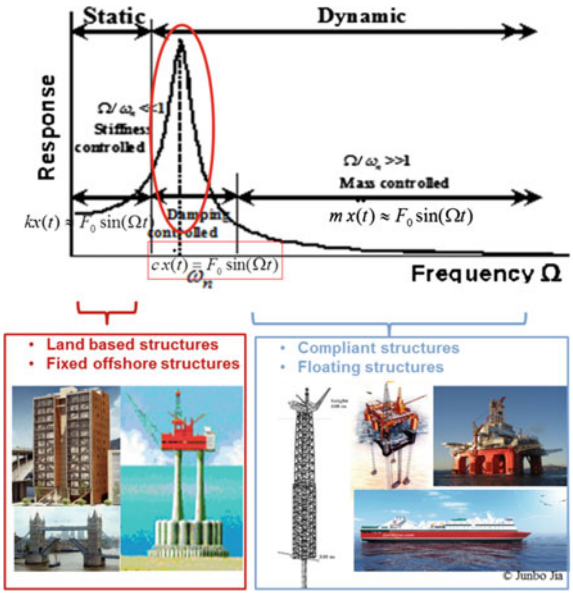


Fig. 2.28 Response of various types of offshore and land-based structures in three frequency ranges subjected to external environmental loading with a frequency of Ω, the natural frequency of the structure is denoted as ω_n (from an oral presentation by the author at the 11th International Conference on Recent Advances in Structural Dynamics, Pisa, 2013)

situation in which the damping is well below 1.0, the responses are much larger than their quasi-static counterparts. From an energy point of view, when the frequency of excitations is equal to the natural frequency, the maximum kinetic energy is equal to the maximum potential energy. Almost all engineering structures are designed to avoid this resonance condition. Possible scenarios of resonance conditions are: when the resonance period (site period) of soil layers at sites due to shear wave transmission is close to the natural period of the structure; when the resonance period of the surface wave is close to the natural period of the structure; or when significant plasticity develops on structural members during a strong earthquake, leading to a decreased natural frequency of the structure, which may track the decreasing predominant frequency of the ground motions, causing resonance with ground motions (moving resonance), etc. These possible resonance conditions will be discussed in Chap. 3.

When the excitation frequencies are well above the natural frequency of the system, the external forces are expected to be almost entirely overcome by the inertia force, the excitations are so frequent that the mass cannot immediately follow the excitations. The transient vibrations are normally more significant than steady-state oscillations. The responses of the mass are therefore small and almost out-of-phase (phase angle approaches 180°) with the excitation forces, as illustrated in Fig. 2.28. From an energy point of view, this reflects the condition in which the maximum kinetic energy is larger than the maximum potential energy [11]. Offshore compliant structures and floating structures are normally designed to behave "softly" in their motion responses, and therefore have natural frequencies of motion ω_n well below the external wave loading frequency. This condition is also a most usual scenario encountered during an earthquake event. It is noted that for most of the engineering structures and typical site conditions, the long period of seismic ground excitations are usually small, except for the ground motions caused by the seismic surface (Rayleigh) waves, which can have dominating long period components of ground motions, which will be presented in Chap. 3. Therefore, subject to seismic ground excitations, a large amount of offshore and land-based structures are likely to reach this condition.

As a structure typically has—or more precisely, has to be represented/modeled by—a large number of degrees-of-freedom, in addition to the natural frequency, which is typically the first eigenfrequency of the structure, it has more numbers/orders of eigenfrequencies, and the total number of eigenfrequencies is equal to the number of degrees-of-freedom of the structure. However, the first few eigenfrequencies, especially the natural frequency, normally dominate the majority of the total modal mass participating in structural vibrations, and are therefore the most important ones contributing to the dynamic response of the structure [11]. From a modal response point of view, the lower order eigenfrequencies of the structure are normally separated well apart. In this frequency range, with small damping, the modal response will generally be dominated by a single mode with frequency close to the loading frequency and a single mode with natural frequency of the structure. And if the loading frequency is lower than the 1st eigenfrequency of the structure, then the structural response will show two peaks at the loading frequency corresponding to quasi-static response and the natural frequency of the structure contributing to dynamic response of the structure. As an example, Fig. 2.29 shows ocean wave induced frequency responses of a welded joint on an offshore jacket structure (Fig. 2.30) in the North Sea. Generally, two peaks in this frequency response graph can be identified: one corresponds to the wave modal frequency (0.08 Hz); the other corresponds to the structure's natural frequency (0.24 Hz). On the other hand, the higher order eigenfrequencies are more closely spaced and modal mass participation of each mode vibration is much less than that of the first few eigen-modes. This is more obvious for highly redundant structures. With a dynamic loading in this frequency region, multiple eigen-modes contribute in a similar extent to the modal response, and vibration modes above the loading frequency will be out-of-phase with those below the loading frequency, the net vibration is likely to be less than any of the single mode vibrations in this frequency range (dynamic cancelation).

