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A COURSE IN

COASTAL DEFENSE SYSTEMS I

CHAPTER 10

COASTAL AND OFFSHORE STRUCTURES

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10. COASTAL AND OFFSHORE STRUCTURES

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10.1 Introduction.

The design wave height – which will normally be the significant wave height – and wave period are defined, associated with a given return period, to form the basis for the detailed design of coastal structures. This will be linked to a maximum water level for design based on HAT plus storm surge with a given return period. Coastal structures form two main groups, sea walls and rubble mound structures (although there are combinations of the two, for example a sea wall with a rubble or rock berm or slope at its base).

10.2 Sea Wall Design.

These again fall into two groups, sloping and vertical faced. In both cases the forces imposed by wave action act perpendicular to the slope as pressures or impacts, and tangential to the slope as shear forces. There is no percolation of water into or out of the face of the structure.

10.3 Wave Breaking.

For a sloping structure, as for natural beaches, it is necessary to consider wave breaking and the type of breaker, of which there are three main type: spilling, plunging and surging. Breaking takes place when the velocity of the particle in the wave exceeds the forward velocity of the wave profile. The parameters involved are:

The breaker height, $H_{b,}$
The depth of water at breaking, h_b,
The crest elevation, y_b. above the still water level

(Subscript b denotes breaking)

The **type of breaker** is determined by the Irribaren Number, often termed the "surf similarity parameter":

$$\xi_o = \tan \alpha / (H_o L_o)^{1/2} \qquad 1.$$

where α is the slope of the structure or beach, H_o is the deep water wave height and L_o is the deep water wave length. ($L_o = gT^2/2\pi = 1.56T^2$)

The limits for breaker types are:

$\xi_o < 0.5$ Spilling breakers
$0.5 < \xi_o < 3.3$ Plunging breakers
$\xi_o > 3.3$ Surging breakers

Obviously, sloping and vertical sea walls will reflect wave energy and this reflection, if too high, will influence the stability of the beach in front of the wall. Generally the reflection is less than 10% for spilling breakers but this increases for plunging conditions reaching a maximum of 60% to 80% for surging breakers. On steeper slopes the reflection is higher and for Irribaren numbers less than 2.5 the reflection coefficient, c_r, (reflected wave height divided by the incident wave height) is given approximately by

$c_r = 0.1(\xi_o)^2$.

Generally in coastal engineering wave breaking is determined by the local water depth. For solitary waves the maximum wave height, H_{max} or H_b, in a depth of water, h_b, is given by $H_{max} = 0.78h_b$ although for oscillatory waves $H_b \approx h_b$.

In intermediate depths of water:

$$H_b/d_b = 0.142 \tanh (2\pi d_b/L_b) \qquad 2.$$

In which d_b and L_b are the depth of water and the wave length at breaking, respectively.

10.4 Wave Run-up.

After a wave breaks on a slope the wave energy not dissipated in breaking is expended in the motion of the water up the slope, above the still water level, termed wave up-rush. The <u>vertical</u> distance reached by the water is the wave run-up, R.

For non-breaking, small amplitude waves on a smooth impermeable slope:

$$R_s/H_I = (\pi/2\theta)^{1/2} \qquad 3.$$

where θ is the slope. For a vertical wall $\theta = \pi/2$ and $R_s = H_I$

For finite amplitude waves acting on a vertical wall (non-breaking):

$$R_s = H_I + (\pi H_i^2/L) \coth(2\pi d/L) \qquad 4.$$

For finite slopes, θ the run-up can be approximated by:

$$R_s = (\pi/2\theta)^{1/2} H_i + \pi H^2/L \coth(2\pi d/L) \qquad 5.$$

When the waves break on the slope the Irribaren Number can be used to predict run-up:

$$R_s = \xi H_I \quad \text{- Hunt's formula.} \qquad 6.$$

To allow for slope roughness, R_s is multiplied by a factor, R_r, which accounts for energy
loss in the run-up, $R = R_s R_r$. R_r is between 0.9 and 1.0 for smooth slopes and 0.5 to 0.7 for slopes covered with rip-rap.

