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BRITISH STANDARD

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Code of practice for

Maritime structures —

Part 1: General criteria

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Foreword

This standard has been prepared under the direction of the Civil Engineering and Building Structures Standards Committee.

Following suggestions from the Maritime and Waterways Board of the Institution of Civil Engineers, the Standards Committee for Civil Engineering Codes of Practice set up an ad hoc panel to make further studies. The panel's report, presented in 1975, concluded that existing British codes were inadequate for the special aspects of maritime structures and that there was a need for such a code. A format was proposed which divided the work into two distinct stages.

It has been assumed in the drafting of this British Standard that the execution of its provisions is entrusted to appropriately qualified and experienced people, for whose guidance it has been prepared.

In this Part of the code, recommendations are given to assist clients and engineers to obtain the basic data relevant to the design of any maritime structure. It has been assumed that limit state philosophy will wherever possible be adopted for the design. However, it should be noted that certain phenomena, such as resonance and fatigue, cannot be treated rigorously by this method and a satisfactory adaptation of limit state methods has yet to be established as a practical basis for the geotechnical aspects of the design. Partial factors are not included in this Part, it being intended that these be given in subsequent Parts covering the limit state design of specific maritime structures.

It is planned to complete the code with a further seven Parts which would deal with the following:

a) the design of quay walls, jetties and dolphins;

b) the design of locks and graving docks, slipways and ship lifts;

c) the design of lock and dock gates, caissons and sluices;

d) the design of buoy moorings, floating and buoyant structures and anchorage systems;

e) the design of breakwaters and training walls;

f) the design of fendering and mooring systems;

g) recommendations for the dredging of water-ways and for land reclamation.

NOTE The numbers in square brackets used throughout this code relate to the bibliographic references in Appendix C.

A British Standard does not purport to include all the necessary provisions of a contract. Users of British Standards are responsible for their correct application.

Compliance with a British Standard does not of itself confer immunity from legal obligations.

Summary of pages

This document comprises a front cover, an inside front cover, pages i to x, pages 1 to 206, an inside back cover and a back cover.

This standard has been updated (see copyright date) and may have had amendments incorporated. This will be indicated in the amendment table on the inside front cover.

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Section 1. General

1 Scope

This Part of BS 6349 gives guidance and recommendations on general criteria relevant to the planning, design, construction and mainenance of structures set in the maritime environment. By definition, the structures to which this code is applicable will be located at, or close to, the shore. Offshore structures and structures in inland waters are not covered by this code although certain aspects may be found to be relevant to such projects.

Guidance is not given on financial criteria although it should be recognized that for each project necessary and proper economic assessments should be made and considered in conjunction with the engineering criteria covered by this code.

This code has been written in relation to conditions which obtain in the UK and while the majority of the contents are directly applicable elsewhere, local conditions or sources may necessitate appropriate modifications.

This Part of the code is arranged on a topical basis. Section 2 discusses environmental factors, in which the environment is taken in its broader sense to include all naturally occurring phenomena likely to be found at a coastal site and guidance is given on methods of investigating and quantifying their effects. Section 3 gives prominence to the need to consider operational requirements throughout the planning of maritime works although only limited guidance is given on some general aspects since the detailed functional requirements of individual structures are beyond the scope of this Part of this code. Section 4 discusses sea state and gives guidance on the characteristics, prediction, recording and effect of waves. Section 5 deals with the selection and evaluation of design loadings arising from the environmental and operational effects discussed in the previous sections, taking due account of dynamic response and fatigue. Section 6 discusses geotechnical aspects, including ground investigations, soil parameters, and geotechnical design requirements. Section 7 gives guidance on the use and specification of appropriate materials and protective measures.

2 References

The titles of the publications referred to in this standard are listed on page 205.

3 Definitions

For the purposes of this Part of BS 6349 the following definitions apply.

NOTE For other terms not separately defined the meanings follow the general usage of the maritime engineering industry.

