

# DESIGN OF OFFSHORE CONCRETE GRAVITY PLATFORMS

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## Abstract

*This technical note reviews the design parameters employed in the design of offshore concrete gravity platforms for the storage of crude oil and gas in the oil and gas industry. Manufacturing and construction methods are discussed. Current trends in construction are also mentioned. The paper carefully illustrates how the principal Environmental loads (wind and wave), current forces, loads from ice and loads from earth-quake for (earth-quake prone zones) are deployed to archive the design and construction of offshore concrete gravity platforms. Two design methods (Analysis and Design of Shell structures) and the Tangent Modulus Methods of design of Offshore Concrete Gravity platforms are discussed. Finally, foundation design of Offshore Concrete Gravity Platforms, the advantages of concrete offshore structures over steel platform are discussed The paper is concluded with an example problem by the author to demonstrate the response of concrete gravity platforms to wave loading.*

**Keywords:** concrete gravity platform, offshore, foundation design, manufacturing, construction

## Introduction

Offshore platforms are mainly related to the provision of services for the oil and gas industry, the sulphur industry utilizes them also. The discovery, in 1969 of the Ekofisk oil field in the North Sea, signaled the beginning of the development of concrete offshore oil platforms. The first such gravity platform is the Ekofisk oil storage tank. Off shore platforms are used to support production drilling equipment and processing equipment, including compressors, storage, and pumping equipment. Many of the dead loads imposed, there-

fore involve vibratory and dynamic loading. Mass is therefore, a desirable feature in the deck structure.

## Manufacture and Construction

Almost all structures designed for submerged or floating service have a common characteristic. They are extremely large and massive. This poses a serious problem for manufacture and construction. As result constructors have evolved some extremely ingenious methods for manufacturing, launching,

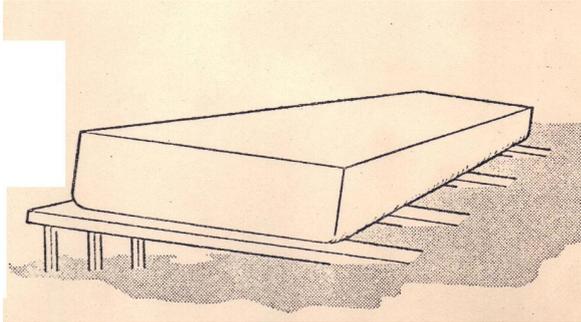


Figure 1: Launching.

assembly and final installation. A brief review of a number of these methods is as follows:

- (a) Construction of the base in a dewatered basin. Many of the largest concrete structures have been cast in a basin, which is later flooded and the structure floated out. In some areas, basins may be excavated, kept dewatered during construction, then flooded and an access channel dug to the waterway.
- (b) Submergence to a minimum freeboard in order to mount the deck which is floated in over the top. Many methods have been devised to ease submergence.
  - (i) *Launching*: Many concrete barges and caissons have been constructed on launching ways, and slid down the ways to flotation in the water. One method is to employ a launching cradle, which rides down the inclined slope, while keeping the barge or caisson level. Another method is to build the barge or caisson on the level at the head of the slope and then rotate it, by jacking beams, to the inclination of the ways. (See fig. 1).

*Tidal launching*: A large number of concrete structures have been launched by making use of the tidal rise, either alone or in conjunction with other methods.

- (ii) *Successive basins*: A system used suc-

cessfully for a number of very large concrete caissons has been as follows:

- (a) The first lift, including bottom plate, is constructed in a tidal zone, so that it may float of as a unit at high tide. Then it is moved to a prepared under-water basin, where it is sunk onto a level bed of gravel or sand at a depth just sufficient to expose the top of the walls at low tide.
- (b) A second lift is poured, and, if necessary, the unit moved again. Thus progressively the unit is floated at high tide, sunk in a new basin at low tide, and then its walls are constructed to a higher stage (See fig 2).
- (c) Ballasting down to the sea level.
- (d) Penetration and final founding by over-ballasting, and placement of scour protection.

### Current Construction Trends

- (1) Pre-casting of large segments of base and roof structures, these are tied by stirrups, transitions or other means to the cast in place concrete so as to perform compositely.
- (2) Prefabrication of piping and mechanical systems in modules or packages that can be set into the structure by cranes or derricks during the construction operations.
- (3) Integration of various conduits for oil, ballast water, and ventilating air into the concrete structure as formed ducts.
- (4) In softer and cohesion less soils, sinking of the structure into the sea floor by a combination of over-ballasting, jetting and air-lifting.
- (5) Use of high strength concrete to reduce weight.
- (6) Use of lightweight concrete for certain upper portions of the structure, e.g. roofs

and shafts. This should be carefully selected for high strength.