2.2.3 Frequency Range of Dynamic Loading

If relevant, subjected to environmental loading, such as wind, earthquakes, ocean waves, current, and ice, etc., all engineering structures should be designed by accounting for their dynamics with a special consideration on resonance, which can be relevant to structural performances associated with ultimate strength, fatigue strength and serviceability limits. Each type of loading has different dominant

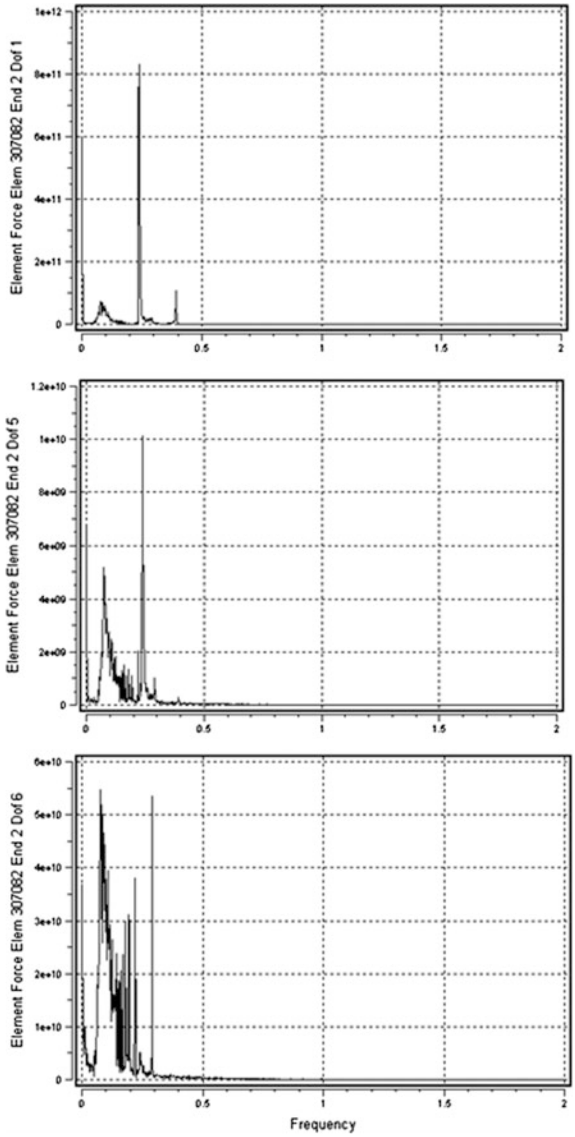


Fig. 2.29 Frequency responses of the axial force (Dof 1) [N], in-plane bending moments (Dof 5) [Nm] and out-of-plane bending moments (Dof 6) [Nm] for a weld joint 10803 (Fig. 2.30) at the top of a jacket, and the jacket is subject to wave loading corresponding to a sea state with a significant wave height H_s = 8.8 m, and a modal wave period T_p = 13.2 s (0.08 Hz) [17]

ranges of loading frequency as shown in Fig. 2.31. In addition, other types of dynamic loading induced by explosion, machinery vibrations, vehicle- or human induced excitations may also require a dedicated consideration on solving relevant dynamic problems in the design.

It is noted that the dominant frequency of seismic ground motions is not only dependent on the frequency of seismic waves generated at the source due to fault

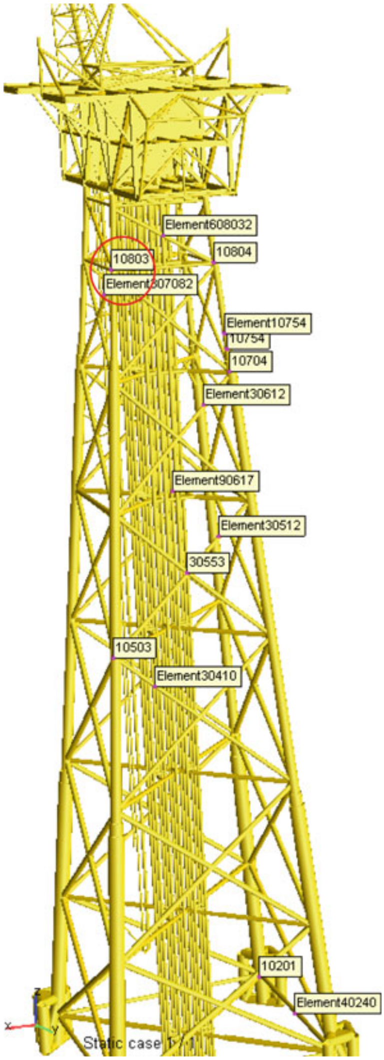


Fig. 2.30 The location of the welded joint 10803 on the jacket

fractures, but also even more influenced by site conditions associated with soil layers and ground topology. Therefore, their peak period has a large range of up to 4 s.

