Technische Hogeschool Delft
Afdeling der Civiele Techniek

Hydrodynamic Aspects of Fixed Offshore Structures

Coastal Engineering Group
Workgroup Offshore Technology
Department of Civil Engineering
Delft University of Technology
Delft, The Netherlands

OFFSHORE TECHNOLOGY

HYDRODYNAMIC ASPECTS OF OFFSHORE STRUCTURES

W.W. Massie
Registered Professional Engineer
Senior Member of the Scientific Staff
Coastal Engineering Group
Department of Civil Engineering

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Delft University of Technology
Work Group Offshore Technology
Delft
The Netherlands.

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Scientists study the world as it is;
Engineers create the world that has never been.

Theodore von Kármán.

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1. INTRODUCTION

These brief notes are intended to provide the student of general offshore engineering some insight in the hydraulic and oceanographical engineering aspects of offshore engineering problems. Because of the limited nature of this class and the varied background of the students involved, the coverage, here, will be summary; only the most important topics will be highlighted. Where available, literature references will be given where those interested can find more extensive information.

The topics to be treated in the following chapters include:

- Ocean waves and their most important properties.
- Other ocean water movements
- Wave forces on slender cylindrical bodies.
- Choice of design wave conditions.

These items are discussed in more detail in the following chapters.

2. OCEAN WAVES

2.1 Introduction

Some knowledge of the properties and mechanics of ocean waves is essential to succesful offshore work. Waves and currents can cause very significant loads on offshore structures of all types and are usually experienced as a nuisance by most everyone working offshore.

Only results of theoretical derivations are given in the following sections and these results are even limited to offshore conditions. A broader overview of such results is available in notes available in coastal engineering - Massie, editor (1976). Kinsman (1965) presents an excelent and readable discussion of the theoretical background.

2.2 Wave Characteristics

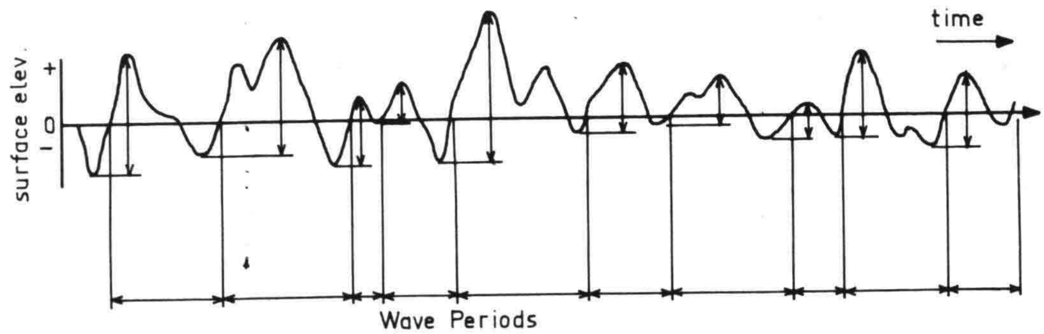
If we attempt a quantatative description of ocean surface waves we can do that most conveniently by noting the height and period of the waves. While these two quantities may seem simple to determine, oceanographers argue continually about the proper definitions for wave height and period in an actual wave record. Rather than join in that discussion, here, we shall define these terms according to common, but not universal practice, using the sketch of water surface elevation versus time shown in figure 2.1.

The wave height is defined as the vertical elevation difference between a wave through (low point) and the following wave crest (high point). This height is usually denoted by H. Often an additional restriction must be placed on the above definition: The crest must be above the mean water level and the through must be below this level; see figure 2.1a.

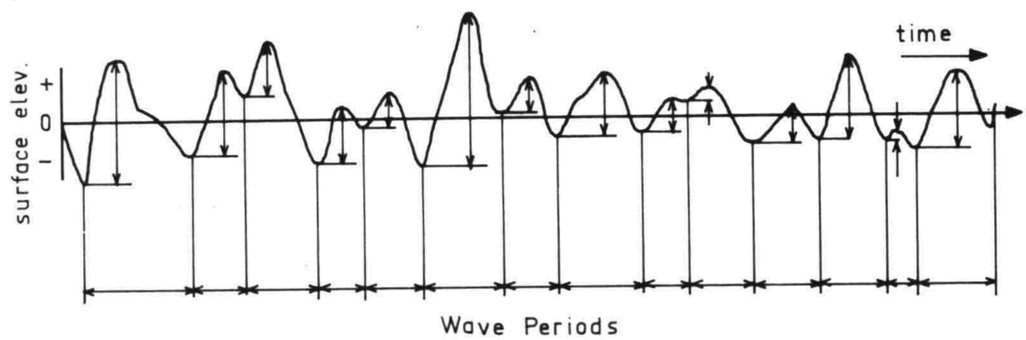
The wave amplitude is derived from the wave height and is the height of a wave crest relative to the mean water level. Schematization of an individual wave by a simple sine wave yields a conclusion that the wave amplitude is half the wave height.

The wave period is usually defined as the time interval between two successive upward crossings of the mean water level. This is often more easily determined than, say, a trough to trough period.

Wave heights in excess of 30 meters can exist at sea; wave period usually range between about 3 and 30 seconds.



a. Trough to peak height with zero crossing.
Upward zero crossing Period.



b. Trough to peak height.
Trough to trough period.

Figure 2.1 RECORDS OF WATER LEVEL VERSUS TIME WITH VARIOUS DEFINITIONS FOR H AND T.

2.3 Physical Phenomona

If we examine a record of water surface elevation versus time for a pattern of regular sinusoidal waves, we see that the elevation pattern repeats itself with a period, T, equal to the wave period. The pattern repeats with a circular frequency of

$$\omega = \frac{2\pi}{T} \text{ rad./sec.} \tag{2.01}$$

Similarly, if we examine the wave pattern at some instant, we see that the pattern repeats itself at regular intervals as well. This interval is called the wave length, λ. In a way parallel to that for the frequency, ω, we can define a wave number, k, as:

$$k = \frac{2\pi}{\lambda} \text{ rad./m.} \tag{2.02}$$

Continuing our assumption of a sinusoidal wave, we can now write out an equation for the water surface elevation as a function of the wave height, length, and period as well as time, t, and location, x.

$$\eta = \frac{H}{2} \sin(\omega t - kx) \qquad (2.03)$$

where: η is the elevation of the water surface at point x and time t.

The speed at which a wave crest passes along the ocean surface is given by:

$$c = \frac{\lambda}{T} = \frac{g}{2\pi} T \qquad (2.04)$$

when the wave occurs in relatively deep water (depth, h, > $\lambda/2$). (Relationships for other conditions will not be given here.)

It should be obvious from (2.04) that a direct relationship exist between wave period and wave length in deep water. Indeed:

$$\lambda = \frac{g}{2\pi} T^2 = 1.56\ T^2 \quad \text{(metric units)} \qquad (2.05)$$

It can be handy to remember that a sort of ordinary Noth Sea wave has a period of about 8 seconds with a wave length of about 100 m in deep water.

A relationship between wave height and wave period (or wave length) is less easily defined even though some relation must exist. (The shortest waves at sea are not the highest and often the longest waves are not the highest either. This latter comment is especially true if the long waves are a swell radiated from a distant storm as opposed to locally generated storm waves.)

In deep water, waves will break when their height exceeds about 1/7 of their wave length. This, then, set a limit on the wave height that is a function of the wave length and hence its period.

2.4 Other Wave Reationships

How does the water move in a wave? If we watch a float in the deep ocean we see it move up and down as wave crests pass. Also, it moves forward (in the direction of propagation of the wave) when on the crest and back when in the trough; its net horizontal movement is zero during a wave period.

Indeed the horizontal velocity of our float at the water surface is:

$$u = \frac{\omega H}{2} \sin\ (\omega t - kx) \qquad (2.06)$$

and its vertical velocity component is:

$$w = \frac{\omega H}{2} \cos\ (\omega t - kx) \qquad (2.07)$$

This is the parametric representation of a particle moving around a circle of radius $\frac{H}{2}$ with period, T.

Deeper in the water, this circular motion continues, but the radii of the circles decrease exponentially with depth. More complete versions of equations 2.06 and 2.07 are:

$$u = \frac{\omega H}{2} e^{kz} \sin(\omega t - kx) \tag{2.08}$$

$$w = \frac{\omega H}{2} e^{kz} \cos(\omega t - kx) \tag{2.09}$$

where z is a vertical coördinate measured upward (positive) from the water surface. Figure 2.2 sketches the orbital motion under a deep water wave.

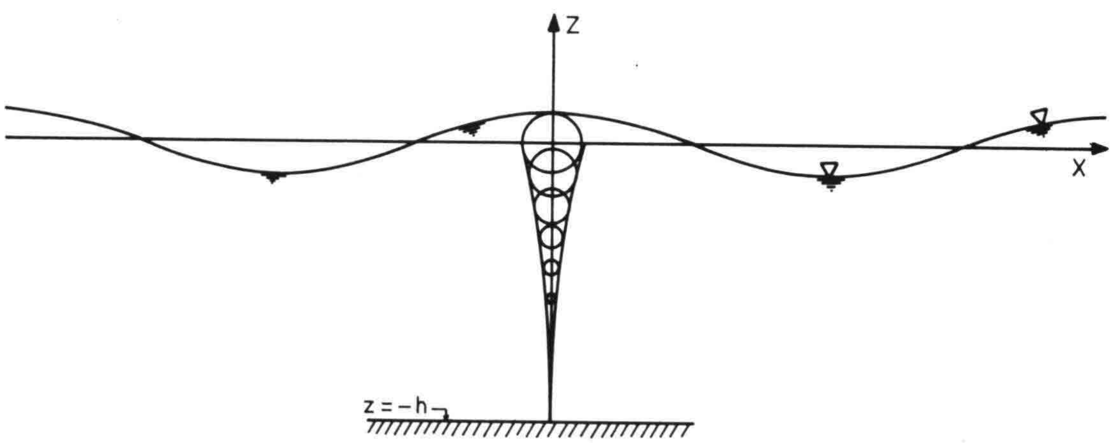


Figure 2.2
ORBITAL MOTION UNDER A DEEP WATER WAVE

This decrease in wave influence below the ocean surface explains somewhat the relative stability of structures such as submarines and semi-submersibles in wave action.