- (7) Use of internal pressurization to reduce excessive hydrostatic pressure during deep immersion for deck mounting and installation at the site.
- (8) Use of temporary additional buoyancy tanks, either steel or concrete to maintain stability during the critical process of submergence across the top of the base.

## Design Considerations

### *Environmental loads*

Waves and winds are the two principal environmental loads considered in the design of offshore platforms. The design environmental loads are normally based on environmental conditions which have a recurrence period of 100 years, and the design life time of offshore structures is about 20 - 30 years. A semi-empirical approach is used to evaluate wave loads on offshore structures. The derivatives of a theoretically derived flow potential function are combined with empirical drag and inertia coefficients to predict wave forces on structural components relative to the position of the wave. There are two methods of evaluating the wave loads on a fixed offshore structure. (a) design wave method (b) spectral analysis method.

### *Wave loads by design wave method*

This is a static method. The load value is derived from the passage of a single regular wave of given height and length past the structure. The most commonly used wave period for the North Sea is 15 secs, and the period which causes the worst loading is used. Currently a 20 secs limit on wave period is recommended.

### *Wave loads by wave response spectrum method*

Spectral analysis has frequently been used to predict maximum wave forces responses as an alternate to the deterministic design wave method. This method is very suitable for gravity structures where the initial wave forces dominate. Spectral methods are useful where nonlinear, drag loads are small compared to the linear inertial loads. Therefore, a linear relationship exists between wave heights and wave force for a given period.

### *Wind loads*

The force exerted by wind on an offshore structure is a function of three basic parameters the wind velocity, the orientation of the structure, and the aerodynamic characteristics of the structure and its members. The wind force exerted on a structure consists of two components, one parallel to the direction of travel of the wind and the other perpendicular to the direction of travel of the wind.

$$F_D = C_d^{\frac{1}{2}} \rho V_Z^2 A \longrightarrow (\text{force parallel to wind})$$

This is also called Drag force and this tends to tilt the structure and is responsible for a large percentage of the overturning moment.

$$F_L = C_L^{\frac{1}{2}} \rho V_Z^2 A \longrightarrow (\text{force perpendicular to wind, tends to lift the structure})$$

$C_d$  = drag coefficient

$C_L$  = lift coefficient

$\rho$  = density of the air

$V_Z$  = wind velocity

$A$  = Area perpendicular to wind velocity.

The wind velocity is not constant because of the shear forces with the earth's surface; it is zero at the surface and increases exponentially to a limiting maximum speed known as the gradient wind. Over water the wind speed at any elevation is represented by the one-seventh power law.

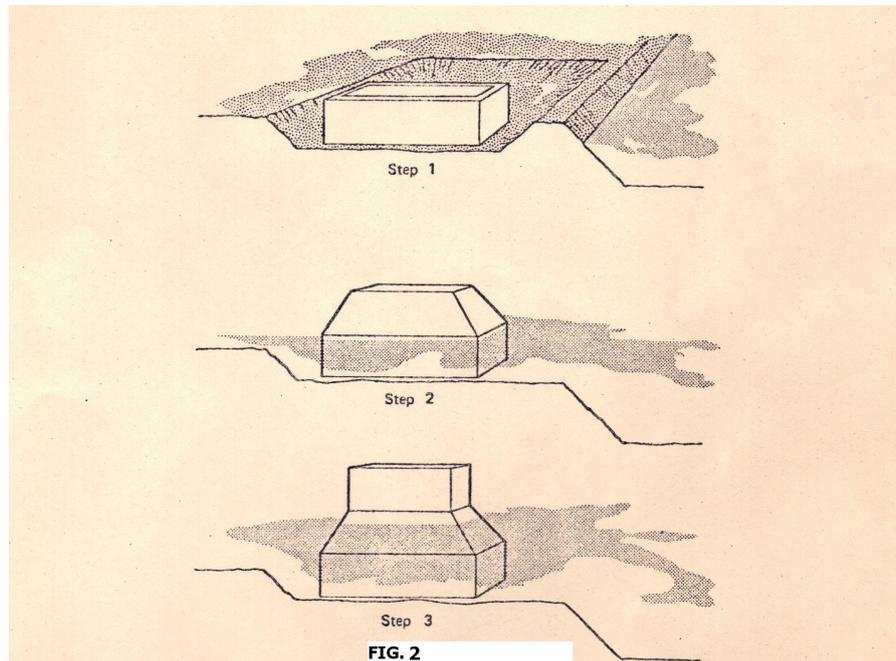


Figure 2: Successive basins.

$$V_Z = V_{ref} \left( \frac{Z}{Ref} \right)^{1/7}$$

$V_{ref}$  = the wind speed at a height of 30 ft (Usually the customary elevation for such measurements)

$Z$  = desired elevation in ft.