10.5 Vertical Walls.

Vertically faced sea walls might be founded on the sea bed itself or on a rubble mound, figures 1 and 2.

For the case of waves that do not break on the wall and which will be almost totally reflected Sainflou's formula is used to determine the total maximum pressure on the wall, as shown in figure 1.

If the wave height at the wall is H then, for h > 2H:

$$p_1 = (p_2 + \rho g h)(H + \delta_o)/(h + H + \delta_o) \qquad 7.$$

$$p_2 = \rho g H / \cosh(2\pi h/L) \qquad 8.$$

$$\delta_o = (\pi H^2/L) \coth(2\pi h/L) \qquad 9.$$

There is no established rule for which value of H to use in these formulae, most designers use H_S but some use $H_{1/10}$.

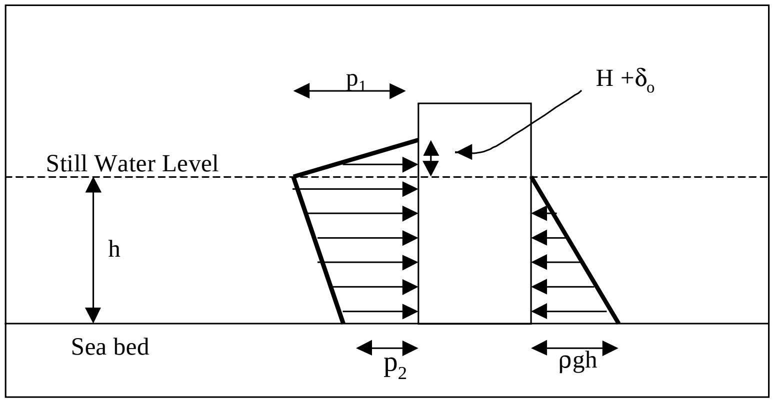

Figure 1. Pressures due to Unbroken Waves on a Vertical Wall

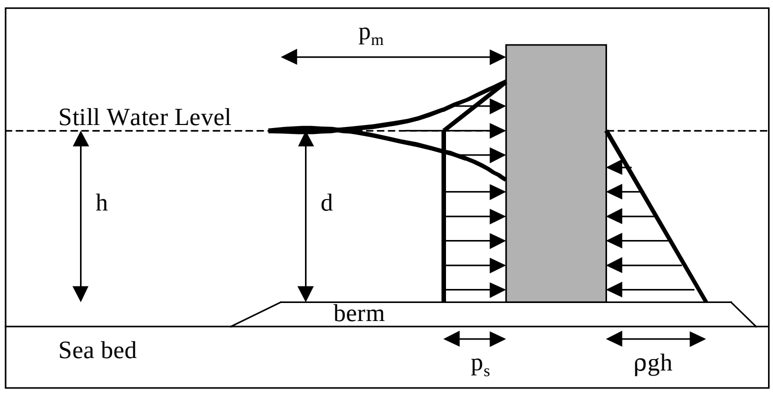

Figure 2. Pressures due to Waves Breaking on a Vertical Wall

For <u>waves breaking on the wall,</u> figure 2, the Minikin formula is used to represent the impact force maximum at the still water level, p_m, with a parabolic pressure distribution:

$$p_m = p_{max} (1 - 2|z|/H^2) \qquad \text{for } H/2 > z > -H/2 \qquad 10.$$

$$p_{max} = 101\ \rho gd (1 + d/h) (H/L) \qquad 11.$$

where d is the depth of water on the berm; d = h if there is no berm.

$$p_s = 0.5\rho gH (1 – 2z/H) \qquad \text{for } 0 < z < H/2 \qquad 12.$$

$$= 0.5\ \rho g H \qquad \text{for } z < 0 \qquad 13.$$

Later developments of these formulae, see Goda (1992) for further references, give more accurate predictions of the pressures. It is, of course necessary to check the stability of the wall against sliding (a friction coefficient of 0.6 might be appropriate) and overturning, including the hydrostatic pressure distribution on the rear face if necessary, uplift pressures on the base of the wall and buoyancy forces.