Wave also posess energy. This energy includes both kinetic and potential energy. It is most convenient to express energy in units of energy per unit wave crest length and unit wave length (in other words, per unit ocean surface area). In such units:

$$E = \frac{1}{8} \rho g H^2 \tag{2.10}$$

where g is the acceleration of gravity and

ρ is the mass density of water.

This energy is dependent only upon the wave height; it is independent of the wave period.

As a storm progresses, the energy of its waves must also be propagated forward. Close examination of the "front" of such a wave field will reveal that the individual waves more forward with a celerity, or speed, c, which is twice as fast as the wave field (group) moves forward as a whole. This latter, slower velocity with which the energy is propagated is referred to as the wave group velocity; in deep water its value is:

$$c_g = c/2 \qquad (2.11)$$

In the next section we return to the problem of the "real" ocean by combining a number of waves.

2.5 Wave Statistics

How can we use the results of the previous section (derived for a simple sinusoidal wave) to describe a real sea? We can do this most easily by expressing this real sea as a sum (theoretically infinite) of sine waves, each with its own amplitude, a_i and phase, ϕ_i:

$$\eta(t) = \sum_{i=1}^{\infty} a_i \sin(\omega_i t - \phi_i) \qquad (2.12)$$

The coordinate x does not appear in (2.12) since we are restricting ourselves to one location. Equation 2.12 can be compared to (2.03) - remember that the wave height, H, is twice the amplitude, a.

The total energy of such a set of components is:

$$\frac{1}{2} \sum_{i=1}^{\infty} a_i^2 \qquad (2.13)$$

which differs from equation 2.10 only in that ρg does not appear.

Noting that each component in (2.12) has its own (different) frequency, we can define a function $S(\omega)$ called an energy density function such that:

$$\frac{1}{2} a_i^2 = S(\omega_i)d\omega \qquad (2.14)$$

as shown in figure 2.3.

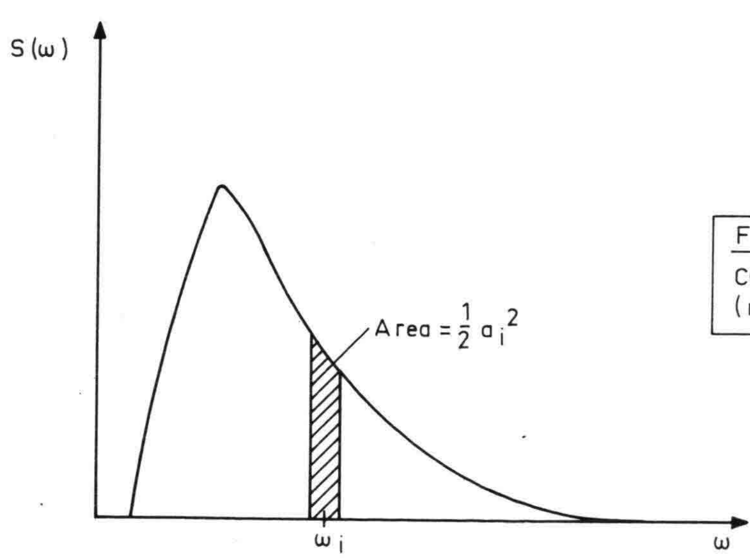


Figure 2.3
CONCEPT OF WAVE SPECTRUM
(no absolute scale)

Such a spectrum is nice, but it is not all that convenient. Imagine having to keep a whole series of graphs in order to record the storm wave history of the North Sea!

Realizing that the total area under the spectrum curve represents an energy, we can characterize this spectrum by some single wave height having an energy which is related to the spectrum energy in some way. For convenience, we can define a significant wave height, H_{sig}, as:

$$H_{sig} = 4\sqrt{\int_0^\infty S(\omega)d\omega} \qquad (2.15)$$

Lord Rayleigh examined the statistics of the sea surface and found that if the water surface elevation at any time was described by a normal distribution, then the distance between extremes (wave heights) were described by a Rayleigh Distribution. For this Rayleigh Distribution, the chance that a wave height, H, is exceeded in a storm characterized by H_{sig} is:

$$P(H) = e^{-2\left(\frac{H}{H_{sig}}\right)^2} \qquad (2.16)$$

which again involves the significant wave height. Table 2.1 lists some values from equation 2.16. We see from the table that the significant wave height, H_{sig}, is exceeded by 13.5% of the waves. Also, it can be shown that H_{sig} is also equal to the average of all waves for which P(H) is less than 1/3. Additionally, and of significant emperical importance, the significant wave height corresponds well to the wave height determined by experienced visual observation.

What does all this mean? It means that we can characterize the spectrum of each storm by a single value, the significant wave height*. Further, we can determine the chance that any given wave height occurs using the Rayleigh Distribution and the given significant wave height.

We can also carry out some statistical work on the series of significant wave height values, each characterizing a single storm. For example, for the southern part of the North Sea, a linear graph of H_{sig} versus log frequency of occurrence results. A few values are given in table 2.2.

In chapter 5, we shall use these statistical relationships in order to determine wave forces on structures and the chance that a given force will be exceeded.

In the remainder of this chapter we briefly examine the other causes of water movement in the oceans which must be considered in the offshore industry.

* We have neglected the frequency (period) data.

Table 2.1 Properties of Rayleigh Distribution

Probability of exceedance P(H)	$\frac{H}{H_{sig}}$
10^{-5}	2.40
$2x10^{-5}$	2.33
$5x10^{-5}$	2.22
10^{-4}	2.15
$2x10^{-4}$	2.06
$5x10^{-4}$	1.95
10^{-3}	1.86
$2x10^{-3}$	1.77
$5x10^{-3}$	1.63
0.01	1.51
0.02	1.40
0.05	1.22
0.10	1.07
0.125	1.02
0.135	1.000
0.20	0.898
0.50	0.587
1.00	0.000

Table 2.2 Significant Wave Heights in Southern North Sea

frequency of exceedance (storms/year)	Significant Wave Height, H_{sig} (m)
10	4.2
5	4.6
2	5.2
1	5.7
0.5	6.1
0.2	6.7
0.1	7.1
0.05	7.6
0.02	8.2
0.01	8.7

3. OTHER OCEAN FLOW PHENOMONA

3.1 Tides

Another source of currents in the oceans the tides. Their influence is most pronounced in areas such as the North Sea and other rather restricted (from an oceanographic view point) waters. Tidal currents well away from the coasts - such as in the mid-Atlantic are generally negligible.

3.2 Ocean Currents

Wind forces and a Coriolis acceleration resulting from the rotation of the earth on its axis cause circulation currents in the ocean. These currents are usually found outside the continental shelves in deep water. Extreme velocities in the order of 1.5 to 2 m/s can be found, for example, near Florida, U.S.A. in the Florida Current - part of the Gulf Stream. The flow involved in such currents is enormous - 60×10^6 m^3/s for the Gulf Stream. Svedrup, Johnson and Fleming (1942) give an excellent summary of the ocean currents then known. (A few equatorial currents have been discovered since then).

3.3 Internal Waves

In certain parts of the world the oceans are stratifed; layers of different density can be found. Internal waves can then develop and propagate along the interface between layers, much like those on the surface between water and air.

Because of the small density difference between layers, the gravitational influence is relatively small on such waves. To compensate for this, they can be very high - in the order of 50 meters is rather common. They move slowly, however (2 m/s for example) and have somewhat longer periods than surface waves. (Periods in the order of 20 minutes are common). Osborne, et al (1977) describe experiences with such waves while drilling in the Andaman Sea (between Burma and Sumatra). Maximum currents observed there were a bit more than 0.5 m/s at a depth of about 110 m.

3.4 Tsunamis

Tsunamis, sometimes incorrectly called tidal waves, are ocean waves generated by geologic action of the sea bed. Actions such as earthquakes or the explosion of submarine volcanoes have been known to cause them. Tsunamis, thus, have nothing to do with tides. The word tsunami comes from Japan where such waves are all too common.

Tsunami waves can range up to tens of meters high (at least near the shore where they are usually observed) and have periods ranging from a few minutes up to, say, one half hour. Usually only one (or at most a few) such wave is generated by a given seismic activity.

4. HYDRODYNAMIC FORCES ON CIRCULAR CYLINDERS

4.1 Introduction

Since the early 1950's an enormous amount of research has been invested in the determination of the hydrodynamic forces on slender circular cylinders. Progress seems, at times, to be slow and no single method to predict the wave and current forces on structural elements of, say, a jacket structure is universally accepted. In this chapter, an attempt will be made to explain the more popular theories. In the following section, we start by discussing the force components acting on a unit length of cylinder placed perpendicular to a two-dimensional flow. Slender, in this discussion implies that the flow characteristics around the cylinder can be characterized by the flow conditions at a single point corresponding to the location of the cylinder axis in an undisturbed flow pattern. As such, this implies that the cylinder diameter is much smaller than the wave length, λ.

4.2 Hydrodynamic Force Components

Consider a cylinder of diameter, D, and unit length placed with its axis perpendicular to an infinite constant uniform velocity field. This unit length of cylinder will experience a drag force, F_D, of:

$$F_D = \left(\frac{1}{2} \rho |V| V \right)(D\ 1)(C_D) \qquad (4.01)$$

where: D is the diameter of the cylinder,
V is the undisturbed velocity,
ρ is the mass density of water, and
C_D is an experimental coefficient.

This drag force is, thus, proportional to the kinetic energy of the undisturbed flow, times the projected area obstructing the flow, times a dimensionless coefficient. Usual values of C_D range from about 0.5 to about 1.5. The drag force acts in the same direction as the velocity, and is caused, primarily by the pressure difference existing between the "front" and "back" of the cylinder.

A second force component, the lift force, acts along a line perpendicular to the flow direction. It can be described by:

$$F_L = \left(\frac{1}{2} \rho\ V^2 \right)(D\ 1)(C_L)(\sin 2\pi f t) \qquad (4.02)$$

where: f is the frequency with which eddies are shed in the vortex street behind the cylinder, and
C_L is an experimental lift coefficient.

The lift force is proportional to the same sorts of quantities as the drag force, but fluctuates in a sinusoidal way with a frequency equal to the frequency with which eddies are shed. The lift force is apparently caused by the alternate eddy formation in the wake of the cylinder. The lift force is only important, thus, when such eddy formation is present.

The above two force components are the only ones present in a uniform steady flow.