$Ref$  = the reference height (30ft).

The wind effects on all parts of the above-water structure should be calculated. The two kinds of wind speeds are normally considered.

- (a) Sustained wind speed, which is defined as the average wind speed during the time interval of one minute;
- (b) Gust wind speed, which is defined as the average wind speed during a time interval of 3 sec. The gust factor is that multiplier which must be used on the sustained wind speed to obtain the gust speed, or the fastest - mile velocity. The average gust factor, F10, at the 30ft elevation is in the range 1.35 - 1.45.

Another commonly accepted formula for calculating the wind force is

$$F = KV^2C_sA$$

Where,

$F$  = wind force

$K$  = constant

$V$  = design wind speed

$C_s$  = shape coefficient

$A$  = projected area

$K = 0.00256$  (lbf, mile/hr, ft<sup>2</sup>)

Typical shape coefficients, for all angles of wind approach could be

Beams  $C_s = 1.5$

Slides of Buildings  $C_s = 1.5$

Cylindrical Sections  $C_s = 1.0$

### **Shielding coefficients**

Shielding coefficients may be used with care when, in the judgment of the designer, an object lies close enough behind another to warrant the use of such coefficients.

### ***Current forces***

Two major components of the current are considered: tidal current and wind driven current. It is generally accepted that the wind driven current at the still water surface may be taken as 1% of the sustained wind speed at 30 ft (10m) above the still water level. The current velocity should be added vectorially to the wave particle velocity before computing the drag force. Because drag depends on the square of the horizontal particle velocity, and because the current decreases with slowly with depth, a comparatively small current can increase drag significantly. In design, the maximum wave height is sometimes increased by 3 - 4% to account for the current effects and the current per se is neglected.

### ***Loads from ice***

In areas prone to snow, the structure should be designed to resist all static and impact forces from ice. In all locations where the hazard of floating ice may exist, the thickness and strength of the ice, and the velocity with which the ice may strike the structure should be determined. With pack or sheet ice, the mode of failure of the ice should be determined. In some locations, ice may accumulate on the structure and superstructure. Allowance should be made for the effect of this accumulation upon the stability of, and upon the stresses in the structure. The additional resistance to wind offered by such ice accumulation should be included in this allowance.

### ***Loads from earthquakes***

Where the site is in an area with a history of recorded earthquakes, the structure should be designed to resist lateral and vertical forces and overturning moments arising from seismic activities. The information used to establish a design earthquake should be taken from data recorded for previous earthquakes

at or near the site, if such data are available. Because there are relatively few recorded stations compared with the number of seismically active regions, the likelihood of selecting a location having a complete historical description of seismic activities is very small. In such cases the given site should be compared with a site whose geology and tectonics are similar and for which records are available.

### **Design Methods**

Design of offshore concrete structures, is based on limit (semi probabilistic methods).

Limit State – A structure or structural member reaches a Limit state of fitness in a condition where it just ceases to fulfill the resistance requirements or other specifications as regards structural performance for which it has been designed. See table 1 for the various limit states used in design.

As a result of the fact that full statistical information is not available, the main uncertainties are included as partial load and material safety factors. Let

$Q_c$  = Characteristic load effects

$R_c$  = Characteristic resistances

These are defined as certain percentiles of the distribution functions for load and resistance.  $Q_c$  is the mean of the characteristic load effects distribution plus one or two standard deviations.

Parameter  $R_c$  is the mean of the characteristic resistance distribution minus one or two standard deviations. The design load effect,  $Q_d$ , is the most unfavorable combination of a specified set of loads and associated partial load factors,  $h_f$  i.e.

$$Q_d = \text{effect of } \sum h_f Q_c$$

The design resistance,  $R_d$ , is the most unfavorable combination of relevant characteristic

and substitutional resistance parameters,  $R_c$  and associated material factors,  $h_m$  i.e.

$$R_d = \text{Combination of } \sum K \frac{R_c}{h_m}$$

in which  $K$  = constants defining the geometry and composition of member sections.

$h_m$  = Material factors

Safety against any limit state requires that

$$\begin{aligned} R_d &\geq Q_d \\ \sum K \frac{R_c}{h_m} &\geq \sum h_f Q_c \end{aligned}$$

### ***Analysis and design of shell structures***

Concrete gravity platforms consist mainly of various cylindrical shells of large dimensions. These shells are capped with spherical or conical domes at bottom and top. Further, all these platforms are equipped with concrete or steel skirts in the form of short cylindrical shells, or both as a means of scour protection and of assuming sufficient resistance against horizontal sliding.