3.1 Tides

3.1.1
semi-diurnal tides

tides that have two high waters and two low waters in a lunar day of approximately 25 h

3.1.2
diurnal tides

tides that have one high water and one low water in a lunar day

3.1.3
range

the difference in height between one high water and the preceding or following low water

3.1.4
spring tides

the two occasions in a lunar month when the average range of two successive tides is greatest

3.1.5
neap tides

the two occasions in a lunar month when the average range of two successive tides is least

3.1.6
mean high water springs (MHWS)

the average, over a long period of time, of the heights of two successive high waters at springs

3.1.7
mean low water springs (MLWS)

the average, over a long period of time, of the heights of two successive low waters at springs

3.1.8
mean high water neaps (MHWN)

the average, over a long period of time, of the heights of two successive high waters at neaps

3.1.9
mean low water neaps (MLWN)

the average, over a long period of time, of the heights of two successive low waters at neaps

3.1.10
mean sea level (MSL)

the average level of the sea surface over a long period, preferably 18.6 years (one cycle of the moon's nodes), or the average level which would exist in the absence of tides

3.1.11
lowest astronomical tide (LAT)

the lowest level that can be predicted to occur under average meteorological conditions and under any combination of astronomical conditions

NOTE It is often the level selected as the datum for soundings on navigational charts.

3.1.12
highest astronomical tide (HAT)

the highest level that can be predicted to occur under average meteorological conditions and under any combination of astronomical conditions

3.2 Ship tonnages

3.2.1
gross registered tonnage (grt)

the gross internal volumetric capacity of the vessel as defined by the rules of the registering authority and measured in units of 2.83 m^3 (100 ft^3)

3.2.2
deadweight tonnage (dwt)

the total mass of cargo, stores, fuels, crew and reserves with which a vessel is laden when submerged to the summer loading line

NOTE Although this represents the load-carrying capacity of the vessel it is not an exact measure of the cargo load.

3.2.3
displacement

the total mass of the vessel and its contents

NOTE This is equal to the volume of water displaced by the vessel multiplied by the density of the water.

3.3 Waves

3.3.1
wave height

the height of a wave crest above the preceding wave trough

3.3.2
wave period

the time for two successive wave crests to pass a fixed point

3.3.3
wavelength

the distance between consecutive wave crests

3.3.4
phase velocity

the speed at which a wave propagates

NOTE The terms "wave celerity" and "velocity of wave propagation" may also be used to describe phase velocity.

3.3.5
wave steepness

the wave height divided by the wavelength

3.3.6
group velocity

the velocity of propagation of a train of waves, i.e. the velocity at which the energy of the wave train travels

3.3.7
significant wave height

the average height of the highest one third of the waves

3.3.8
significant wave period

the average of the periods of the highest one third of the waves

3.3.9
zero-crossing wave period

the average period of all the waves with troughs below and crests above the mean water level

3.3.10
spectral density

a measure of the energy of the sea state expressed as a function of wave frequency and direction

4 Symbols

The following symbols are used in this Part of this code. Several meanings are given to some of the symbols and the specific meaning is given in each case in the text where the symbols are used.

A	Cross-sectional area.
A_L	Longitudinal projected area of vessel above waterline.
A_n	Area normal to flow.
a	Exponent in JONSWAP spectral density function.
B_v	Beam of vessel.
B_e	Breadth of earth-retaining structure.
b	Wave ray separation.
b_a	Wave ray separation at arrival point.
b_0	Wave ray separation in deep water.
C_B	Block coefficient.
C_{CL}	Depth correction factor for longitudinal current forces.
C_{CT}	Depth correction factor for transverse current forces.
C_c	Shallow water correction factor.
C_D	Drag force coefficient.
C_I	Inertia force coefficient.

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C_{LC} Longitudinal current force coefficient.

C_{LW} Longitudinal wind force coefficient.

C_M Hydrodynamic mass coefficient.

C_S Slamming force coefficient.

C_{TC} Transverse current force coefficient.

C_{TW} Transverse wind force coefficient.

c_u Undrained shear strength of soil.

c' Effective cohesion.

D_{50} Median size of graded stone (other subscripts for different percentages passing).

d Still water depth.

d_p Sheet pile penetration.

d_e Effective depth of sheet pile penetration.

d_c Maximum displacement under cyclic loading.

d_0 Deep water depth.

d_1 Water depth at one wavelength from wall.

d_t Depth of tension crack.

d_x Effective embedment of sheet piles.

d_A Depth of lowest trough below wave chart mean line.

d_B Depth of second lowest trough below wave chart mean line.

d_m Mean draught of vessel.

d' Depth below still water level of bottom of wall.

d_b Depth of water $5 \times H_s$ from wall (H_s calculated for depth "d").

d_m Depth of water over mound.