If, we now allow the undisturbed flow to oscillate as a function of time a third force component, the inertia force, appears. This force component is described by:

$$F_I = \left(\frac{1}{4} \pi D^2 \rho 1 \right) \left(\frac{\partial V}{\partial t} \right) (C_M) \tag{4.03}$$

The inertia force is proportional to the acceleration of the water times the mass of water displaced by the cylinder, times an experimental coefficient, C_M. The force is directed in the same way as the instantaneous acceleration.

Morison, et al (1952) seems to be the first to have suggested a formula for the wave force acting on a vertical circular cylinder. The formula which bears his name is:

$$F = F_D + F_I$$

$$dF = \frac{1}{2} \rho u|u| C_D DdL + \frac{1}{4} \pi D^2 \rho \frac{\partial u}{\partial t} C_M dL \tag{4.04}$$

where: dF acts on an element of length dL, and

u is the horizontal component of the velocity in the wave (equation 2.06). Morison assumed, probably unconsiously, that velocity and acceleration components parallel to the axis of the cylinder did not contribute to the hydrodynamic force in the direction perpendicular to the cylinder axis.

Why did Morison neglect the lift force? There are probably two reasons: First, with a vertical cylinder in waves, the line of action of the lift force is perpendicular to the line of action of the other two force components. Secondly, the lift force is directly coupled on the eddy formation in the wake of the cylinder. Unless a single eddy extends over the entire length of the cylinder - very unlikely in view of the varying flow conditions under a wave - the resulting lift force - integrated over the cylinder length - will be much less than that predicted by an equation like 4.02. For these reasons lift forces are often neglected in the determination of design loads on an offshore structure *as a whole*, used, for example, to design the foundation. Lift forces may not be neglected, however, when considering, for example, vibration of an individual structural element.

Figure 4.1 shows the inertia and drag force components on an element of a vertical cylinder of 1 m length located at a depth of 10 m in infinitely deep water. The cylinder diameter is 0.5 m and the wave height and period are 5 m and 10 seconds, respectively. Values of C_M and C_D are chosen (quite arbitrairily for now), to be 1.2 and 0.7 respectively.

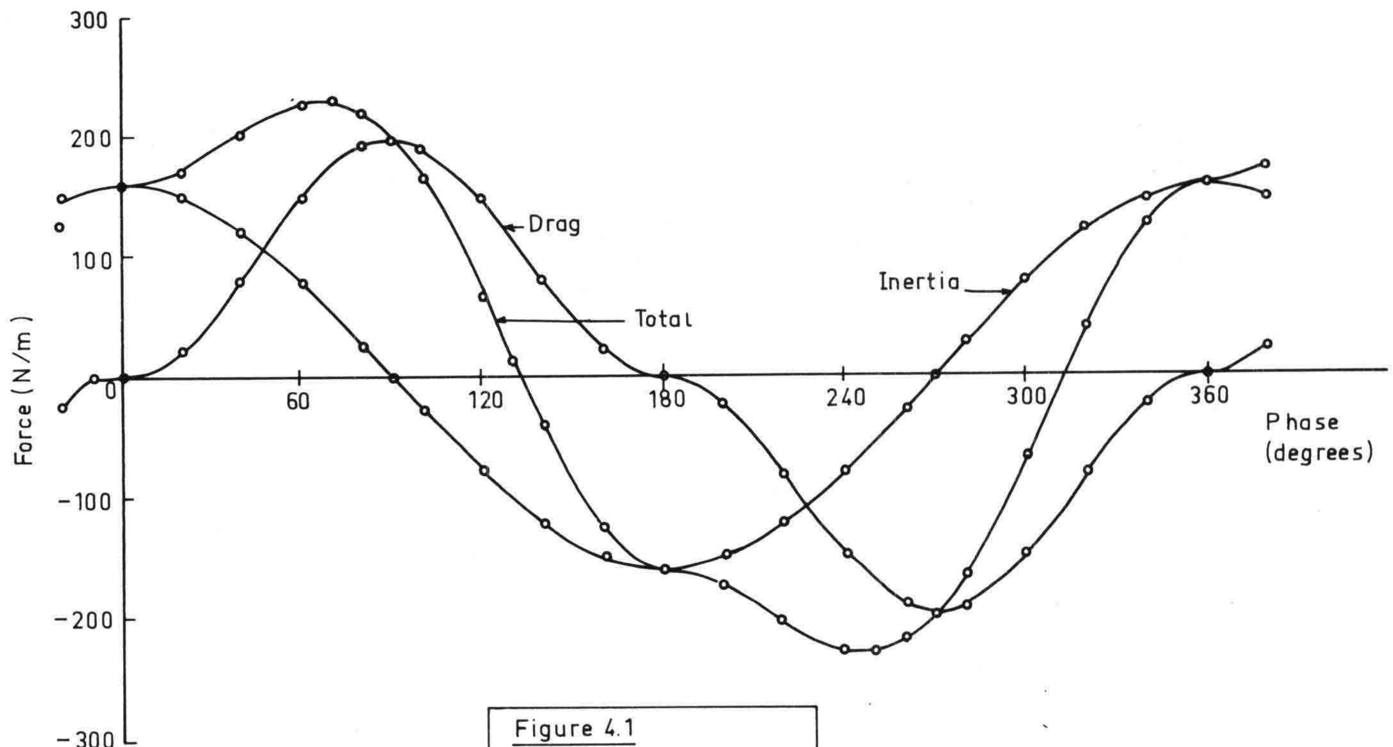


Figure 4.1
EXAMPLE OF INERTIA AND DRAG FORCES

Note that the drag force has a decidedly different character from the velocity. This comes from the fact that it is proportional to the square of the velocity. This non-linearity, a quadratic dependence upon velocity, will lead to many practical problems when wave forces are to be computed in real random seas. This will be discussed in more detail later in this chapter and in chapter 5.

Of course, the velocity and acceleration components are 90^0 out of phase. This implies that the maximum drag force occurs when the inertia force is zero and visa versa. Note, also, that the maximum force does not, in general, occur at either of these times.

4.3 Sloping Cylinders

With the advent of the large steel offshore jacket structures, it has become increasingly important to predict hydrodynamic forces on cylinders having an arbitrary orientation relative to the waves. The most common procedure for calculating such lift and drag forces at present is to attribute the transverse force components to their respective perpendicular components of velocity and acceleration. Recent evidence from studies carried out here in Delft indicates that the above approach may not be correct. Unfortunately, testing has not yet progressed far enough to define a better prediction technique.

A more conservative but no more correct approach is to determine the force per unit length for the sloping cylinder in the same way as for a vertical cylinder using horizontal velocity and acceleration components. This resulting force per unit length is then applied *undimished* along the entire length of the sloping element. Such a procedure is recommended in the *Shore Protection Manual*; it is most likely conservative.

4.4 Parameters and Coefficients

The traditional parameter to which drag force coefficients in constant currents have been related for decades is the Reynolds Number, Re. It is defined as a ratio of viscous forces to inertia forces and is usually expressed as:

$$Re = \frac{VD}{\nu} \qquad (4.05)$$

where: ν is the kinematic viscosity of water (usually about 10^{-6} m^2/s). Indeed, a reasonably consistent experimental relationship exists between drag coefficient and Reynolds Number for constant currents.

Such a relationship is less successful in waves, however. Keulegan and Carpenter (1956) found that for an oscillatory flow, both the drag and inertia coefficients could be related to the Keulegan-Carpenter Number or Period Parameter:

$$KC = \frac{\hat{u}T}{D} \tag{4.06}$$

where: $\hat{u}$ is the maximum velocity component, and

T is the wave period.

If we assume, further, that the velocity component varies sinusoidally as a function of time, then KC can be expressed as:

$$KC = \pi^2 \frac{C_M}{C_D} \frac{\text{Drag force amplitude}}{\text{Inertia force amplitude}} \tag{4.07}$$

Thus, the Keulegan Carpenter Number can be seen as a ratio of drag force to inertia force in waves. Further, since C_M is often a bit larger than C_D, the two force components contribute about equally when KC $\sim$ 12.

Another physical interpretation of KC is the ratio of water displacement to cylinder diameter.

$$KC = 2\pi \frac{\text{water displacement amplitude}}{\text{cylinder diameter}} \tag{4.08}$$

When waves are combined with currents, the Keulegan-Carpenter Number loses significance. Also, as the Keulegan-Carpenter number increases, drag coefficient values approach those for a corresponding Reynolds Number in steady flow. This seems logical in light of equation 4.07, above. Since the inertia force becomes less important as KC increases, one still often finds graphs relating C_D to Re. - see, for example, volume II of the *Shore Protection Manual*.

The current tendency is to relate the coefficient values to both Reynolds and Keulegan - Carpenter Numbers. Figures 4.2 and 4.3 summarize the data of design interest.

It is well to note that many organizations include data such as presented in figures 4.2 and 4.3 in their own guides of recommended practice.

4.5 Waves Plus Currents

When currents are superimposed on the waves (a tide superimposed on waves, for example) one must be sure to add the necessary velocity components vectorially *before* computing drag forces. The resulting drag force will be directed perpendicular to the cylinder axis and be in the plane defined by the resulting velocity vector at that instant and the cylinder axis.