The complexity of the various shell components connected together coupled with the complex actions of the environmental loads and other load combinations renders the analysis and design of such structures difficult.

It is important that the walls of concrete shell be properly proportioned to prevent catastrophic collapse against various periods of large hydrostatic pressure exposure. Potential failure modes to be considered are material failure and structural stability.

### ***Tangent modulus methods***

Research has shown that the tangent modulus buckling concept is proper for the prediction of the maximum strength of many types of columns i.e. centrally located columns. This concept is not adequate for cylindrical shells, because the tangent modulus approach pertains to idealized perfect structural members while actual structural mem-

bers contain significant geometric imperfections and/or lateral loads. However, tangent modulus approach is a powerful simplified model to an actual structure in that:

- (1) It takes account of nonlinear stress-strain behavior of the material.
  - (2) It uses linear elastic buckling analysis.
  - (3) If in-plane forces are dominating, this approach gives reasonable predictions.
  - (4) The design method proposed by Furnes is briefly described in the following section.
- (1) Obtain the linear elastic buckling stress for an idealized perfect structural member with constant modulus  $E_c$
  - (2) Express critical stress  $\alpha_{cr}$  as

$$\alpha_{cr} = \rho E_c \quad (1)$$

$\rho$  = an ideal buckling number

For the case of a simply supported column it has the value

$$\rho = \frac{\pi^2}{\left(\frac{1}{y}\right)^2} \quad (2)$$

$\frac{1}{r}$  = the slenderness ratio. For the case of a complete cylindrical shell with end enclosures acting as a simple radial supports and subject to hydrostatic pressure, it has the value

$$\rho = \frac{q^4 + s(q^2 + m^2)^2(q^2 + m^2 - 1)^2}{(q^2 + m^2)^2(m^2 - 1 + q^2/2)} \quad (3)$$

$$q = \pi \frac{R}{L}$$

$$S = \frac{t^2}{12R^2(1 - V^2)}$$

$V$  = Poissons ratio

$m$  and  $n$  = buckling waver integer numbers

$t$  = wall thickness

$R$  = mean radius

$L$  = length

For other boundary conditions and loads, the buckling number may be obtained from handbooks, tests or computer analysis.

(3) Take the nonlinear stress-strain behavior for concrete into account by simply replacing the Young Modulus  $E_c$  by the tangent modulus  $E_{ct}$ , which is a function of concrete stress  $\alpha$ . Substitution of  $E_{ct}$  and  $E_c$  in equation above. Equation 1 yields

$$\frac{\sigma_{cr}}{f'_c} = \alpha = \frac{2}{1 + \sqrt{1 + (\epsilon_{ult}/\rho)^2}} \quad (4)$$

For short term loading,  $\epsilon_{ult}$  = strain at  $f'_c$  is normally chosen to be 0.002. For long term load  $\epsilon_{ult}$  may be modified by creep factor  $\mu$

$$\epsilon'_{ult} = \epsilon_{ult}(1 + \mu) \quad (5)$$

To allow for the effect of creep, a range of  $0.5 \leq \mu \leq 1.0$  is found appropriate. The effect of reinforcement may be considered by changing  $\alpha$  to

$$\alpha' = \alpha \left( 1 + \frac{f_s A_s}{f'_c A_c} \right) \quad (6)$$

In which  $\alpha$  is defined in equation 4.  $f_s$  = the steel strength;  $A_s$  and  $A_c$  = the steel and concrete sectional areas, respectively. Magnification factor  $(1 - \sigma/\sigma_{cr})$  may be used to obtain design moment  $M_d$  by simply multiplying this factor to first-order moment  $M_{do}$ . The design moment is obtained, i.e.

$$M_d = \frac{M_{do}}{1 - \frac{1}{\rho t} \frac{\sigma}{\sigma_{cr}}} \quad (7)$$

### ***Special design considerations***

As a result of the fact that offshore concrete gravity structures are usually very huge, their size, coupled with the large environmental forces, present very difficult design problems. The establishment of a realistic serviceability Limit State (SLS) is very important because of the following reasons.

- (a) Oil storage is usually one of the principal functional requirements, structural components containing oil should be designed completely tight with respect to oil leakage, which means that only very limited cracking of the concrete may be allowed. The Author of this paper is an eye witness of the consequences of frequent oil-spillage in Nigerian swamps. when oil containers cracked. Vast expanses of land were usually totally destroyed and such areas are then declared disaster areas.
- (b) The internal pressure of the stored oil should always be kept below the external pressure of the surrounding water, in which case somewhat less severe requirements may be used. Sometimes a separate water-containing chamber is provided between the stored oil chamber and the outside ocean water.
- (c) As a means of checking cracks in the offshore concrete gravity structures, the author suggests the installation of probes or strain gauges that will measure cracks around potential crack regions and constant monitoring of such probes to detect unusual signs of danger.