Furthermore, Fig. 2.31 also indicates that the difference of dominant frequency range for different types of environmental loading is significant. This can pose a challenge in designing an optimized structure to resist the different types of dynamic loading. For example, for fixed offshore structure or land-based structures, to avoid significant dynamic amplification and/or excessive vibrations, it is usually desirable to design a stiff structure to resist wind and ocean wave loading, so that the natural period of the structure is far below the dominant period range of wind and wave loading, thus avoiding resonance and limiting excessive vibrations. On

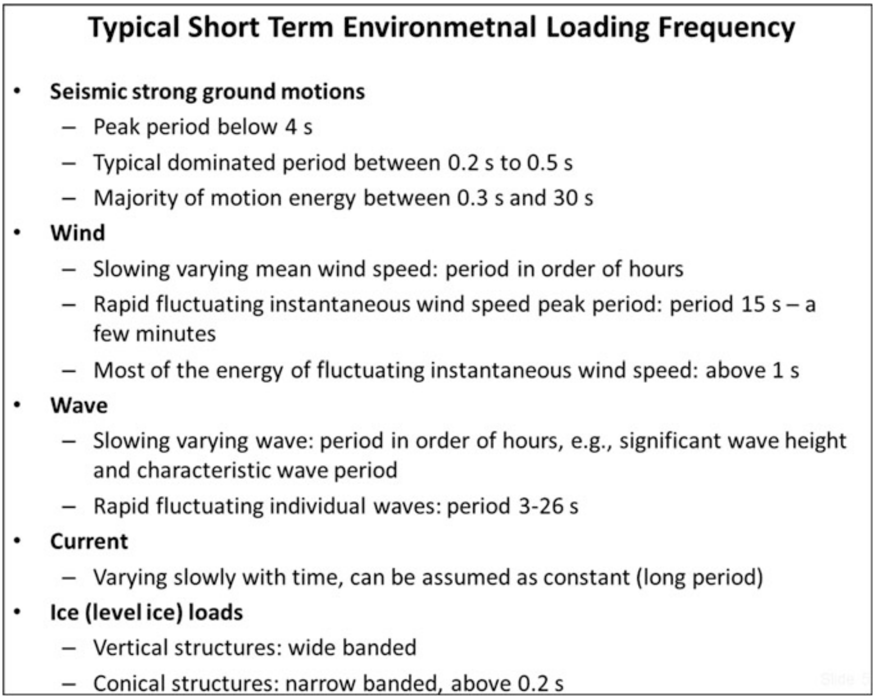


Fig. 2.31 Typical short term environmental loading frequency/period (from an oral presentation by the author at the 11th International Conference on Recent Advances in Structural Dynamics, Pisa, 2013)

the other hand, the low dominant period of seismic loading requires a “softer” structure design that cherishes a higher natural period. This contradiction has been encountered in various structural design projects. Sometimes a “balance” between the two needs to be sought, such as the design of Taipei 101. It has a natural period of around 7 s, which is obviously above the dominant period of earthquake loading. Even though this natural period of 7 s is also far below the period of loading due to wind turbulence (fluctuating part of wind), it can induce significant peak acceleration at the top of Taipei 101, causing both human discomfort and structural metal fatigue. To solve this problem, a large tuned mass damper (TMD) weighing 660 t (Fig. 2.32), as will be discussed in Chap. 24, was introduced to mitigate sway motion of the building, particularly in major typhoons or earthquakes where movement of the top floor can exceed 1.5 m. The TMD will reduce peak acceleration of the top occupied floor from 7.9 to 5.0 mg due to wind storm with a return period of half year. For a 1000–2500 year return period of strong earthquake, the TMD will be rather effective to mitigate the dynamic response of the structure, and

Fig. 2.32 A tuned mass damper (*upper*) suspended from the 92nd to the 87th floor at Taipei 101 (*lower*) (under licenses of CC BY-SA 3.0 by Guillom and Peellden)

to remain in place and intact after strong seismic ground motions cease and the vibration of the structure terminates. In addition, another two small TMDs are designed to mitigate vibration at two tip vibration modes at periods of around 1 s.