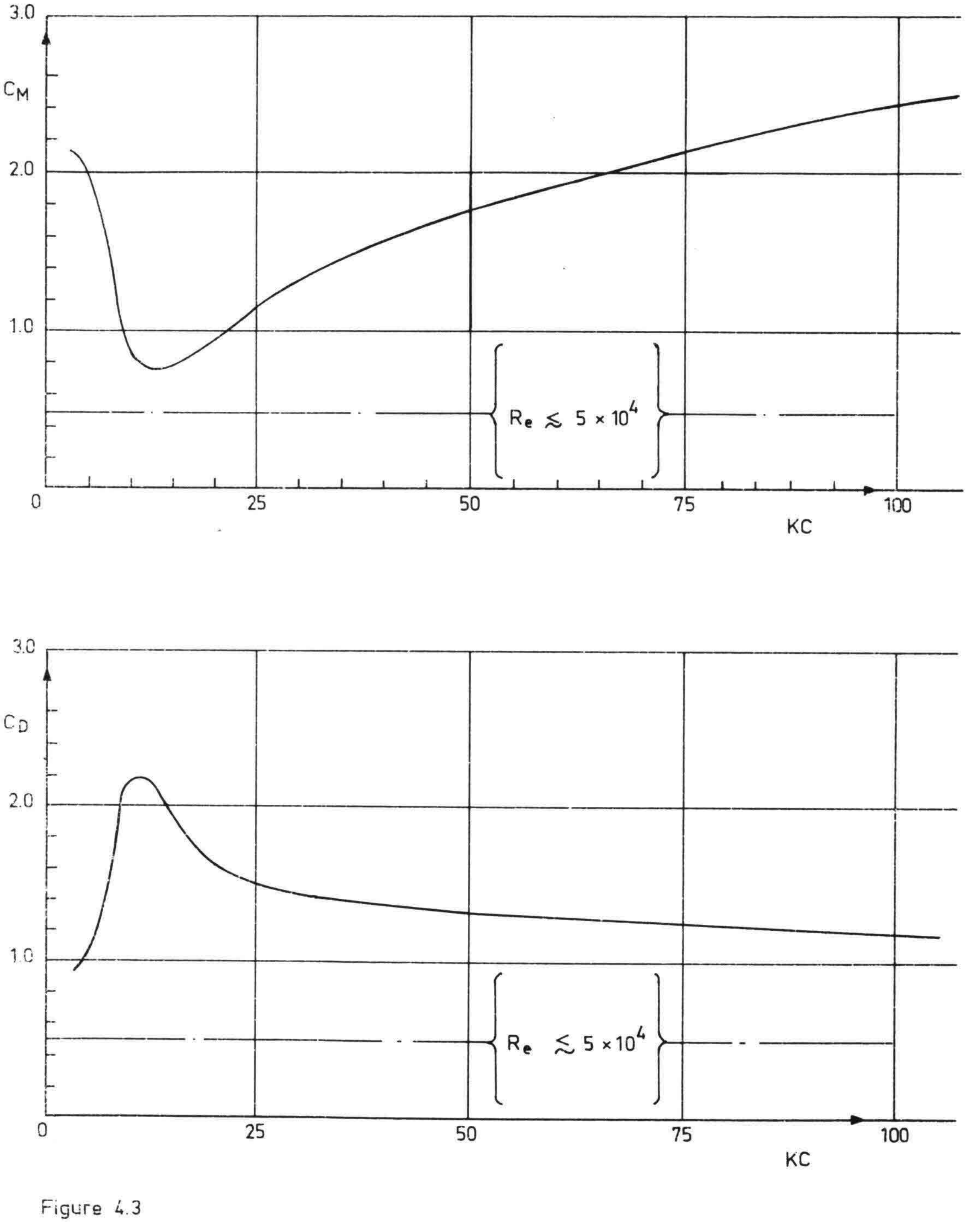
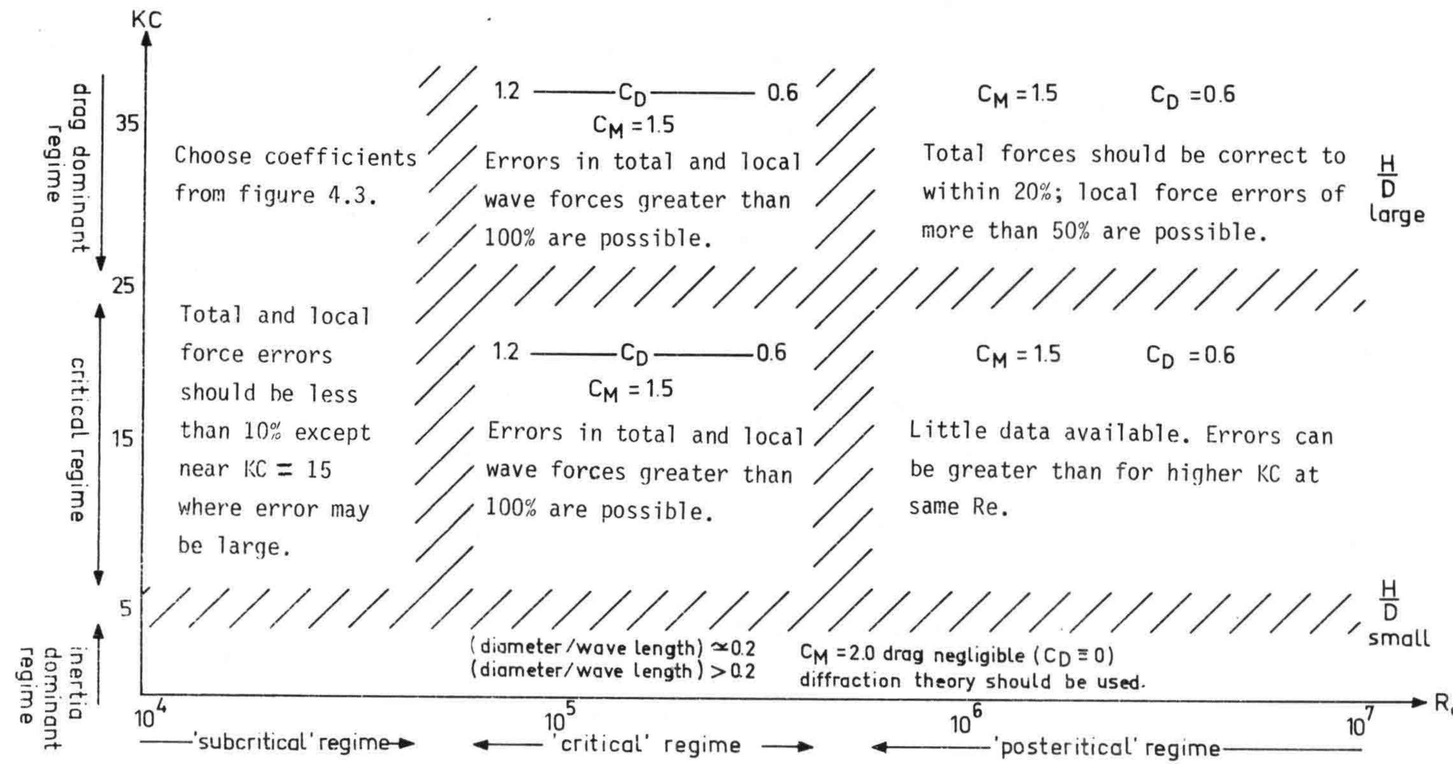


Figure 4.3
SUGGESTED VALUES OF C_M AND C_D AT SUBCRITICAL R_e FROM KEULEGAN AND CARPENTER, FOR THE WAVE FORCE NORMAL TO THE AXIS OF A SMOOTH CYLINDER.



Data above are for smooth isolated cylinders in deep water.

This figure adapted from Anon. (1976)

Figure 4.2
MORISON COEFFICIENTS AS FUNCTION OF R_e AND KC

Note that the non-linear character of the drag force makes it *incorrect* to determine the drag forces from the wave and constant current separately and then to add these two force components. In the more correct method outlined above, the velocity components are first added as vectors before the resulting drag force is computed.

4.6 Simplifications

Under certain conditions, the Morison equation (4.04) can be simplified. Since the non-linear drag term is the most troublesome, it is helpful to investigate the conditions under which this can be simplified.

If the drag force component is small relative to the inertia force, then the drag force term in (4.04) can be either neglected or approximated by a linear relationship. Remembering, that the ratio of drag force to inertia force is represented by the Keulegan-Carpenter Number, we can see that KC must be small if the drag force is to play an unimportant role in our problem. From equations 4.07 and 4.06 we see that the drag force component is less important when velocity or wave period is small or when the cylinder diameter is large. In general, the drag force term can be neglected without significant error whenever the Keulegan-Carpenter Number is less than about 3. Such low KC values occur often with large floating bodies or when more slender bodies are subjected to very short period movement relative to the water. This last case can be experienced when an offshore structure is subjected to an earthquake , for example.

For somewhat larger but still small KC values, the drag force term can be approximated by expressing $V|V|$ as a Fourier Series and then retaining only the first harmonic. If the velocity can be written as:

$$V = a \sin \omega t \tag{4.09}$$

then $V|V|$ yields a Fourier Series without a constant term and with exclusively odd harmonics of $\sin \omega t$. The first term has amplitude:

$$\frac{8a^2}{3\pi} = 0.8488\ a^2 \tag{4.10}$$

This means that $V|V|$ can be approximated by:

$$\frac{8a}{3\pi} V \tag{4.11}$$

Note that the peak value of the drag force will be somewhat reduced in the linearized approximation. The importance of this remark will become apparent in chapter 5.

If, on the other hand, the Keulegan-Carpenter Number is very large, the inertia force component becomes relatively unimportant. Such is the case, for example, for a cylinder in a tidal current for which the period is relatively very long. Steady current data can be used with success.

4.7 Additional Remarks

As we have seen in chapter 2, velocity components under a wave decrease as we move deeper into the ocean. The most straightforward practice is to use the computed values of velocity, etc. at each depth to determine the necessary parameters such as Re and KC which in turn determine the values of C_D and C_M to use at that depth. This is the most popular but not universal practice.

An alternative but apparently less correct approach is to evaluate the flow parameters and coefficients at the ocean surface and use these coefficients as constants valid over the entire depth.

Figure 4.2 also gives some indication of the uncertainty involved in the computation of wave forces. Note that the uncertainties are greatest for individual structural elements and when the drag force component is relatively more important.

All of this discussion until now has been concerned with a smooth cylinder. In reality, marine growth soon makes the members of offshore structures rough and even larger. Examples of offshore structural elements whose diameter have been doubled by marine growth are not hard to find. Often, larger diameters are substituted into the Morison Equation (4.04) when computing forces. Additionally, the roughness tends to increase the drag coefficient, C_D, somewhat. Minimum C_D values of about 0.8 to 1.0 can now be expected. Even slight roughness can often double C_D values.

4.8 Example

Since it can be instructive to illustrate a wave force computation, let us compute the hydrodynamic force on a 10 m long element of a structure. The diameter of the element is 2.5 m and it is placed in a vertical position and extends from 95 m below the still water level to 105 m below this level.

The design wave has a height of 20 meters and a period of 15 seconds. A current of 0.5 m/s flows in the same direction as the waves are propagated. Determine the maximum force acting on this portion of the structure.

We first determine the relevant flow parameters at the location of the element. Using equation 2.08:

$$\hat{u} = \frac{\omega H}{2} e^{kz} \tag{4.12}$$

$$= \frac{2\pi}{T} \frac{H}{2} e^{\frac{2\pi}{\lambda} z}$$

$$= \frac{(2)(\pi)}{15} \frac{20}{2} e^{\frac{(2)(\pi)(-100)}{(1.56)(15)^2}}$$

$$= \frac{4}{3} \pi e^{-1.790} = 0.70 \text{ m/s} \tag{4.12}$$

where (2.05) has been used in the third line, and we have assumed conditions at z = -100 m to be typical.