### ***Other important design problems***

The following additional important design considerations should not be overlooked.

- (a) Cumulative fatigue damage at the column bases.
- (b) Temperature stresses within the caisson caused by storage of hot oil.
- (c) Structural instability of the columns and the inner walls of the caisson.
- (d) Local earth pressures on the base raft.
- (e) Impact load capacity of reinforced concrete.
- (f) Ultimate load carrying capacity in complex regions where structural members of various geometry are connected.

- (g) Potential instability of the structural system during tow-out operation due to top-sided structural weight.

### Foundation Design for Offshore Concrete Platforms

Soil parameters of the soil should be obtained by thorough investigation because the foundation design depends entirely on them. The site investigations should include a survey of sea bottom topography, site geology, geophysical investigations, in-situ determination of soil parameters by means of soundings, vane shear and cone penetration tests, and sampling in boreholes with laboratory investigations, of the samples.

The stability of the foundations is the most important problem often encountered in foundation design. Several possible modes of failure for a typical gravity structure is shown in Fig. 3. The simplest mode of failure is sliding between the base of the structure and the sea floor. This is critical if the shear resistance at the interface is smaller than that in the soil mass. Sliding is checked by adding steel skirts to the bottoms of the concrete cells of the caisson and by adding several steel tubular dowels that extend below the skirts. During de-ballasting for placement the dowels contact the soil first. Both dowels and steel skirts penetrate the soil and preclude sliding. The second mode of failure occurs if the shear strength of the soil is exceeded. This failure mode is typical for foundations of clay. The analysis should consider the possible reduction in shear strength due to repeated loadings. For large foundations on sand the possibility for a bearing capacity failure for transient wave loading depends on the un-drained shear strength of the soil. In this case a bearing capacity failure like that in clay soil is not likely. However, a high stress level may lead

to large deformations with high hydraulic gradients along the rim of the foundation and repeated loading may lead to softening of the soil and rocking failure. Liquefaction is another mode of failure for gravity structures on sandy soil. This mode of failure would most likely occur during a storm wherein repeated shear stress applications lead to a gradual increase in pore water pressure which causes a reduction or possibly a complete loss of shear strength of the sand so that it behaves like a heavy liquid. If there is eccentric loading on the deck, the structure sinks into the fluidized soil, tilting at the same time. To account for such defects in the design, tests on the actual soil have to be carried out.

### *Advantages of concrete offshore structures over steel platforms*

- (1) Concrete structures, not relying on piles require great mass to stay put in the face of sea storms even severe storms. The Ekofisk platform tank weighs 212,000 metric tons exclusive of its extra ballast of lean concrete and oil and/or water that fill its tanks at all times. It must be mentioned that the concrete structures are so massive that they include oil storage tanks at little or no extra cost.
- (2) Because the concrete tanks are so massive, they can carry heavy oil production platforms up top. So if North Sea platform is going to extract oil at a high rate say 300,000 bpd (barrels per day) a concrete structure may be less costly.
- (3) Steel structures offshore tend to be harder to inspect than concrete, steel structures consist of many tubes and joints, many of them at locations which are not easily accessible. The few, large pieces in a concrete structure can be more easily inspected visually from a diving bell or sub.
- (4) Concrete itself performs well under low temperature conditions. It is well known

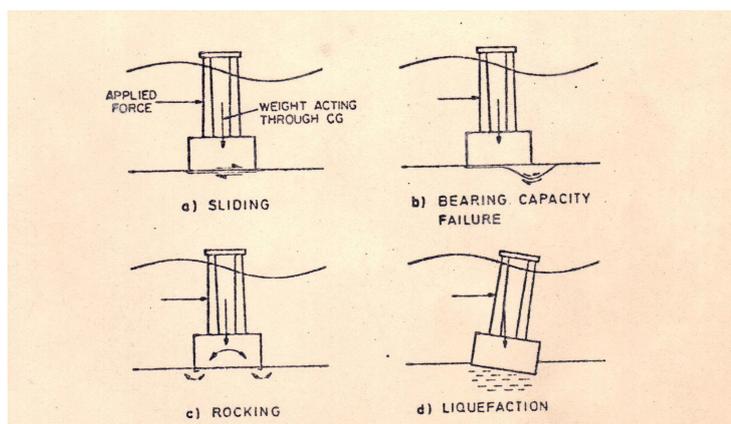


Figure 3: Possible failure modes for foundation of typical gravity platform.

that the strength of concrete increases with lower temperature. This feature gives concrete advantages as a building material for vessels carrying LNG (Liquefied Natural Gas) or working in arctic areas.