Slender light weight structures such as guyed steel stacks, chimneys, slender tips of flare booms or other elevated structures, with two examples shown in Figs. 2.32 and 2.33, the structural design is governed by the wind loading rather than the seismic loading because the structure has a high natural period (compared with the dominant period of earthquake loading) and the wind loading increases with the height from the ground surface. However, due to the tips of those slender structures normally being much softer (with much lower stiffness) than the structural parts below the tips, during earthquakes, they can exhibit significant vibrations, which is referred to as a whipping effect, as will be discussed in Sect. 15.7. Therefore, the design of the slender tips of structures may be governed by seismic loading and therefore requires a dedicated consideration of their seismic resistance.

Fig. 2.33 A flare boom with a slender tip (courtesy of Aker Solutions)

2.3 Difference Between Offshore and Land-Based Structures

The major difference between offshore structures and their land-based counterparts is reflected in the aspects regarding cost and consequence, possibility of evacuation, the availability of seismic ground motion records, structural dynamic behavior and geometry characteristics, structural and hydrodynamic damping, other accompanied environmental and operational loads, as well as special geotechnical issues such as site conditions and sudden subsidence. Therefore, the assessment and design experience adopted for land-based structures must be borrowed with care before applying them for offshore structures.

Compared to that of an infrastructure on land, evacuation during a strong earthquake at a site offshore is almost impossible and can have very serious consequences. This also leads to a higher safety requirement for structural and non-structural elements for offshore structures than structures onshore.

The cost and consequence of structural failure or collapse of offshore structures are normally much more significant than a typical land-based structure such as a building. Therefore, seismic performance assessment for offshore structures is generally associated with a higher required reliability and assessment accuracy. This may limit the applicability of some simplified analysis methods and simplification in the structural modeling that are typically adopted for structural analysis for typical land-based structures.

Offshore structures generally have a much wider range of natural period (1–120 s, depending on the types) than typical onshore building structures (below 9 s). This may sometimes significantly alter the dynamic response subject to earthquake loading and other types of environmental loading.

Due to the significant environmental loading mainly induced by ocean waves, offshore structures are generally much larger in plan than most buildings but do not have a common foundation form to resist the overturning moment generated by the wave and other environmental loads. Hence, subjected to seismic or ocean wave loading, offshore structures are more likely to exhibit a combined torsion and bending (translation) response [12].

An offshore platform generally has a heavy topside at the top of the supporting structure as shown in Fig. 2.9. For example, for a typical jacket (Fig. 2.8) or a jackup (Fig. 2.34) platform, the weight of the topside is normally up to a few times higher than the weight of the supporting structure. Elevation control (as will be discussed in Sect. 19.3) is applied for onshore buildings or tower (Fig. 2.35) structures to avoid a global resonance associated with one or two vibration modes due to earthquake and other dynamic loadings. This requires that a structure should generally be designed with a pyramid shape (with a gradually decreased stiffness and mass per unit height with the increase in its height) to prevent the resonance amplification. This is difficult to achieve in offshore structural design, because a significant mass (in many cases the major mass) is located at the top of the supporting structure. This makes the modal mass participation corresponding to the most important global bending or torsional vibration mode rather high, so that when the resonance occurs due to the vibration participation of the topside, a significant inertia force will be generated and applied on the supporting structure, leading to a large vibration response.

The presence of heavy offshore foundations together with the weight of the platform may change the soil properties, which influences both the foundation impedance (stiffness and damping) and the foundation capacity control.

Compared to that of onshore structures, the space in an offshore platform is generally limited, making it more difficult to place dampers and other mitigation equipment to attain an effective reduction in dynamic response.

Fig. 2.34 Two jackups with supporting structures made of tubular legs (*left*) or trusses (*right*). The jackups are towed to the site and legs are jacked down, engaging the seafloor raising the platforms (courtesy of Dong Energy)

Fig. 2.35 The 320 m high Eiffel tower with an elevation control design, comprising more than 15,000 wrought-iron structural members joined by 2.5 million rivets. The structural weight is 7300 t (photo courtesy of Jing Dong)

The presence of sea water induces additional loads due to added mass, hard marine growth and hydrodynamic damping. Marine growth also adds weight on supporting structures. Therefore, in addition to inertia forces due to ground accelerations transferred to structures, for offshore structures, the relative motions between the submerged structural members and their surrounding fluids also create hydrodynamic damping forces. Furthermore, the surrounding fluids will also enhance the inertia effects of the submerged structural members, which are referred to as the effects of added mass. Both the added mass and the hard marine growth can introduce significant inertia effect to the structure, leading to an increase in natural periods.