Since the constant current acts in the same line as $\hat{u}$, we can add it directly. The maximum water velocity will then be:

$$0.70 + 0.50 = 1.20 \text{ m/s} \tag{4.13}$$

The Keulegan-Carpenter Number is, now:

$$KC = \frac{U}{D} = \frac{(1.20)(15)}{2.5} = 7.2 \tag{4.14}$$

This implies that both drag and inertia will be important with the latter term dominating slightly.

Checking the Reynolds Number:

$$Re = \frac{uD}{\nu} = \frac{(1.20)(2.5)}{10^{-6}} = 3 \times 10^6 \tag{4.15}$$

we see that this is in the postcritical area.

This allows us to determine C_D and C_M from figure 4.2:

$$C_D = 0.6 \qquad C_M = 1.5 \tag{4.16}$$

The total velocity at the location of our element now varies about the constant current velocity. The maximum velocity - equation 4.13 - is 1.20 m/s; the minimum velocity is:

$$0.5 - 0.70 = -0.20 \text{ m/s} \tag{4.17}$$

or expressing the total velocity, V, as a function of time:

$$V = 0.50 + 0.70 \sin(\frac{2\pi}{15}t) \tag{4.18}$$

The acceleration follows from differentiation:

$$\frac{dV}{dt} = (0.70)(\frac{2\pi}{15}) \cos(\frac{2\pi}{15}t) = 0.29 \cos(\frac{2\pi}{15}t) \tag{4.19}$$

Now, using (4.01) for a 10 m length of cylinder:

$$F_D = (\tfrac{1}{2})(1025)(0.6)(2.5)(10) \left[0.5+0.7 \sin(\omega t)\right] \cdot \left[\left|0.5+0.7 \sin(\omega t)\right|\right] \tag{4.20}$$

$$= 7688 \left[0.5+0.7 \sin(\omega t)\right] \left[\left| 0.5+0.7 \sin(\omega t)\right|\right] \tag{4.21}$$

Also, using (4.03)

$$F_I = \tfrac{1}{4}\pi\ (2.5)^2(10)(1025)(1.5)(0.29)\ \cos(\omega t) \qquad (4.22)$$

$$= 21887\ \cos(\omega t) \qquad (4.23)$$

where $\omega = 2\pi/15$.
Our suspicion about the dominance of the inertia force seems confirmed.

One can see by inspection that the maximum sum of F_D and F_I will occur in the interval during which both terms have the same sign. Choosing the positive interval (which will yield the maximum force in this case), then:

$$F = F_D + F_I$$

$$= 7688\ \left[0.5+0.7\ \sin(\omega t)\right]^2 + 21887\ \cos(\omega t) \qquad (4.24)$$

This is maximum when $\frac{dF}{d(\omega t)} = 0$

Thus, at the maximum:

$$(2)(7688)\ \left[0.5+0.7\ \sin(\omega t)\right]\ \left[0.7\ \cos(\omega t)\right]\ -\ 21887\ \sin(\omega t)=0 \qquad (4.25)$$

or:

$$7688 + 7534\ \sin(\omega t)\ \cos(\omega t) - 21887\ \sin(\omega t) = 0 \qquad (4.26)$$

$$\sin(\omega t) = \frac{7688 + 7534\ \sin(\omega t)\ \cos(\omega t)}{21887} \qquad (4.27)$$

a trial and error solution yields:

$$\omega t = 30^\circ \qquad (4.28)$$

Thus using (4.24):

$$F = 5558 + 18950 = 24508\ \text{N}. \qquad (4.29)$$

which is our desired answer.

One might like to attack the same problem, but now with the cylinder placed horizontally parallel to thw wave crests at a depth of 100 m. What will be the maximum horizontal force acting on this cylinder segment?

The answer is: 24528. N (4.30)

which by chance is not much different than the answer to the first problem.

5. DESIGN WAVE CHOICE

5.1 Introduction

In the previous chapter, we have seen how to predict the hydrodynamic forces on an element of an offshore structure once the wave conditions are known. The wave conditions which we need are really a wave height, H, and a wave period, T. If we are not working in deep water, the we will need to know the water depth as well; we shall restrict ourselves to deep water, however.

A quick review of chapter 2 reminds us that the actual sea can be described best as a random sort of combination of a large number of small wave components - a spectrum. If the relationship between wave height (for example) and hydrodynamic force were linear (it is not, because of the quadratic drag force), we could transfer the know wave spectrum to a loading spectrum. One method to determine loadings is, thus, to transform a wave spectrum using a linearized transfer function.

Another approach which avoids the approximations involved in the linearization, above, is to choose a single - or at most a small number of - design wave and to compute design loadings based upon this design wave. Now, the wave information can be transformed to the wave force via the classical Morison Equation.

In the following sections, we examine each of the two above methods before comparing them.

5.2 Design Wave Method

We may remember that the wave heights within a storm can be characterized by the significant wave height and described by the Rayleigh Distribution - see section 2.5. Further, the storm history of a given area can be described by a semi-logarithmic plot of significant wave height versus frequency of exceedance.

The Rayleigh Distribution was given in chapter 2 as:

$$P(H) = e^{-2\left(\frac{H}{H_{sig}}\right)^2} \qquad (2.16) \qquad (5.01)$$

where P(H) is the chance that an individual wave of height H is exceeded in a storm characterized by H_{sig}. The data represented by table 2.2 giving the storm statistics of the southern North Sea can be plotted on semi-logarithmic paper or the following equation can be fitted:

$$f(H_{sig}) = 10^{3.786-0.669\ H_{sig}} \qquad (5.02)$$

where $f(H_{sig})$ is a frequency in storms per year. It would be safest, of course, to design our structure to with stand the *maximum* wave load that is possible. This would imply that P(H) in (5.01) would be zero; This, in turn, implies that H would be infinite; thus, the maximum wave force is infinite. It is, of course, impossible to design a structure to withstand an infinite load. We must be content, then, to accept some finite chance that a design load (wave) will be exceeded during the lifetime of the structure.

Ideally, we would choose a chance of exceedance and, from that, compute a design wave height, H_d. Unfortunately, this problem cannot be solved; we must be content to determine the chance that a given design wave height, H_d, is exceeded one or more times in the design life of our structure. The procedure for this problem is described in more detail by Bijker and Paape in Massie (ed) (1976).

Storms at sea do not last forever*. The total number of waves encountered in the storm is, thus, finite. This number of waves, N, will depend upon the time period over which the storm is assumed to rage - usually about 6 hours for the North Sea - and upon the average wave period in the storm. Usually, either N or the average wave period is included in wave statistics.

Let us consider first a single storm characterized by some value of H_{sig}. This storm will contain N waves. Further, we wish to determine the chance that a chosen design wave height, H_d, is exceeded at least once.

Using (5.01), the chance that H_d *is exceeded* by an single wave is:

$$P(H_d) = e^{-2\left(\frac{H_d}{H_{sig}}\right)^2} \tag{5.03}$$

The chance that this wave *is not exceeded* is, then:

$$1 - P(H_d) \tag{5.04}$$

The chance that this wave is not exceeded in a series of N waves is, then:

$$[\ 1 - P(H_d)\ ]^N \tag{5.05}$$

and finally, the chance that the design wave height, H_d, is exceeded at least once in the single storm containing N waves is:

$$E_1 = 1 - [1 - P(H_d)]^N \tag{5.06}$$

Since the wave of height H_d can occur in many different storms. We must now couple E_1, found above, to the storm statistics data. If we knew the chance that H_{sig} used to compute E_1 occurred, we could compute the chance that *both* the storm characterized by H_{sig} occurs *and* H_d occurs in that storm. Unfortunately, equation 5.02 gives the chance that H_{sig} is exceeded rather than occurs.

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* It may well seem so, however, if you happen to be seasick on board a ship in the storm!

However, the chance that H_{sig} falls in an interval between H_{sig1} and H_{sig2}:

$$p(H_{sig}) = p(H_{sig1} \leqslant H_{sig} \leqslant H_{sig2}) \qquad (5.07)$$

is equal to:

$$p(H_{sig}) = f(H_{sig1}) - f(H_{sig2}) \qquad (5.08)$$

H_{sig} on the left of the two above relations is a value of H_{sig} used to characterize the wave height interval between H_{sig1} and H_{sig2}. Assuming that all of the storms in the interval can be approximately characterized by H_{sig}, the change that *both* the given *storm occurs* and the design wave *is exceeded in that storm* is, somply:

$$E_2 = p(H_{sig}) \cdot E_1 \qquad (5.09)$$

We are not yet done, however, since the design wave can also occur in another storm outside the interval characterized by our chosen H_{sig}. Therefore, we must carry out a computation outlined above for a whole series of values of H_{sig}, each characterizing a different interval of the total storm record. H_d will, of course, remain constant, but values of N and $p(H_{sig})$ will vary. If we use N' values of H_{sig} to characterize the total range of storm conditions then the N' resulting values of E_2 must be combined.

Since each value of E_{2i} for i = 1 to N' represents the chance that the design wave *is exceeded* in a given storm and the storms are mutually exclusive (only one storm is raging at any one time), then the chance, E_3, that H_d *is not exceeded* at any time (in any storm) during the one year is:

$$E_3 = 1 - (E_{21} + E_{22} + \ldots + E_{2i} + \ldots E_{2N'}) \qquad (5.10)$$

$$= 1 - \sum_{i=1}^{N'} E_{2i} \qquad (5.11)$$

where $\sum_{i=1}^{N'}$ is the sum of the N' terms.

If the structure has a lifetime of ℓ years, then the chance that the design wave, H_d, will be exceeded at least once during the lifetime of the structure is:

$$P(H > H_d) = 1 - E_3^{\ell} \qquad (5.12)$$

This resulting chance is our objective! By repeating this whole computation for various values of H_d, we can determine the relationship between H_d and the chance that this wave will be exceeded.

5.3 Example

Compute the chance that a design wave height of 20 meters occurs at least once in a period of 25 years in the southern North Sea.