- (5) Concrete vessel construction time is 18 months steel vessel construction time is 36 months.
- (6) Concrete vessel costs U.S.\$33 million based on 1975 cost proposals as against steel vessel - \$40 million, based on the same 1975 cost proposals.
- (7) Constructions of concrete vessel in a dry dock makes use of traditional civil engineering materials and less skilled labor, steel vessel require skilled welders and a lot of other costly materials for connections.

### ***Response of concrete gravity platform to wave loading***

An example problem will be used to illustrate how the design engineer can incorporate all the factors so far mentioned in this paper. In this example, the problems arising during the analysis of gravity type structures are investigated. The example, apart

from showing how all the factors mentioned in this paper are usually incorporated in the design, will show how the soil structure interaction is included and secondly will illustrate the use of spectral methods for handling the dynamic analysis solely in the frequency domain that is considered. Let us consider a structure, 50m diameter by 100m high circular tank which is typical of an offshore soil storage tank. As shown in Fig. 4. The structure is to be analyzed completely in the frequency domain using the *spectral method*. For simplicity only two degrees of freedom will be assumed, namely the rocking and translative motion of the *rigid* structure on the *soil* springs.

The equation of motion is

$$M(X) + C(X) + KX = F(t),$$

Where the vector  $C = \{X, 0\}$  is the horizontal translation and angle of rocking of the system. Considering the first terms in the left hand side of the equation, then the forcing terms.

**Mass:** Three component masses must be included, namely, the mass of the structure and contents, the added water mass and the effective mass of the soil participating in the motion. The position of the centre of gravity and the moment of inertia must also be calculated.

The calculations shown for the components parts below, the centre of gravity is given in metres from the mudline (positive upwards) and the moments of inertia are about the individual component mass centres.

- (i) Structural Mass

$$\text{Mass} = 2.4 \times 10^8 \text{Kg}$$

$$\text{Moment of Inertia} = 2.04 \times 10^{11} \text{kgm}^2$$

$$\text{Centre of gravity} = 43.05 \text{m}$$

- (ii) Added Mass

The added mass of the water which becomes entrapped and moves with the structure is calculated

$$\text{Added Mass} = 1.62 \times 10^8 \text{kg}$$

**TABLE 1 —Design Limit States**

Limit states of unfitness (1)	Main characteristics (2)
<p>ULS: ultimate limit state</p>	<p>Ultimate load-carrying capacity Rupture or yielding of sections Collapse or instability of single member or structure Transformation into mechanisms Loss of equilibrium, etc.</p>
<p>PLS: progressive collapse limit state</p>	<p>Accidental loss or overloading of single members that may render the structure or major parts thereof into a condition where progressive failure may take place</p>
<p>FLS: fatigue limit state</p>	<p>Cumulated effects caused by cyclic or repeated stresses during service life Disintegration caused by cumulated fatigue damage Insufficient residual strength</p>
<p>SLS: serviceability limit state</p>	<p>Specifications as regards serviceability or durability Excessive deformations (vibrations) without loss of equilibrium Damage caused by corrosion Aspects of durability in the general sense including unforeseen amount of maintenance and repair</p>

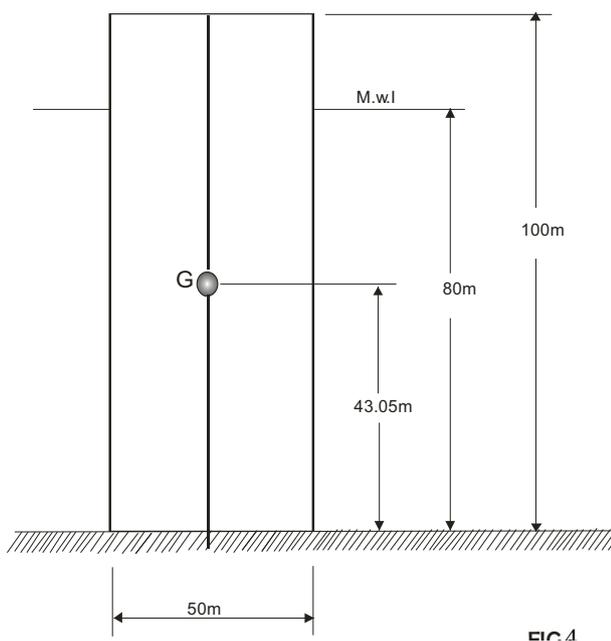


Figure 4: Mass of structure and contents =  $2.4 \times 10^8$  kg; Distance of c.g from base = 43.05m; Moment of inertia of structure and contents about G =  $2.04 \times 10^{11}$  kg m; Water depth = 80m; Density of sea water = 1030kg/m; Density of soil = 2000kg/m; Coefficient of subgrade modulus for soil =  $7 \times 10^7$  N/m