10.6 Rubble Mound Structures.

These structures are popular in coastal defence because they reflect far less wave energy than vertical walls and if natural rock is used they blend in well with the context. There are many variations in rubble mound structure design; here we consider the basic design of a breakwater with a core of smaller rock, filter layers of larger rock and an outer protective or armour layer of large rock or man-made armour units. The latter have been developed to take the place of natural rock when the latter is in short supply and therefore expensive to import to the site or when the size of natural rock needed would be prohibitively large.

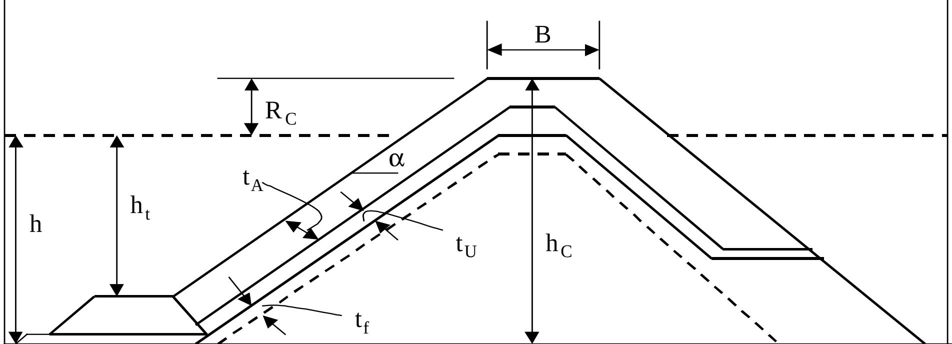

Figure 3. Main Parameters of a Breakwater Cross-section.

Design is normally based on the significant wave height, H_S, couple with the mean wave period, T_m, or the spectral peak wave period, T_p. A characteristic of the rock armour is its relative buoyant density, Δ, equal to $(\rho_r/\rho) - 1$, where ρ_r

is the density of the rock (typically 2500kg/m^3) and ρ is the density of sea water (1030kg/m^3).
The size of the rock is characterised by a "nominal diameter", D_{n50}, = $(M_{50}/\rho_r)^{1/3}$ where M_{50} is the median mass of the rock grading.

$H_S/\Delta D_{n50}$ is a key design parameter and is denoted by N_S
The parameters related to the cross-section of the breakwater are illustrated in figure 3.

10.6.1 Rock Armour Layers

The first equation relating rock size to wave parameters was given by Hudson and is still used in rock breakwater design:

$$M_{50} = \rho_r H^3 / (K_D \Delta^3 \cot \alpha) \qquad 14.$$

Where K_D is a stability coefficient taking onto account all the other variables. K_D values in the literature are for "no damage" conditions defined so that up to 5% of the armour rock may be displaced. For rough angular rock placed in two layers on the trunk of a breakwater K_D = 2.0 for non-breaking waves and K_D = 4.0 for breaking waves, with the <u>H in the equation being taken as $H_{1/10}$</u>. Here "breaking" means wave breaking directly onto the armour rocks. The Hudson formula has the advantage of simplicity but later research has resulted in improved formulations. Noting that equation 14 can be re-written as:

$$H_S/\Delta D_{n50} = N_S = (K_D \cot \alpha)^3 \qquad 15.$$

Van der Meer gave two equations:
For plunging waves on the armour face:

$$H_S/D_{n50} = 6.2\ P^{0.8}\ (S/(N_S)^{1/2})0.2\ \xi_m^{-0.5} \qquad 16.$$

and for surging breakers:

$$H_S/D_{n50} = 1.0\ P^{-0.13}\ (S/(N_S)^{1/2})^{0.2}\ (\cot\alpha)^{0.5}\ \xi_m^{P} \qquad 17.$$

Here S is a "damage level" = $A_e/(D_{n50})^2$ where A_e is the erosion area around the still water level per unit length of breakwater. P is the permeability factor (around 0.1 for a rock armoured structure with an impermeable core and 0.4 to 0.6 for porous mounds with high permeability). ξ_m is the "surf similarity parameter" based on the mean wave period (subscript m), that is, $\xi_m = \tan \alpha / (H_o L_o)^{1/2}$ and N_S is defined above. Van der Meer also gave a critical value of the surf similarity parameter for the transition from plunging to surging breakers as:

$$\xi_{mc} = [6.22\ P^{0.31}\ (\tan\alpha)^{1/2}]^k \qquad \text{with } k = 1/(P + 0.5) \qquad 18.$$