E Effective kinetic energy of berthing vessel.

e The exponent function.

F_D Drag force.

F_H Horizontal soil friction force.

F_I Inertia force.

F_{LC} Longitudinal current force.

F_{LW} Longitudinal wind force.

Fr Froude number.

F_S Wave slam force.

F_{TC} Transverse current force.

F_{TW} Transverse wind force.

F_V Vertical soil friction force.

F_W Total wave force.

F_1 Soil force (similarly F_2 and F_3).

F_B Bed friction stress.

f Wave frequency.

f_c Frequency of cyclic loading.

f_m Frequency at which peak occurs in spectrum.

f_N Natural frequency of structure or member.

g Acceleration due to gravity.

H Wave height.

H_R Retained height of structure.

H_a Wave height at arrival point.

H_b Breaking wave height at structure.

H_{inc} Incident wave height.

H_n The nth height or height interval in a rank-ordered set of n_x values.

H_{max} Maximum wave height.

$\overline{H}_{max}$ Average maximum wave height.

H_{n_x} The n_x th (maximum) height or height interval in a rank-ordered set of n_x values.

H_0 Wave height in deep water.

$Hr\theta$ Wave height at position (r,θ).

H_s Significant wave height.

$H_{s,n}$ The nth significant wave height in a set.

$H_{s,1}$ Component significant wave height from wave chart (similarly $H_{s,2}$).

H_1 Component wave height from wave chart (similarly H_2).

H_r Height of retained soil.

H_{0c} Clapotis set-up.

H_{sd} Significant height of set-down.

H_A Height of highest crest above wave chart mean line.

H_B Height of second highest crest above wave chart mean line.

H_L Limiting wave height in probability distributions.

H_w Height of wave run-up above still water level.

H_z Height of wind measurement above sea surface.

H_D Design wave height.

$H_{1/250}$ Mean height of the 0.4 % highest waves.

H_0' Effective deep water wave height.

h_c Height of top of wall above still water level.

$I(f,\phi)$ Spectral density function.

$I_0(f,\phi)$ Offshore spectral density function.

$I_D(\theta)$ Wave diffraction intensity factor.

i Number designating a stress range.

J Ratio of offshore wave steepness to square of beach slope.

K_A Coefficient of active earth pressure.

K_d Wave diffraction coefficient.

K_f Wave height reduction factor due to bed friction.

K_0 Coefficient of at-rest earth pressure.

K_p Coefficient of passive earth resistance.

K_r Wave refraction coefficient.

K_s Wave shoaling coefficient.

K_1 Empirical added mass coefficient (similarly K_2 and K_3).

K_b Bed friction factor.

K Stiffness of equivalent spring.

K A constant.

k Permeability

L Length of vessel.

L Wavelength.

L_{BP} Length of vessel between perpendiculars.

L_0 Wavelength in deep water.

L_w Waterline length of vessel.

L' Overall length of pile from deck to apparent fixity.

L_F Fetch length.

L_s Submerged length of member.

l Length of cylinder.

l_e Effective length of anchorage.

l_0 Ineffective length of anchorage.

l' Length of cylinder from water level to apparent fixity.

m_D Displacement of vessel.

m_{50} Median mass of graded stone.

m_L Mass per unit length of member.

$\overline{m}$ Equivalent excited effective mass per unit length of member.

m_e Equivalent mass of structure.

m_c Machine payload capacity.

m_s Mass of cellular structure and contained soil.

N Number of waves in design condition duration.

N_s Scale factor.

N_i Number of waves in ith stress range during design life.

N_z Number of zero up-crossings on wave record.

n A number between 1 and n_x.

n_T Total number of stress ranges.

n_x A number.

n_i Number of waves in the ith stress range during design life.

P Maximum applied cyclic load.

P_A Active soil force per unit length of wall.

P_{max} Maximum water pressure on wall at seabed.

P_{min} Minimum water pressure on wall at seabed.

P_p Passive soil force per unit length of wall.

$P_{w\,1}$ Average maximum wave pressure on wall at still water level.′

$P_{h,1}$ Maximum hydrodynamic pressure on wall at still water level.

P_u Pore water pressure.

p Probability.

p_n Probability of nth value being equalled or exceeded.

P_{w2} Average maximum wave pressure on wall at its foot.

P_u Effective uplift wave pressure at foot of wall.

q Proportion of critical damping.