Moment of Inertia =  $8.64 \times 10^{10} \text{kgm}^2$   
 Centre of gravity = 40.00m  
 (iii) Soil Mass  
 Effective Horizontal Mass =  $\rho 0.76R^3 / (2 - \nu)$   
 $R = \frac{BL}{\pi}$  = Radius of foundation  
 $\rho$  = soil density  
 $B$  = width of rectangular foundation  
 $L$  = Length of rectangular foundation  
 Soil Mass =  $1.58 \times 10^7$  Kg.  
 Effective rocking inertia of soil =  $0.64\rho R^5 / (1 - \nu)$   
 =  $2.5 \times 10^{10} \text{Kg m}^2$   
 Centre of gravity = -5.84m  
 (It is assumed that the  $1.58 \times 10^7$  Kg of soil is contained within a hemisphere below the base radius of which is calculated directly as 15.6 m for the specified soil density).

It is interesting to note that the soil mass is less than 1/10th of the structure and added mass, therefore, even if it is neglected entirely, the influence in natural frequency is small.

## (iv) Total Mass

The total mass, centre of gravity and moment of inertia (about the composite c.g.) for the vibrating system are:

$$\text{Mass} = M_{horiz} = 4.18 \times 10^8 \text{ Kg}$$

$$\text{Centre of gravity} = L_g = 40.0\text{m (above mudline)}$$

$$\text{Moment of Inertia} = M_{rock} = 3.51 \times 10^{11} \text{ Kg m}^2$$

**Stiffness:** In this example, the structure is taken as rigid. The only flexibility is the horizontal and rocking resilience of the soil. For modeling the soil stiffness the *lumped* element technique is used. *The subgrade modulus* method is considered more appropriate to this size of structure.

Horizontal Spring Stiffness:

$$K_{horiz} = 0.5C_V A = 6.87 \times 10^{10} \text{ N/m}$$

Rotational Spring Stiffness:

$$K_{rock} = 1.7C_V I = 3.65 \times 10^{13} \text{ Nm/rad}$$

$A$  = area of horizontal contact between foundation and soil.

$I$  = Rocking inertia of foundation

$C_V$  = Subgrade modulus

To ensure a lower bound to the natural frequency for this example, 50% of the horizontal stiffness and 33% of the rocking stiffness is taken in the analysis.

**Damping:** The damping ratio of the soil is calculated from equations. Horizontal Damping ratio =  $0.31B'$

$$B' = \frac{M}{\rho R^3}$$

$M$  = Mass of foundation

$\rho$  = soil density

$R$  = effective radius

$E_{horiz} = 0.086$

Rocking Damping Ratio =  $\xi_{rock}$

$$= 0.05 + \frac{0.1}{B'(1 + B/4)}$$

$$\xi_{rock} = 0.055$$

Arrange the mass and stiffness in the mass and stiffness matrices and the natural frequencies and calculate normal modes. The *origin* is taken at the centre of gravity.

**Mass Matrix:**

$$M = \begin{bmatrix} M_{horiz} & 0 \\ 0 & M_{rock} \end{bmatrix} \\ = \begin{bmatrix} 4.18 \times 10^8 & 0 \\ 0 & 3.51 \times 10^{11} \end{bmatrix}$$

**Stiffness Matrix:**

$$K = \begin{bmatrix} K_{horiz} & -L_g K_{horiz} \\ -L_g H_{horiz} & L_g^2 K_{horiz} + K_{rock} \end{bmatrix} \\ = \begin{bmatrix} 3.44 \times 10^{10} & -1.37 \times 10^{12} \\ -1.37 \times 10^{12} & 6.70 \times 10^{13} \end{bmatrix}$$

**Natural Frequencies:**

The natural frequencies are found from the roots of the frequency determinant.

$$-M_w^2 + K = 0$$

$$\begin{bmatrix} W1^2 \\ W2^2 \end{bmatrix} = \begin{bmatrix} 10.754 \\ 262.156 \end{bmatrix} \text{ rad/sec}^2$$

Or

$$\begin{bmatrix} N_1 \\ N_2 \end{bmatrix} = \begin{bmatrix} 0.52 \\ 2.56 \end{bmatrix} \text{ Hz}$$

**Normal Modes:**

The normal modes are found by substituting the values of  $W1$  and  $W2$  back into the frequency determinant to find relative magnitudes of  $X$  and  $\theta$  (The absolute values cannot yet be found until the *forcing* terms are considered).

$$\text{Mode 1: } X = 1.00, \theta = 2.173x10^{-2}$$

$$\text{Mode 2: } X = 1.00, \theta = -5.472x10^{-2}$$

So that

$$\phi = \begin{bmatrix} 1 & 1 \\ 2.173 \times 10^{-2} & -5.472 \times 10^{-2} \end{bmatrix}$$