For design, the following values for S are used:

Slope	Initial Damage	Intermediate Damage	Failure
1 : 1.5	2	3 to 5	8
1 : 2	2	4 to 6	8
1 : 3	2	6 to 9	12
1 : 4	3	8 to 12	17
1 : 6	3	8 to 12	17

10.6.2 Run-up.

The run-up of waves on a rock slope is obviously critical to its performance in coast protection to minimise overtopping and potential flooding. Van de Meer, based on empirical studies related run-up, R, to the surf similarity parameter, ξ_m, although other methods are sometimes used.

$R/H_s = a\xi_m$ for $\xi_m < 1.5$ 19.

$R/H_s = b\xi_m^{\ c}$ for $\xi_m > 1.5$ 20.

$R/H_s = d$ for a permeable structure ($P > 0.4$) 21.

Recognising that different exceedence levels will apply to run up in random waves, a,b,c and d are given in the table below:

Level (%)	a	b	c	d
0.1	1.12	1.34	0.55	2.58
1	1.10	1.24	0.48	2.15
2	0.96	1.17	0.46	1.97
5	0.86	1.05	0.44	1.68
10	0.77	0.94	0.42	1.45
significant	0.72	0.88	0.41	1.35
mean	0.47	0.60	0.34	0.82

10.6.3 Overtopping.

The overtopping rate is often the primary design criterion for sea wall and breakwater design since their function is to protect land from flooding.
Defining a dimensionless parameter, Q^*, for the mean overtopping discharge by:

$$Q^* = [\ \overline{Q}\,(s/2\pi)^{1/2}] / (gH_s^{\,3})^{1/2} \qquad 22.$$

then $Q^* = a \exp(-bR_m^{\ *}/r)$ $\qquad R_m = (R_c/H_s)(s/2\pi)^{1/2}$ 23.

and $s = 2\pi H_s/gT_m^{\ 2}$ with T_m being the mean wave period. 24.

The coefficients a and b are derived from model test results for straight smooth slopes as:

Slope	a	b
1:1	0.00794	20.12
1:1.5	0.0102	20.12
1:2	0.0125	22.06
1:3	0.0163	31.9
1:4	0.0192	46.96
1:5	0.025	65.2

It is important to determine the allowable overtopping rate which is acceptable for various activities taking place in the area at the top of the breakwater slope. The following table gives those values in **litres per second per metre**.

	Safe	Unsafe	Dangerous
Vehicles	<0.001	<0.02	>0.02
Pedestrians	<0.005	<0.05	>0.05
Buildings	<0.001	<0.05	>0.05
Embankments	< 1.0	< 20	> 20
Revetments	< 75	< 200	>200

<u>Note that these are indicative values only and for design it is necessary to refer to the original tables.</u>

10.6.4 Structural Aspects.

The thicknesses of the armour, underlayer and filter layer, t_a, t_u and t_f, respectively are given by:

$$t_a = t_u = t_f = n\, k_t\, D_{n50} \qquad 25.$$

Where n = the number of layers and k_t = layer thickness coefficients.
The number of units per m^3 , N_a is given by:

$$N_a = n\, k_t\,(1 - nv)(Dn50)^{-2}$$ where n_v is the volumetric permeability 26.

Unit	k_t	n_v
Smooth rock n = 2	1.02	0.38
Rough rock n = 2	1.00	0.37
Rough rock n > 3	1.00	0.40
Graded rock	-	0.37
Cubes	1.10	0.47
Tetrapods	1.04	0.50
Dolosse	0.94	0.56

Note here that the table includes man-made armour units.
The bottom of the armour layer should extend down-slope to at least one significant wave height below the lowest water level.