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R	Resistance per unit area to shear along soil slip surface.
Re	Reynolds number.
r	Radius.
r	Polar coordinate.
$S(f)$	One-dimensional spectral density function.
s	Bed slope (tangent of angle relative to horizontal).
T	Wave period.
T_0	Period over which observations are taken.
T_p	Period at which peak occurs in wave spectrum.
T_R	Return period
T_s	Significant wave period.
T_z	Zero-crossing wave period.
t	Variable used in normal distribution.
t	Time variable.
t_{90}	Time required for 90 % of original bacteria in a sample to die.
U_w	Wind speed 10 m above sea surface.
U	Instantaneous water particle velocity normal to member.
U_z	Wind speed at z m above sea surface (other subscripts for different heights).
$\dot{U}$	Instantaneous water particle acceleration normal to member.
u	Horizontal component of water particle velocity.
$\dot{u}$	Horizontal component of water particle acceleration.
V	Incident current velocity.
V_B	Velocity of vessel normal to berthing face.
V_C	Design current speed.
V'_C	Average current velocity over mean depth of vessel.
V_{crit}	Critical current velocity.
V_w	Design wind speed.
V_η	Vertical velocity of water surface.
v	Vertical component of water particle velocity.
v_s	Average fall velocity of sediment in still water.
$\dot{v}$	Vertical component of water particle acceleration.
v^*	Friction velocity.
v_c	Velocity of wave propagation.
v_{cg}	Wave group velocity.
v_{cga}	Wave group velocity at arrival point.
v_{cg0}	Wave group velocity in deep water.
v_{c0}	Velocity of wave propagation in deep water.
w_p	Water particle orbit width at surface.
w_F	Fetch width.
w_s	Width or diameter of submerged structure or member.
X_i	Component of radial in direction parallel to mean wind.
x	Rectangular coordinate.
x_w	Wave prediction parameter.
Y_a	Dimension of submerged member.
Y_b	Dimension of submerged member.
Y_c	Dimension of submerged member.
y	Variable used in normal distribution.
	Rectangular coordinate.
Z	Tidal lag.
z	Pile wall thickness.
α	Angle of current off bow.
α_r	Angle of wind radial relative to mean wind direction.
α_s	Angle of slope from horizontal.
α_0	Angle of wind off bow of vessel.
α_c	Angle of current relative to member axis.
α_1	Coefficient of wave pressure at surface dependent upon wave period.
α_2	Coefficient of wave pressure at surface due to shoaling.
α_3	Ratio between wave pressures at surface and at depth d'.
β	Angle of resultant wind force off bow of vessel.
β	Angle between direction of wave approach and normal to breakwater.
β_0, β_1 and β_{max}	Coefficients used in calculation of significant wave height in surf zone.
β^*_0, β^*_0 and β^*_{max}	Coefficients used in calculation of maximum wave height in surf zone.

γ	Term in JONSWAP spectral density function.
γ	Effective bulk weight density of soil.
γ_d	Drained bulk weight density of soil.
γ_s	Submerged bulk weight density of soil.
γ_w	Weight density of ground water.
Δ	Logarithmic decrement of structural damping.
δ	Angle of wall friction.
δ_m	Mobilized angle of friction between soil and structure.
δ_{max}	Maximum angle of friction between soil and structure.
δ_s	Angular ray separation at harbour entrance.
η	Instantaneous height of water surface above still water level.
η^*	Elevation above still water level to which wave pressure is exerted.
θ	Polar coordinate.
θ_0	Angle of incident wave relative to breakwater axis.
κ_J	Coefficient in JONSWAP spectral density function.
κ_P	Coefficient in Pierson-Maskowitz spectral density function.
λ	Term used in wave chart analysis ($= \log_e N_z$)
λ	Coefficient of local wave pressure.
μ	Coefficient of friction.
υ	Kinematic viscosity of water.
π	Ratio of circumference to diameter of circle.
ρ	Mass density of water.
ρ_A	Density of air.
σ	Stress normal to plane of sliding.
τ_0	Shear stress exerted on bed by flowing water.
ϕ	Angle defining wave direction.
ϕ_r	Angle of soil shearing resistance.
ϕ_m	Mean wave direction.
ϕ_0	Offshore wave direction.
ϕ_1	Limiting wave direction (similarly ϕ_2).
ϕ'	Effective angle of soil shearing resistance.
ψ	Angle between wave front and bed contour.
ψ_0	Angle between wave front and bed contour in deep water.
ω	Term in spectral density functions.