The hydrodynamic damping induced by fluid–structure interaction will generally slightly decrease the dynamic response, even though this effect is rather limited and can normally be neglected in mild sea states. When a strong earthquake occurs together with a significant storm (i.e., large wave height), the hydrodynamic damping forces applied on offshore structures can be dramatically increased. Note that the joint probability of occurrence of both events (significant earthquakes and extreme storm waves) is practically extremely low, and is therefore not considered by typical offshore structural design codes.

Due to the abrasion of soil surface by the passing of current, wave and flood, the shear stress generated from the flowing water may exceed the threshold value of the soil erosion resistance, removing the sediment such as sand and rocks from around the foundation (such as piles, bridge abutments or piers). A hole is then formed at the upper soil surface, which is called scour as shown in Fig. 2.36. Scour leads to a reduction in capacity for both the upper structure and the foundation. It particularly reduces the stability of the foundation and increases the maximum design moments in the pile, which requires a larger pile penetration depth and pile cross section area.

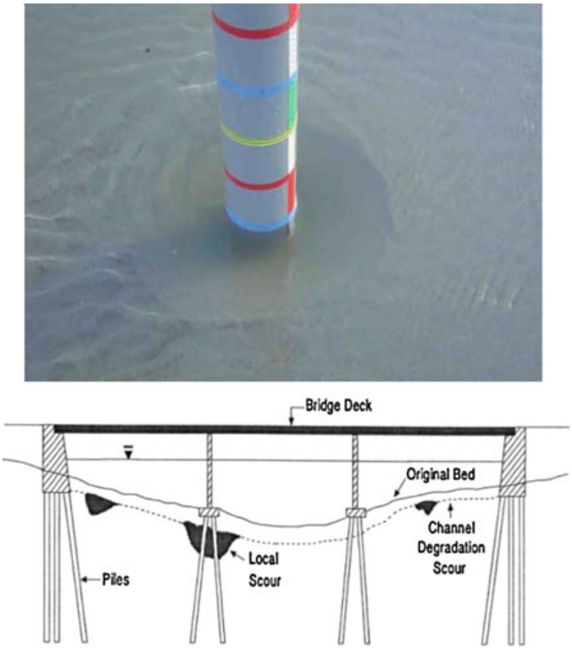


Fig. 2.36 Steep-sided local scour pits around a single pile (*upper figure*), and an illustration of scour at the bottom of a bridge foundation (*lower figure*)

For a small diameter subjected to monotonic horizontal load, the maximum pile bending stress increases almost linearly with the scour depth [13]. Due to the degrading of the foundation stiffness, the presence of scour hole also decreases natural frequency of the structure system. In case the change in natural frequency is significant, it can dramatically alter (decrease or increase) the seismic spectral acceleration value corresponding to the natural frequency, thus significantly changing the seismic force and responses in the foundation and superstructure. In addition, geometrical variation of the mudline leads to more complicated design requirements for the pipeline and cables at seabed.

In addition, due to the effects of water column, depending on the water depth, vertical ground motion acceleration at seabed will be decreased by the effects of finite water column at some frequency ranges while being amplified at other frequency ranges. However, in general, the peak vertical acceleration at seafloor can be reduced by as much as 50 % [14, 15]

The variation of fluid tank levels on offshore structures also changes the mass of the structure. This alternates the dynamic response, and presents potential challenges to perform vibration-based structural health monitoring [11], which is to detect structural damages through the observation of changes in measured eigenfrequencies and corresponding vibration mode shapes.

Compared to land-based structures, offshore structures are generally designed to resist more significant lateral environmental loads due to the presence of wave, current and/or ice crushing loading. Therefore, even though the action points of those loadings on structures may be far from that of the earthquake induced ground excitations, the seismic performance of offshore structures are normally better than their land-based counterparts.

Finally, but not the least consideration, as a consequence of a special type of earthquake, the sudden subsidence of offshore platforms, as will be elaborated in Chap. 16, has been realized to be a serious risk by more and more energy companies as well as authority bodies, which needs consideration during the design.

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http://www.springer.com/978-3-642-31853-5

Modern Earthquake Engineering
Offshore and Land-based Structures
Jia, J.
2017, XXV, 848 p. 559 illus., 12 illus. in color.,
Hardcover
ISBN: 978-3-642-31853-5