Table 5.1 shows the data and computations involved. The characterizing values of H_{sig} (col. 4) are first chosen. The values in column 1 representing the limits of the intervals are then chosen. Values of $f(H_{sig})$ follow from equation 5.02 which has been fitted to data for the southern North Sea. $p(H_{sig})$ follows by subtracting adjacent values in column 2 of table 5.1.

$P(H_d)$ comes from substitution of values of H_{sig} (col. 4) and H_d = 20 m into (5.03). Values of E_1 then follow using equation 5.06.

Values of E_2 are found by multiplying values found in columns 3 and 7 of the table - equation 5.09. E_3 is found using (5.11) for the N' = 11 intervals. Notice that the values of E_{2i} are maximum near the middle of the table. At the top of the table, E_2 values are small because the chance that the storm occurs, $p(H_{sig})$ is small. On the other hand, at the bottom of the table, the chance that a 20 m wave occurs in a given (mild) storm is extremely small.

The final result of the computation is that a wave 20 m high has a chance of about 3.75% of being encountered in a period of 25 years on the southern North Sea.

5.4 Wave Period Choice

In order to calculate velocities and accelerations in a wave we need to know the wave period (frequency) as well as the wave height. What wave period should we combine with the design wave height in order to determine velocities and accelerations near our structure?

Sometimes the wave statistical data available includes separate wave period data. This can be helpful in determining the design wave period; a significant bit of "engineering judgement" will be needed, however.

Another, but extremely conservative, alternative will be to assume that the design wave is nearly breaking. In section 2.3 the limiting condition for breaking was indicated as:

$$\frac{H_d}{\lambda_d} = \frac{1}{7} \qquad (5.13)$$

In our example problem, this means that λ_d is at least:

$$\lambda_d \geqslant (20)(7) = 140 \text{ m} \qquad (5.14)$$

and, using (2.05)

$$T_d \geqslant 9.47 \text{ s} \qquad (5.15)$$

say, T_d = 10 s.

Table 5.1 Design Wave Height Probability Computations H_d = 20 m.

H_{sig} (m)	$f(H_{sig})$ ($\frac{storms}{year}$)	$P(H_{sig})$ ($\frac{storms}{year}$)	Char. H_{sig} (m)	N ($\frac{waves}{storm}$)	$P(H_d)$ -	E_1 -	E_{2i} -
∞	0.00						
		5.7×10^{-6}	14	400	0.0169	0.9989	5.69×10^{-6}
13.5	5.7×10^{-6}						
		2.08×10^{-5}	13	500	0.0088	0.9879	2.05×10^{-5}
12.5	2.65×10^{-5}						
		9.72×10^{-5}	12	600	0.0039	0.9021	8.77×10^{-5}
11.5	1.24×10^{-4}						
		4.54×10^{-4}	11	700	0.0013	0.6101	2.77×10^{-4}
10.5	5.77×10^{-4}						
		2.12×10^{-3}	10	800	3.35×10^{-4}	0.2354	4.99×10^{-4}
9.5	2.69×10^{-3}						
		0.0099	9	900	5.14×10^{-5}	0.0452	4.47×10^{-4}
8.5	0.0126						
		0.0461	8	1000	3.73×10^{-6}	3.72×10^{-3}	1.71×10^{-4}
7.5	0.0587						
		0.2151	7	1000	8.12×10^{-8}	8.12×10^{-5}	1.75×10^{-5}
6.5	0.2738						
		1.004	6	2000	2.23×10^{-10}	4.0×10^{-7}	4.02×10^{-7}
5.5	1.28						
		4.68	5	2500	1.27×10^{-14}	0	0
4.5	5.96						
		21.87	4	3000	≈ 0	0	0
3.5	27.83						

$E_3 = 0.9985$

$P(H > H_d) = 0.0375 = 3.75\%$

Our chosen wave, therefore, has the following properties:

Height	20 m	(chosen)
Chance of Occurrance	3.75%	(computed)
Period	10 s	(semi-computed)

This wave would then be used in the Morison Equation to determine the design loads.

5.5 Spectrum Transformation Method

A second (and independent) approach to the problem of determining the loads on a structure is to transform the spectrum of waves (such as shown in figure 2.3) to a spectrum of wave forces. This can be done only if the Morison Equation is expressed in linearized form. Only then are two necessary conditions satisfied:

a. The wave force is directly proportional to the wave height, and
b. The frequency of the wave force is the same as the frequency of the wave.

The transfer function used to determine the spectrum of wave forces from the wave spectrum can be determined as a function of frequency simply by determining the wave force exerted on the desired element as a function of frequency using a linearized Morison Equation for a constant wave height of 1 meter.

The wave spectrum is then transformed to a force spectrum simply by multiplying the wave height spectrum value for a given frequency by the transfer function value. The resulting force spectrum can then be used in further design analysis.

5.6 Comparison of The Methods

The two methods of determining the design loads on an offshore structure just presented in the previous sections are not, in general, equivalent. Only when the drag force plays an insignificant role in the total force on a structural element will there be agreement between the methods. It might be better to say that the results from the two methods would not, then, be in conflict; after all, the two methods do yield rather different information. Even so, however, some comparison is possible.

Consider, for example, that we have a record of waves measured during some period at sea. We could determine the spectrum from this record and determine a transfer function for wave height to wave force as mentioned in the previous section.

An alternate procedure is to determine the wave height and associated wave period data from the wave record needed to use the Morison Equation directly. Such a procedure would yield a sort of record of wave force versus time which could be caused by the given wave record. Of course, we can then easily determine the spectrum of the force - time record. The important question is: "How do the two resulting force spectra compare?" The steps outlined above are shown schematically in figure 5.1.

Linnekamp (1976) carried out such a comparison. (It involves a lot of work!) He found that the spectrum transformation method agreed well with the more complicated Morison Equation approach for forces smaller than about the "Significant Wave Force" - the value exceeded by about 13.5% of the force peaks in the record.

For the more extreme peak loadings, however, he found that the spectrum transformation method yielded force values which were too low when compared to the design wave method.

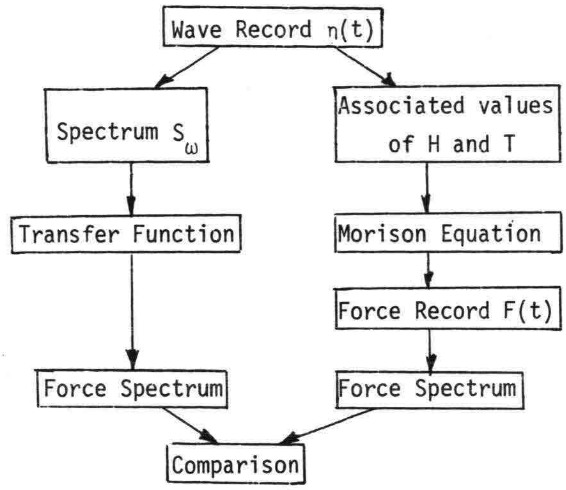

Figure 5.1 Representation of Alternate Methods

Therefore, we might make the following conclusions:

a. When the drag force plays no significant role in the force determination (Keulegan-Carpenter Number < 3) both methods are identical. The spectrum transformation method is then the better one because of its simplicity.

b. When the drag us important and we are interested in *maximum* forces which seldom occur (These could lead, for example, to a total failure of the structure.) we must use a design wave approach in order to get an accurate force prediction.

c. If, on the other hand, we are interested in loads which occur many times, (needed for material fatique studies for example) then a spectrum method appears to yield adequate results.

REFERENCES

The following list includes more complete bibliographic data on most (and hopefully all) of the references listed in the text.

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CORRECTIONS AND ADDITIONS TO

HYDRODYNAMIC ASPECTS OF FIXED OFFSHORE STRUCTURES

compiled by W.W. Massie

Coastal Engineering Group
Department of Civil Engineering
Work Group Offshore Technology
Delft University of Technology
Delft
The Netherlands

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Corrections and additions to Hydrodynamic Aspects of Fixed Offshore Structures - revised edition April 1979 by W.W. Massie

Page	Correction or remark
i	This table of contents is no longer complete.
1	Replace chapter 1 with the revised text included here
2	Replace with chapter A, "Sediment Transport at Sea", included here.
3-7	This information will be presented in a different sequence in class
8-9	Add extra note near the bottom of page 8 and replace table 2.2. See page included here
13	Add item (included) on Lift Forces at bottom of the page
22	Correct equation 4.14: $\ldots = \frac{\hat{u}T}{D} = \ldots$
23	Correct equation 4.24: $\ldots = 7688 \left[0.5 + 0.7\sin(\omega t)\right]^2 \ldots$ correct equation 4.25: $\ldots (7688) \left\|0.5 + 0.7\sin(\omega t)\right\| 0.7 \ldots$
24	Replace with text on Wind Loads (included)
25	Delete equation 5.02 and the sentence including ~~them~~ this equation
26	Line 9 of text, add: as corrected in May 1979 second line above footnote, change "equation 5.02" to: table 2.2
27	Correct equation 5.07: $\ldots = p(H_{sig1} \ldots$ correct equation 5.08: $\ldots = P(H_{sig1}) - P(H_{sig2})$ correct the 6th line after equation 5.09: $\ldots N$ and $p(H_{sig}) \ldots$ replace the two lines above equation 5.10 and equations 5.10 and 5.11 by: the chance, E_3, that H_d *is exceeded* at least once in the storm period is: $E_3 = E_{21} + E_{22} + \ldots + E_{2N'}$ (5.10) $= \sum_{i=1}^{N'} E_{2i}$ (5.11)

27 replace the last 7 lines (including eq. 5.12) with:

The chance that this wave height is not exceeded in the single storm period is, of course,

$$1-E_3 \qquad (5.11a)$$

and the chance that H'_d is not exceeded in a structure lifetime of ℓ years containing $M\ell$ storms is:

$$(1-E_3)^{M\ell} \qquad (5.11b)$$

Finally, the chance the H_d is exceeded at least once during the lifetime is:

$$P'(H_d) = 1 - (1-E_3)^{M\ell} \qquad (5.12)$$

28 - change 6th line of section 5.3 text to read:

Values of $P(H_{sig})$ follow from table 2.2 representing

- Drop first word in the following line.
- In the 13th line: ... N' = 13 ...

29 Replace entire page with new table and text (included).

33 Add the following reference:

Yamamoto, T.; Koning, H.L.; Sellmeijer, H.; Hijum, E. (1978): On the Response of a Poro-Elastic Bed to Water Waves: *Journal of Fluid Mechanics*: vol. 87, pp. 193-206.