Section 2. Environmental considerations

5 General

A fundamental prerequisite to designing a maritime structure is the understanding and assessment of the naturally occurring phenomena to which that structure will be exposed. Information concerning these phenomena may already be available from existing sources; however such data may often be limited in scope and application and further detailed investigations may be required to permit the selection of design parameters and to assess the behaviour or impact of the proposed works on their environment.

This section describes the various environmental phenomena which should be considered for investigation at a coastal site and gives information and guidance on methods of data collection.

6 Survey control

6.1 General. The validity of many measurements undertaken on site is dependent on the accuracy to which they have been positioned and levelled. Whenever possible, investigation work should be related to the established land survey system, which in the UK would be the Ordnance Survey National Grid. Should this not prove possible then a local grid system orientated by azimuth should be established prior to all other site operations, with sufficient permanently monumented survey stations to allow recovery of the survey grid throughout at least the construction time of the structure. It should always be borne in mind that although a simple and relatively inaccurate scheme of survey control may suffice for the purposes of the initial investigations, it may at a later date be required to control other work that is connected with the structure but will demand a much more rigorous approach.

6.2 Level control. Most investigations connected with surveys for maritime structures will require a vertical reference in the form of a water level or tide gauge. Such a gauge should be established as close as practicable to the scene of the investigations, the required degree of closeness being largely dependent on the nature and range of the tide in the locality. Siting of the gauge is important and the following factors should be considered:

a) the gauge should be in sufficiently deep water to avoid drying out;

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b) it should be sheltered as far as possible from the effect of sea and swell;

c) it should not be in a position where water is impounded as the tide drops;

d) it should be reasonably close to a national or local land levelling datum reference point;

e) it should be sheltered from accidental damage by vessels and should not be mounted or fixed on members which are subject to settlement.

Types of gauges are described in **10.4**.

6.3 Location control

6.3.1 *General.* Various techniques are available for determining the positioning of measurements undertaken over water in relation to the land survey system established ashore. They fall into three broad categories, i.e. visual, radio and electro-optical, with range capabilities extending from a few hundred metres to several hundreds of kilometres. Survey work associated with maritime structures will rarely call for ranging capabilities beyond a few kilometres and it is within this limitation that the following reviews of positioning systems are contained.

6.3.2 *Visual methods.* Generally the overall accuracy of visual positioning methods decreases with increasing distance and the maximum range for these techniques is approximately 8 km.

Sounding sextants provide probably the most economical means of positioning in situations where distances to coordinated targets are not more than about 4 km and where visibility is not a problem. In experienced hands, sextants can provide a positioning accuracy of ± 3 m or better although such accuracies are unlikely to be obtained at a range of 4 km. It has to be emphasized that accurate sextant work is a skill acquired only through practice and experience and inexperienced observers may produce erroneous results.

Positioning is usually achieved by resection using two horizontal sextant angles, observed simultaneously from the boat, to three coordinated targets ashore. The accuracy of such a fix is governed by the following factors:

a) the calibration of the sextants;

b) the observing technique of the observers;

c) the positioning of the observers relative to the point of measurement: both observers should stand close together and preferably over the point of measurement;

d) the geometry of the fix;

e) other "common sense" factors, such as not observing through glass.

A secondary method of positioning by sextant which can be usefully employed in certain situations is the use of a single horizontal sextant angle and transit. Provided that the observer is on the transit, the cut-off angle will provide an accurate measurement of the range to the forward transit target subject to the observation marks being located each side of the transit. The method again calls for skilful technique both on the part of the observer and the boat handler. It is a method which is particularly useful in confined situations and, as a general rule, should not be employed over distances greater than 200 m from the forward transit marker.

Theodolites set up ashore can be used to position a boat offshore by intersection. Although a theodolite can measure to an accuracy of 25 mm as opposed to the sextant's capability of 300 mm, a higher order of positioning accuracy is not necessarily achieved. Lack of synchronism between the shore and afloat elements of the survey work is the main source of error. Use of theodolites also reduces the flexibility of a survey operation since it may be necessary to have frequent changes in the position of the theodolites in order to maintain well conditioned intersections.