Using orthogonality to decouple the equations of motion

$$\begin{aligned} \phi^T M \phi &= \begin{bmatrix} 5.837 \times 10^8 & 0 \\ 0 & 1.47 \times 10^9 \end{bmatrix} \\ &= \begin{bmatrix} M_1 & 0 \\ 0 & M_2 \end{bmatrix} \end{aligned}$$

$$\begin{aligned} \phi^{TK} \phi &= \begin{bmatrix} 6.278 \times 10^9 & 0 \\ 0 & 3.853 \times 10^{11} \end{bmatrix} \\ &= \begin{bmatrix} K_1 & 0 \\ 0 & K_2 \end{bmatrix} \end{aligned}$$

Introducing Damping at this stage,

$$C = \begin{bmatrix} C_{horizant} & -Lg C_{horiz} \\ -Lg C_{horiz} & Lg^2 C_{horiz} + C_{rock} \end{bmatrix}$$

But  $\phi^T C \phi$  would not be diagonal and the equations would not decouple. To overcome this limitation for this example, the damping is included on a modal basis bearing in mind that the first mode is predominantly rocking and the second mode ground translation.

Thus, damping ratio relevant to rocking is associated with the first uncoupled mass term and that for translation with the second mass term. Therefore the decoupled damping matrix is taken as

$$\phi^T C \phi = \begin{bmatrix} C_1 & 0 \\ 0 & C_2 \end{bmatrix}$$

And  $C_n = 2\xi_n M_n W_n$

For n = 2

So that

$$C_{i,2} = i,2 M_{1,2} W_{1,2}$$

$$\begin{aligned} C_1 &= 2 \times (.055) \times (5.837 \times 10^8) \\ &\quad \times \sqrt{10.754} \\ &= 2.106 \times 10^8 \text{Ns/m} \end{aligned}$$

$$\begin{aligned} C_2 &= 2 \times (.086) \times (1.470 \times 10^9) \\ &\quad \times \sqrt{262.156} \\ &= 4.093 \times 10^9 \text{Ns/m} \end{aligned}$$

The modal transfer functions are

$$H_1(jf) = \frac{1}{K_1 - M_1(2\pi f)^2 + jC_1(2\pi f)}$$

$$H_2(jf) = \frac{1}{K_2 - M_2(2\pi f)^2 + jC_2(2\pi f)}$$

Assuming a Spectrum of the JONSWAP form

$$S_{nn}(f) = \frac{\alpha g^2}{(2\pi)^4} \frac{1}{f} 5 \exp(-1.25(\frac{fm}{f})^4) \Upsilon$$

$\alpha$  = Philips empirical constant (=0.00081)

$g$  = Acceleration due to gravity

$fm$  = frequency at which mps. Spectral energy is maximum.

The force may be computed from the water surface elevation as follows. For a large body, such as an offshore concrete platform under consideration, the force is largely dependent on the water particle inertia terms with small contribution from the velocity dependent drag terms. The water particle acceleration at any horizontal co-ordinate,  $x$ , and depth,  $y$ , may be related to water surface elevation by equation below.

$$U(t) = (2\pi f)^2 H \frac{\text{Cosh}[(2\pi t)^2(y + d)]}{\text{Sin} \left[ 2\pi \left( \frac{x}{t} - \frac{t}{T} \right) \right]} \frac{\text{Sinh}[(2\pi f)^2 d]}{g}$$

$H$  = wave height

$L = g/2\pi f^2$  , the wave height

$d$  = water depth

$y$  is positive upwards from the m.w.l.

The force over a length of cylinder  $dy$ , may be related to the water particle accelerations by

$F(t) = \rho\pi R^2 dy C_m U(t) = \text{Acceleration} \times \text{volume}$

$C_m$  = inertia coefficient which for a large body should be calculated by diffraction theory.

## Conclusion

In the early eighties, the ACI Committee 357 on Offshore Structures issued a report on Recommended Practice for Fixed Offshore Concrete Structures. This report was issued in the anticipation of the forthcoming usage of large fixed concrete sea structures all over the world. Apart from going by the design methods presented in this paper, a survival strength analysis is needed in order to prevent the structure from catastrophic collapse in the event of rare natural or man-made events such as ductility level earthquake, adverse ice condition, collisions or explosion.

Finally, all aspects of the installation of the structure, including its immersion and placing on the sea bed, should be planned and carried out with the greatest care. The arrangements made for installation should ensure that the structure is placed in position with the given tolerances. For large structures, lifting and replacing should be considered only as an emergency.

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