The weight of armour units is given by the Hudson formula, equation 14, with appropriate K_D values. Typically, for rough angular rock placed in two layers $K_D = 2.0$ for non-breaking waves and 4.0 for breaking waves, the latter breaking directly onto the armour.

The underlayer rock is normally one-tenth to one-fifteenth of the armour rock weight but sometimes such layers are omitted. The filter layer must then be designed so that filter material will not pass through the armour face. In this case geotechnical filter rules are applied so that D_{15} (armour) / D_{85}(filter) > 4 to 5.

Toe protection is an important feature in rock armoured and artificially armoured breakwater design. If the rock in the toe is the same weight as the armour then it will be stable but it is often desirable to reduce the toe armour weight for economic reasons. For this refinement reference must be made to the literature.

The Breakwater Head is also a critical feature in design since it suffers a higher level of wave attack than the trunk. K_D values for breakwater heads are given in the literature.

This is a very brief summary of the major features of the design of a rock armoured breakwater. Additionally, in region where rock is expensive a wide range of man-made armour units has been used as armour, ranging from simple concrete cubes to more complex interlocking shapes such as tetrapods and dolosse.

Another type of unit is in the form of hollow blocks of various shapes. These are laid in a close pattern and in a single layer which results in tight interlock and high resistance to damage. In general it is necessary to pay a fee for the use of patented artificial armour units.

Revetments – rock or artificially armoured slopes used as coast defence**,** are important structures and similar design factors apply. The armour is sized on the same basis as for detached or harbour breakwaters but, in addition to toe stability, underlayers and their function to retain the fine material in the core of the structure, an additional important factor is the continuity of the slope at its upper end, where it often meets a reflective, curved wave wall. It is at this point that wave action often results in partial failure of the rock armour. There is a tendency for revetment slopes to be relatively steep, using large armour rocks, so that the slope does not encroach on the beach area. However, it is

10.7. Wave Loading on Cylinders.

Cylinders are a popular form of structural element in coastal and offshore engineering. Many offshore platforms and jetties are pile-supported. Wave loading on cylinders is therefore an important matter and it is also important in design to consider uplift forces on the pier deck if there is any possibility of waves hitting its underside.

10.7.1 The Morison Regime.

The hydrodynamic field acting on a cylinder can be derived from linear or higher order wave theory, so the time variation of velocity and acceleration of the water are known.

The fluid loading consists of a **“drag” force**, proportional to the **velocity squared**, and an **“inertia” force**, proportional to the local **fluid acceleration**. The origin of the drag force is obvious, resulting from flow separation around the cylinder with a non-symmetric pressure distribution. The inertia force arises from the fact that the presence of the cylinder will prevent accelerations of the water that would otherwise have been present. To do this a force must be imposed on the flow, with an equal and opposite reaction on the cylinder.
The total force, dF, acting on an elemental length of cylinder, ds, is given by

$$dF = dF_D + dF_M \qquad 27.$$

Where the “drag” component is $dF_D = ½\, C_D\, \rho\, D\ u|u|\, ds$ 28.

and the “inertia component is $dF_M = C_M\, \rho\, (\pi D^2/4)\, (Du/Dt)\, ds$ 29.

This is Morison’s equation, first proposed in 1956, and still widely used in the calculation of wave loading on cylinders.