1 Introduction

These brief notes are intended to provide the student of general offshore engineering some insight in the civil hydraulic engineering aspects of offshore engineering problems. Because of the limited nature of this class and the varied student background, coverage here will be summary with emphasis on problem understanding rather than theoretical details. More extensive data on many of the topics can be found in the references listed.

The subject material in this portion does not stand alone. Use will be made here of information presented in the classes on Physical Oceanography and in the review of Fluid Mechanics. Further a certain coupling with the Soil Mechanics aspects is also presented. The treatment of waves reviews the Physical Oceanography of surface waves and works toward both Civil Engineering and Naval Architecture (seakeeping) applications.

The form of these notes may seem a bit cumbersome. This results from the constantly changing list of topics to be treated and rapid developments in this specific branch.

These notes should be read using these pages as a guide. The order of presentation will be effectively that that results from making the corrections and additions listed here in the original book.

Because of its special significance for the offshore industry, the statistics of design conditions - here applied to ocean waves - will be handled somewhat more deeply than other topics, see chapter 5.

A. Sediment Transport at Sea

A.1. Introduction

Sea bottom material - sand or finer soil particles - is often transported along with ocean currents driven, for example, by tides or waves. Consideration of the equilibrium of a single bed material particle resting on the sea bed shows that friction between the particle and the rest of the bed holds it in position until the driving force (from the water flowing above it) exceeds a certain critical value. Since this driving force (friction between the moving water and the bed) is proportional to the square of the current velocity, it appears that the current velocity must exceed a certain critical velocity before bed material will be brought into suspension and movement will be initiated. Waves, causing in principle only an oscillatory water motion near the saa bed, can also bring material into suspension, but the lack of a resultant water movement prevents transport. The various possible combinations of waves and currents can be evaluated as to sediment transport via the following two questions:

Is the maximum velocity ever greater than the critical value? and Is there a resultant water movement? Only if both answers are yes will there be an actual bed material transport.

The presence of bed material transport is not necessarily detrimental to the sea bed or to a structure. Indeed, as long as just as much bed material is transported into a given region as is transported out of that region the bed remains in a state of dynamic equilibrium; no net deposition or erosion takes place. Erosion results only if the sediment transport capacity increases from one place to another; deposition results from a derease.

A.2. Influence of Structures

Given a stable sea bed which is in equilibrium, what is now the effect of placing an obstacle such as a pipeline or offshore structure on the sea bed? These will be discussed individually beginning with a large circular offshore gravity structure places on a sandy bottom.

Figure A.1 shows a plan view of the sea near the gravity structure. Some streamlines are sketched qualitatively. Remembering from fluid mechanics that continuity requires that all of the water flowing in section A (between two streamlines) also passes through B and C and since the stream path is narrower at B, the velocity there will be higher than at A. Since the velocity is higher, so, also, will be the sediment transport capacity at B as compared to at A and C - provided, of course, that the velocity at B is greater than the critical value.

Application of that which we know about erosion - from above - to this problem leads to only one conclusion: If sediment moves at all, there will be erosion of material between A and B and deposition of that same material between B acn C. In practice, the bed changes will be more or less concentrated near B where a hole will develop; the deposition will be spread over a wider area more downstream. The deepest part of the hole will be near the structure.

The effects of this scour hole on the foundation will be discussed more fully in the classes on Foundation Engineering.

The discussion above can be repeated for the leg of a jacket structure. Because such a leg is of much smaller diameter than a gravity structure the scour hole will also be smaller but probably of larger relative size.

What can be done to prevent such scour? One solution is to cover the sea bed near the structure with a protective layer of less erodable material - coarse gravel or stone, for example. Problems can still arrise, but now at the edges of the protective layer. This will become more obnous a bit later in this chapter.

A.3. Exposed Pipelines

Consider, now, a pipeline laying on the bottom. Any realistic person will not even dare to hope that this pipe will be in contact with the bottom over its entire length, some parts will have settled in to the bottom, other sections will form a free span just above the bottom.

The presence of the bottom will have a marked influence on the flow pattern near the pipe. The exact nature of the influnece will depend upon many factors: the pipe and bottom roughness, the pipe diameter, and the original clearance between the pipe and the bottom. In general, the following can be expected.

The pipe obstruction will cause a local increase in the average velocity between the pipe and the bottom; this, in turn, erodes the bottom material so that a trench forms under the pipe. If the pipe elevation is fixed, this trench deepens until the sediment transport capacity remains constant along the entire stream path extending from well upstream of the pipe to well downstream of it. What happens to the eroded material?

Just as with the offshore structure, this material is deposited downstream from the pipeline.

One's initial reaction to the fact that the erosion under a pipeline section reaches an equilibrium state might well be one of indifference. "A section of pipeline gets a hole under it. So what?" Examination of a lengthwise profile - figure A2 - reveals two difficulties.

1. Erosion will continue near the end-points of the span making the span perpetually longer, and

2. The pipeline will inevitably sag between the supports, decreasing the bottom clearance and stimulating additional erosion.

Obviously, neither process* can go on forever. If the span becomes long enough, the pipe will break of its own weight while additional erosion deepening under a pipeline often halts after the pipe itself has largely sagged into its erosion trench.

One solution to this whold problem could be to bury the entire pipeline; this solution is examined in the following section.

A.4. Pipeline Burial

One obvious-looking solution to erosion problems near pipelines is to bury the pipeline completely from the start. Three individual sub-problems arrise with this approach, however; these are discussed separately in the remainder of this section.

Pipeline Trenching

The first step in getting a pipeline underground on the sea bed is to make a trench in which to lay it. In shallow water (less than, say 30 m) rather neat trenches can be dredged using rather conventional equipment. In deeper water dredging becomes first inaccurate - resulting in a wide (expensive!) trench - and later impossible.

In water too deep for dredging it has been proposed to first lay the pipe on the sea bed and then to liquify the soil under the pipe using a jetting machine towed along the pipeline. Such a machine also has its practical problems. The pipeline cannot be bent too sharply during the process of sinking it in the bed and some soild (stiff clays for example)

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* Additional, equally troublesome, processes will be considered after hydrodynamic forces have been treated.

are unsuited to such jetting operations. A more recent Dutch development has been a specially designed plow which, guided by a pipe laying on the bottom, is towed by a tugboat. The problem with pipe bending is still present but the plow can be used in more varied soild. A disadvantage of the plow is that it leaves the pipe laying in an open vee-shaped trench. Also, the depth of the trench (size of the plow) is somewhat limited by the available tugboat pulling force. Indeed, such plows have only been designed since the introduction of the 'super tugs' such as the 'Smit Rotterdam'.

Pipeline covering

Once a pipeline is laying in a trench the next problem is to cover it. (Under certain conditions, we may wish to cover a pipeline laying on the sea bed surface.) Generally new soil material will have to be brought into accomplish this covering. What are the problems associated with the placement of this covering?

First, we must be sure that the material actually lands and comes to rest where it is wanted. This may sound silly, but sometimes carefully dumped material seems to disappear without a trace! One way to be more sure of the placement is to convey the cover material to a point just above the pipeline by dumping it through a vertical pipe.

Secondly, we must be sure that the pipe remains in place. A mixture of soil particles and water in which the soil particles are loose from one another behaves as a liquid with the specific weight of the mixture. If this specific weight is greater than the net specific weight of the (empty!) pipeline, the latter will float on the mixture. The result is a neatly filled trench with the pipeline resting on top of the backfill material!

Stability of Covering

The third problem is to be sure that the covering material and the surroundings remain in place. This relates back to our knowledge of sediment transport. Stability implies that no erosion or deposition is to take place anywhere in the vicinity. This, in turn, requires that the sediment transport remain constant along a streamline passing over the disturbed bed area.

The pipeline covering material must, therefore, have effectively the same characteristics as the surrounding undisturbed bed. Not only must the fill material remain in place, it must also not cause a flow disturbance which could lead to erosion downstream from the pipe covering material.

A.5. Influence of Waves

It has been assumed for simplicity that the erosion and deposition phenomona described so far in this chapter have been caused by steady unidirectional currents. What happens if wind generated water surface waves are also present? (The discussion here will be kept qualitative since exact data on wave action is presented only later.).

If the water is not extremely deep (greater than, say 100 to 200 m) the surface waves will cause water pressure variations and water movement near the sea bed. Both of these phenomona are cyclic with period equal to the wave period.

The oscillatory water velocity near the sea bed caused by the waves can be added vectorially to the constant current. Bottom material will be brought into suspension whenever this resulting time-dependent current velocity is greater than the critical velocity as explained earlier in this chapter. It is even possible that a current which alone would cause no bottom material movement combined with an equally weak wave action will cause, together, a resulting bed material transport.

The influence of the water pressure fluctuations is more subtle and has only come to attention quite recently - Yamamoto et al (1978). The pressure fluctuations in the sea near the sea bed are propagated down into the pore water in the sea bed. These pore water pressure fluctuations can cause loss of stability of the soil mass for short periods of time (less than one surface wave period). Thus, a buried pipeline, for example, can find itself surrounded by 'high density liquid' for repeated short intervals so that pipe movements within the sea bed become possible.

More of the soil mechanics background of this phenomona is given in the classes of Prof. van Weele.