6.3.3 *Radio-positioning methods.* Radio-positioning systems can be broadly divided into three categories, i.e. short, medium, and long range. This review is concerned only with the short range systems which, typically, provide a positioning capability up to 30 km offshore. Some short range systems are capable of ranges in excess of 30 km but this requires very high antennae and/or increased transmitter power.

Short range systems operate in marine radar bands S, C and X which correspond to approximately 3 GHz, 6 GHz and 9 GHZ, respectively.

Most operate in a range-range mode, measuring the ranges from two shore reference stations to a mobile master station. The usual update rate is better than one second and accuracies of ± 3 m or better on each range are generally achieved under good conditions.

One X band system is designed to produce position by range and azimuth. Ranging accuracy is similar to the range-range systems. Azimuth accuracy is claimed as better than 600 m. This system is particularly useful in situations where range-range shore reference stations cannot be positioned far enough apart to provide a well conditioned fix or in narrow rivers where the use of range-range systems would involve the setting up of a much larger number of shore reference stations.

A particular feature of all these systems is their overall effectiveness in the inshore situation. They are light and portable and can be installed by relatively unskilled personnel easily and quickly. They are generally very reliable and have low power consumptions allowing them to operate off standard 12 V batteries for several days without attention.

All such systems are essentially line-of-sight systems and thus their effective range is dependent on the height of both shore and mobile station antennae. Heavy rain can attenuate transmitted signals and significantly reduce ranging capability. Reflections from large vessels or shore features can cause gross ranging errors but such situations are usually obvious to the user and can be guarded against.

The short range systems achieve ranging either by phase comparison or by pulsing techniques. In general, phase comparison systems have a better ranging accuracy with ± 1 m possible in good conditions. They are, however, more complex and more expensive. The accuracy of pulsing systems is generally quoted as ± 3 m, which is usually attainable in good operation conditions.

6.3.4 *Electro-optical, infra-red and laser systems.* Electro-optical, infra-red and laser systems are generally single range measuring instruments developed for land survey and with limited application for survey work afloat. Certain types of electro-optical or laser devices are sometimes useful in situations where it is required to run a survey craft along a line of constant bearing. The instrument is set up ashore or on a fixed structure and the observer in the survey craft can steer visually along the narrow beam produced by the instrument.

Laser instruments have also been developed which can provide a single range from a moving craft to any surface capable of reflecting the laser beam. This type of instrument can be particularly useful for work in confined areas such as docks or close to harbour walls. Range capabilities of existing instruments are presently limited to around 120 m, when using existing reflecting surfaces, but up to 600 m and beyond can be obtained from specially installed retro-reflectors.

7 Meteorology and climatology

7.1 General. This clause describes the meteorological and climatological considerations which should be taken into account during the data collection, design, construction and operational stages of a proposed maritime structure. The type and method of collection of information and some possible formats for the final presentation of data are also outlined.

Authoritative meteorological and climatological data can generally be obtained from the meteorological office covering the area under consideration.

Of particular interest to the construction industry is the quick reply climatological service (CLIMEST) developed by the Meteorological Office, Bracknell, Berkshire, England, to assist tendering and planning in the UK especially in estimating the time likely to be lost because of adverse weather. CLIMEST gives estimates of average monthly and annual values of rainfall, low temperatures and strong winds applicable to the site. The information provided by this service is not generally adequate for detailed planning and a further site-specific data search should be undertaken.

7.2 Wind

7.2.1 *General.* The constantly maintained temperature difference between the poles and the equator provides the energy supply necessary to drive the planetary atmospheric circulation. This circulation involves both vertical and horizontal movements of air although the average horizontal wind speeds are generally of the order of one hundred times greater than the average vertical speeds. Important exceptions to this rule do occur, for example in convective storms or hurricanes, tropical cyclones or typhoons, for which special structural design is usually indicated in the local codes of practice. Naturally occurring vertical air movement also becomes important in the study of dispersion of chimney and stack emissions and at sites subject to mountain and valley winds or winds affected by major topographic barriers. The predominant controls on the horizontal movement of air near the earth's surface are the pressure gradient force, frictional forces and forces due to the rotation of the earth, i.e. Coriolis and centripetal forces.

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Irrespective of long term origin and subsequent modification, the wind at and surrounding a maritime construction site is of great significance to the designer, both for the loads it imposes directly on the structure and for the associated wave heights, wave loading and mooring forces. Loading is discussed in section 5, whilst the forecast of waves from wind measurements is discussed in section 4. With regard to the latter, it is important that wind direction is known so that the fetch can be adequately defined, thereby avoiding the error of forecasting high waves from strong winds blowing off the land.