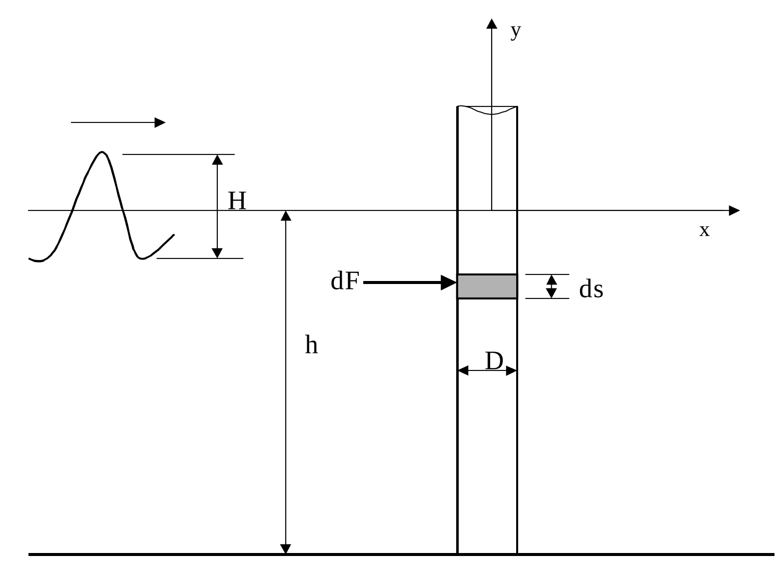

Figure 4. Definition Sketch for Wave loading on Cylinders.

C_D and C_M are the drag and inertia coefficients respectively and ρ is the density of water,
u is the instantaneous value of the horizontal water particle velocity and Du/Dt is the instantaneous value of the horizontal water particle acceleration. The absolute value on one of the velocity terms ensures that the drag force always acts in the direction of the water flow.
In ideal flow $C_M = 2.0$ but this does not account for wake effects and therefore C_M must be determined experimentally, as must the drag coefficient, C_D.

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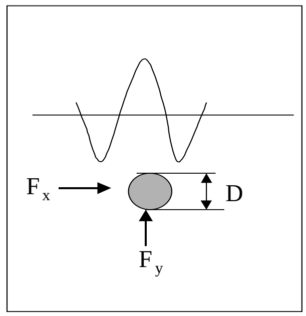

Figure 5. Horizontal Cylinder

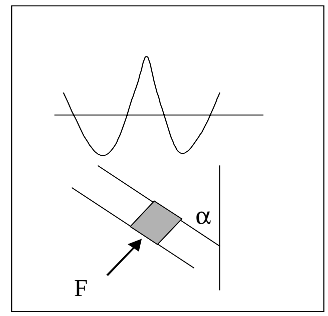

Figure 6. Inclined Cylinder

Figure 5 illustrates a horizontal cylinder in waves

In this case the drag force acts in the direction of the local velocity vector which has a magnitude $(u^2 + v^2)^{1/2}$. Its horizontal component is therefore $1/2C_d \rho D u(u^2 + v^2)$.

Similarly the inertia force acts in the direction of the local acceleration vector and has a horizontal component of $C_M \rho (\pi D^2/4) (Du/Dt)$.

The total forces are therefore:

$$F_x = 1/2C_D\rho D u(u^2 + v^2)^{1/2} + C_M \rho (\pi D^2/4) (Du/Dt) \qquad 30.$$

$$F_y = 1/2C_D\rho D v(u^2 + v^2)^{1/2} + C_M \rho (\pi D^2/4)(Du/Dt) \qquad 31.$$

In the case of an inclined cylinder, figure 7, the force per unit length is given by:

$$F = 1/2C_D\rho D V_N|V_N| + C_M\rho (\pi D^2/4)(DV_N/Dt) \qquad 32.$$

V_N and DV_N/Dt are the instantaneous components of the water particle velocity and acceleration normal to the axis of the cylinder, respectively.

First order wave theory gives these as:

$$V_N = (\pi H/T) \left\{\frac{\cosh[k(y+d)]}{\sinh(kd)} \cos(kx - \omega t) \cos\alpha + \sinh[k(y+d)] \sin(kx - \omega t) \sin\alpha\right\} \qquad 33.$$

$$DV_N/Dt = (2\pi^2 H/T^2) \left\{\frac{\cosh[k(y+d)]}{\sinh(kd)} \sin(kx-\omega t) \cos\alpha - \sinh[k(y+d)] \cos(kx - \omega t) \sin\alpha\right\} \qquad 34.$$