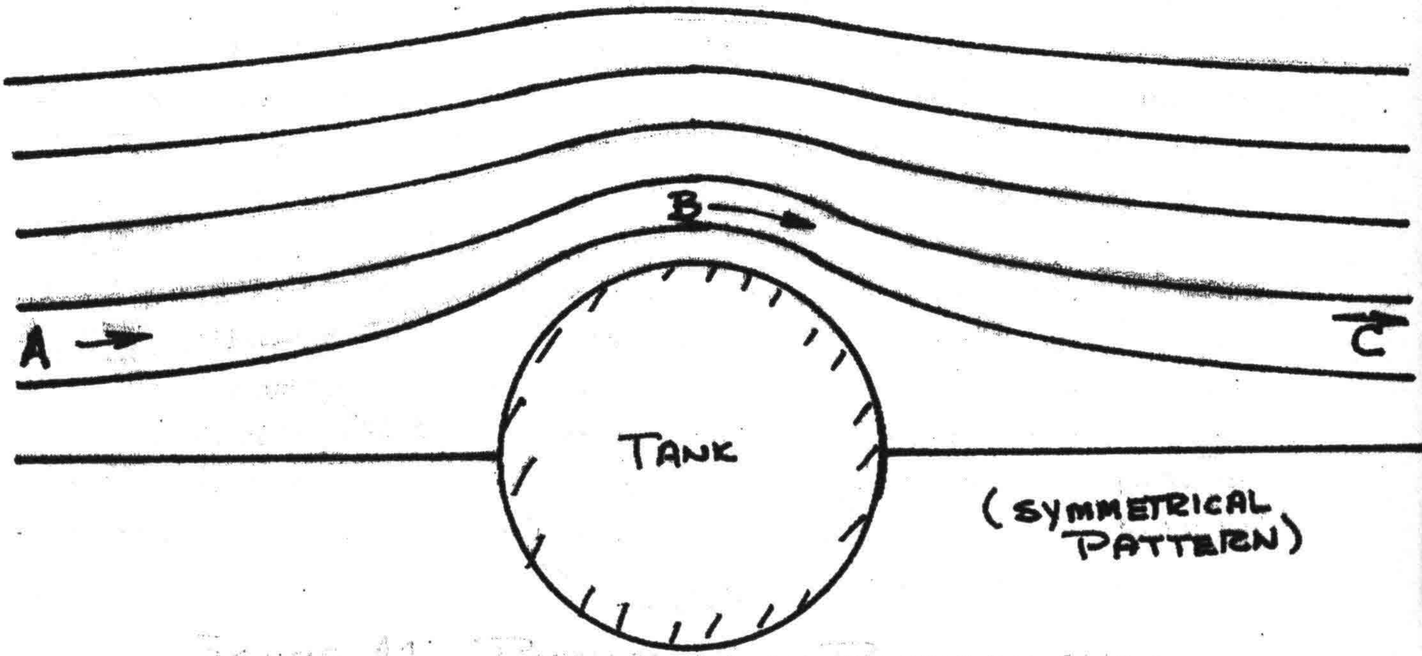

FIGURE A1 PARTIAL FLOW PATTERN AROUND CYLINDER IN CURRENT

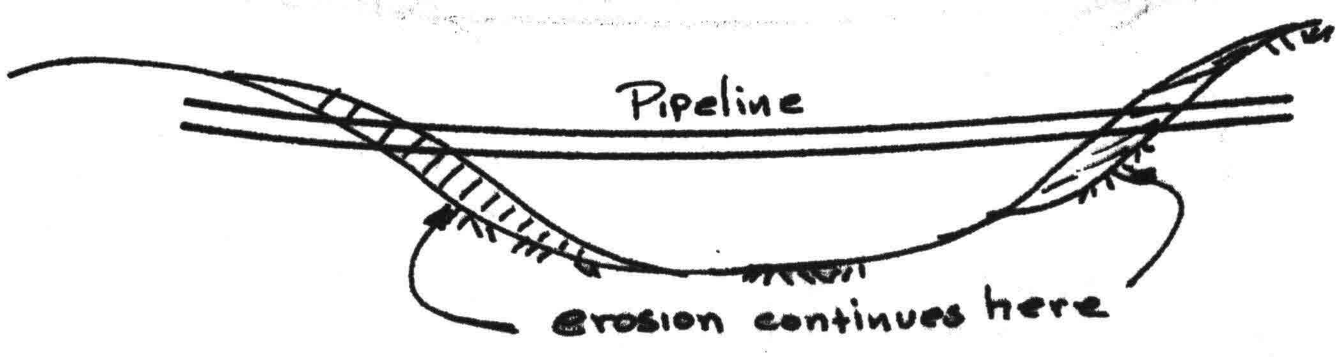

FIGURE A2 Pipeline Profile

Corrections on pages 8 and 9

Add the following text at the end of 7th line above the footnote:

In that table, two frequencies are given. The first is an absolute frequency, $P(H_{sig})$ - the chance that a storm at least as severe as is indicated by the adjacent H_{sig} value is occurring at any arbitrary instant. This frequency, just as any true probability, is always less than 1.

The second frequency listed, $F(H_{sig})$ is no longer a true statistical frequency but indicates approximately how many storms will exceed the given intensity per year.

Replace the data in table 2.2 with:

Significant Wave Height, H_{sig} (m)	Number of waves per storm, N (-)	Frequency of $P(H_{sig})$ (-)	Exceedance $F(H_{sig})$ (storms/year)
13.5	400	2.283×10^{-8}	1/100000
12.5	500	2.283×10^{-8}	1/30000
11.5	550	1.070×10^{-7}	1/6400
10.5	600	4.556×10^{-7}	1/1500
9.5	600	1.957×10^{-6}	1/350
8.5	800	8.252×10^{-6}	1/83
7.5	1000	3.805×10^{-5}	1/18
6.5	1000	1.802×10^{-4}	1/3.8
5.5	2000	8.699×10^{-4}	1.27
4.5	2000	3.808×10^{-3}	5.56
3.5	2500	1.803×10^{-2}	26.32
2.5	3000	8.082×10^{-2}	118
≃ 0	3500	1	1460

Addendum Page 13 - lift forces

Vibrations play a significant role in the development of important lift forces. A rigid cylinder will experience vortex shedding which is more or less randomly distributed along the cylinder length; the resulting integrated lift force will be small.

If, on the other hand, the cylinder is not rigidly fixed, but is moving perhaps even slightly back and forth perpendicular to the flow direction, then this oscillatory motion will stimulate the development of a wake vortex in the most sheltered location. Since relatively long portions of the cylinder will be oscillating in the same phase, the generation of long vortices extending over a considerable cylinder length is now stimulated; this increases the magnitude of the lift force integrated over the cylinder length. If the frequency of vortex shedding, f, is much different from the natural frequency of the transverse vibration/then nothing very spectacular happens. (Be sure to check for a fatique failure, though.) When, on the other hand, f and the natural frequency of the transverse oscillation are nearly alike, a "locking-in" takes place - the vortex frequency shifts to agree with the natural frequency - and a forced resonant vibration results.

Addendum, Chapter 4 Wind Loads

Wind loads can be treated in much the same way as steady current forces. The important force components will be drag and lift with the former being the most important. Inertia forces - conceivable as a result of wind gusts - are universally neglected.

Drag forces can be predicted with a formula analogous to (4.01):

$$F_D = (\tfrac{1}{2}\ v^2)(A)(C_D) \qquad (4a)$$

where A is the projected area of the obstruction. Note that , above, is now the density of air - possibly containing water droplets (spray); common values range upward from about 1-2 kg/m^3.

A significant complicating factor in the computation of wind loads is the choice of the proper velocity for substitution in (4a). Friction along the earth's (ocean) surface will cause a velocity profile to develop; wind velocities become a function of elevation so that a design wind speed (for use in 4a, above) will be higher for the top of a crane structure than for the exposed truss work of the main jacket structure. This wind velocity profile can extend up to a height of, say, 150 meters. Wind speeds reported by meteorologists are usually measured at a standard elevation of 10 m above the earth's surface.

Values of the drag coefficient are also usually different from those listed in these notes. Values are too numerous to give details here, however.

New table data and text page 29.

H_{sig} (m)	$P(H_{sig})$ (-)	$P(H_{sig})$ (-)	char. H_{sig} (m)	N (waves/storm)	$P(H_d)$ (-)	E_1 (-)	$E_{2\,i}$ (-)
∞	0						
		$6.849\text{x}10^{-9}$	14	400	$1.688\text{x}10^{-2}$	$9.989\text{x}10^{-1}$	$6.841\text{x}10^{-9}$
13.5	$6.849\text{x}10^{-9}$						
		$1.598\text{x}10^{-8}$	13	500	$8.794\text{x}10^{-3}$	$9.879\text{x}10^{-1}$	$1.579\text{x}10^{-8}$
12.5	$2.283\text{x}10^{-8}$						
		$8.417\text{x}10^{-8}$	12	550	$3.866\text{x}10^{-3}$	$8.812\text{x}10^{-1}$	$7.417\text{x}10^{-8}$
11.5	$1.070\text{x}10^{-7}$						
		$3.486\text{x}10^{-7}$	11	600	$1.345\text{x}10^{-3}$	$5.540\text{x}10^{-1}$	$1.931\text{x}10^{-7}$
10.5	$4.556\text{x}10^{-7}$						
		$1.501\text{x}10^{-6}$	10	600	$3.355\text{x}10^{-4}$	$1.823\text{x}10^{-1}$	$2.738\text{x}10^{-7}$
9.5	$1.957\text{x}10^{-6}$						
		$6.295\text{x}10^{-6}$	9	700	$5.137\text{x}10^{-5}$	$3.532\text{x}10^{-2}$	$2.223\text{x}10^{-7}$
8.5	$8.252\text{x}10^{-6}$						
		$2.980\text{x}10^{-5}$	8	900	$3.137\text{x}10^{-6}$	$3.348\text{x}10^{-3}$	$9.978\text{x}10^{-8}$
7.5	$3.805\text{x}10^{-5}$						
		$1.422\text{x}10^{-4}$	7	1000	$8.119\text{x}10^{-8}$	$8.120\text{x}10^{-5}$	$1.154\text{x}10^{-8}$
6.5	$1.802\text{x}10^{-4}$						
		$6.897\text{x}10^{-4}$	6	2000	$2.234\text{x}10^{-10}$	$4.000\text{x}10^{-7}$	$2.759\text{x}10^{-10}$
5.5	$8.699\text{x}10^{-4}$						
		$2.938\text{x}10^{-3}$	5	2000	$< 10^{-10}$	≃ 0	≃ 0
4.5	$3.808\text{x}10^{-3}$						
		$1.422\text{x}10^{-2}$	4	2500	$< 10^{-20}$	≃ 0	≃ 0
3.5	$1.803\text{x}10^{-2}$						
		$6.279\text{x}10^{-2}$	3	3000	$< 10^{-30}$	≃ 0	≃ 0
2.5	$8.082\text{x}10^{-2}$						
		$9.192\text{x}10^{-1}$	2	3000	$< 10^{-80}$	≃ 0	≃ 0
≃ 0	1						
		$\sum = 1.000$					$\sum = 8.976\text{x}10^{-7}$

$$P'(H > H_d) = 0.0323$$

Our chosen wave, then, has the following properties:

Height	20 m	(chosen)
Chance of Exceedance	3.23%	(computed)
Period	10 s	(semi-computed)

This wave would be used in the Morison Equation to determine design loads. The structure designed for these loads would have 3.23% chance that it would fail during its lifetime.