For steady, uni-directional flow the values of C_D are well established as a function of the Reynols Number, N_R, ($N_R = VD/\nu$, where ν is the kinematic viscosity of water, $\approx 10^{-6}$ m^2/s) but the unsteady case is far more complex. Numerous experimental studies have been performed, including some at full scale, and there is still uncertainty over appropriate values to be used, especially considering that in the sea cylinder may quickly be covered in marine growth which will certainly influence C_D and C_M. Design guidance notes such as the British Standards give recommended values, typically between 0.5 and 1.2 for C_D, ($\approx$ 0.6 for $N_R > 5 \times 10^6$) and between 1.5 and 2.0 for C_M.
If the body is moving relative to the wave then Relative velocities and accelerations must be used to determine the wave loading via the Morison equation.
The ratio of the maximum drag to maximum inertia force, from equations 28 and 29 and using linear wave theory is:

$$F_D(max)/F_M(max) = (C_D/C_M)(K/\pi^2) \qquad 35.$$

Where K is the non-dimensional "Keulegan-Carpenter Number" = $U_{max}T/D$ which is the ratio of the amplitude of the wave motion to the cylinder size.
Clearly, as the diameter of the cylinder increases the inertia component of the total force becomes dominant.

10.7.2 The Diffraction Regime.

When the cylinder diameter is of the order of 0.2 times the wave length the incident wave field is scattered as out-going reflected waves. This then dominates the loading condition and is known as the "diffraction" regime – with parallels to the diffraction of waves at a breakwater. Large diameter structures may not be regular in shape and for diffraction regime numerical models must be used to find the loading. Some analytical solutions are available for simple shapes. For a cylinder a solution of the Laplace equation including the scattered wave gives an expression for the force at a given elevation below still water level, F(y), and the total force on the cylinder, F, as:

$$F(y) = (\pi/8)(\rho g H D^2)\,\frac{\cosh[k(y+d)]}{\cosh(kd)}\,C_M \cos(\omega t - \theta) \qquad 36.$$

$$F = (\pi 8)(\rho g H D^2)\tanh(kd)\cos(\omega t - \theta) \qquad 37.$$

Where θ is the phase angle relative to the wave and C_M is an effective inertia coefficient.
C_M is a function of ka where $k = 2\pi/L$ and $a = H/2$. C_M = 2.0 at ka = 0 reducing to 1.0 at ka= 1.0 and to 0.3 at ka = 3.0.
Obviously wave loading on a ship fall into the diffraction regime but prediction of the loading is complicated by the fact that the vessel may be moving in some or

all of its six modes of motion (surge, sway, heave, pitch, roll and yaw). In this case, as with moving cylinders, the relative velocities and accelerations must be used.
For design the choice of wave height is obviously critical. For offshore structures it is usually necessary to predict the **maximum individual wave height** with a return period of 50 or 100 years, using extreme value probability density functions as discussed in the random waves section of these notes.
Clearly, in a design situation reference must be made to guidance notes for the selection of C_D and C_M values for the specific conditions.

10.7.3 Vortex Shedding.

In many coastal and offshore engineering applications an important phenomenon is often overlooked, that of vortex shedding. In uni-directional flow at a velocity, V, a body will shed vortices at a specific frequency. If this frequency corresponds to the natural, damped frequency of the body, a resonant response will be generated which can result in large oscillations and early failure due to fatigue.
Vortex shedding can be characterised by the non-dimensional Strouhal Number, N_S.

$$N_S = f_e . D / V \qquad 38.$$

where f_e is the eddy shedding frequency.
For circular cylinders, $N_S \approx 0.20$ for Reynolds numbers, N_R , between 10^3 and 2×10^5.

10.8. Summary.

A range of methods is available to calculate wave-induced loading on a variety of coastal and offshore structures. The literature contains good guidance on the coefficients that inevitably have to be used in the design process but it is always important to test the sensitivity of calculated loading to changes in those coefficients.
Wave conditions for design are often difficult to determine with precision due to lack of measured data, for example. Although loads calculated for design will always be subject to a design safety factor but uncertainty in calculations might reduce that safety factor to 1.0 which is not acceptable. It is therefore essential to consider the cost of designing additional capacity into the structure because this cost is often marginal in comparison to the overall cost of the project. Equally it is good practise to design so that the capacity of a structure can be increased at limited cost if required at some time in the future, e.g., as a result of increased wave activity due to the consequences of global warming.
In some cases it will be appropriate to build a physical model of the design for laboratory testing which generally requires a channel with a computer controlled wave generator capable of generating random waves with specified wave spectra.