Long term wind records are particularly necessary for coastal construction for estimating weather windows and likely delays during site investigation, construction and operation. The effect of the wind on water set-up, storm surge generation and coastal up-welling is of importance. Local variations in the wind regime caused by topographic influence should be considered when defining design criteria. In addition, a daily cycle of land and sea breezes may be superimposed on a larger scale wind field. Typical velocities of sea breezes are 4 m/s to 7 m/s and of land breezes only about 2 m/s. The time of observation is thus important and daily readings at say 0900 hours local time may not necessarily give data representative of the full 24 h.

Reliable records of wind speeds and directions are available covering most parts of the world, including the oceans, for a large number of years. Such records should provide adequate information for the derivation of the relevant design parameters, where not already available from local wind-loading codes of practice. Records required should include those relating to:

a) fetch areas where incident waves could be generated;

b) cyclone or cyclonic depressions, for correlation with surge occurrences;

c) typhoon or hurricane tracks and intensities in the vicinity of the site or relevant fetch areas;

d) wind speeds and directions in the vicinity of the site.

Both maximum gust and average wind speeds should be obtained as well as wind direction (see **7.2.4**). Records should be checked to ensure that corrections appropriate to the particular recording station have been applied (see **7.2.3**).

7.2.2 ***Wind sensors***

7.2.2.1 *General.* Wind speed and direction are usually measured by means of some form of anemometer and wind vane. The output may be either recorded or read at regular intervals. The former provides more information, especially regarding the strength and frequency of gusts, and the latter yields only short term averaged readings.

Several types of instrument are available as described in **7.2.2.2** to **7.2.2.5**. All instruments have advantages and disadvantages and expert advice should be obtained before installation.

7.2.2.2 *Cup and vane anemometers.* Cup anemometers work on the principle that the rate of rotation of vertically mounted cups is proportional to the wind speed. The output is transmitted either as a series of pulses or as a variable voltage. The former is generally totalized and read daily giving an estimate of the run of the wind. The voltages from both the cups and the wind vane on the latter are either output onto voltmeters and read regularly or recorded continuously.

7.2.2.3 *Impeller type.* The impeller-type anemometer is similar in function to the cup instrument except that the impeller is mounted horizontally along the axis of the wind vane, resulting in a more compact rugged unit.

7.2.2.4 *Pressure type.* The pressure-type anemometer consists of a hollow tube, one end of which is open and balanced to face into the wind and the other is connected to a sensitive manometer. The pressure in the tube is directly related to the wind speed.

7.2.2.5 *Other types.* Many other types of anemometer have been developed, but they are mainly for specialized uses and are not generally considered as field instruments. They include vortex velocity meters, sonic anemometers, hot-wire anemometers and laser-Doppler anemometers.

7.2.3 *Exposure and level of measurements.* Care should be taken when siting an anemometer since eddies created by obstructions such as trees and buildings may produce unreliable measurements of the wind speed and direction and hence inaccurate estimates of the local wind regime.

In order to allow the comparison of wind records from different locations and heights, measurements should be referred to the standard height of 10 m above ground level. Whenever possible, measurements are made at 10 m or at a greater height to obtain an exposure clear of surrounding obstacles. Each instrument siting should be given an effective height which is defined as the height over open level terrain in the vicinity which, it is estimated, would have the same mean wind speeds as those actually recorded on the anemometer. For comparison with an effective height of 10 m a correction has to be applied which will vary with different averaging time intervals and site conditions and expert advice should be sought before measured values are corrected.

Historically, the smallest time interval which could be identified was 3 s, due to the averaging time of the Dines pressure tube anemograph and thus the concept of the 3 s maximum gust was developed. This gusting interval has been accepted for general structural design purposes and its use is discussed in full in CP 3:Chapter V-2; however gusts at intervals of several milliseconds can now be measured and research may be necessary to define significant gusting intervals for structures which are particularly sensitive to excitation by very short gusts. In particular, the spectral approach has been considered for wind loading due to gusts. For very large vessels which have slow response times, considerably longer gusting intervals will be necessary to develop the full fender and mooring line strains. Further studies may be desirable in order to evaluate the lower design gust speed which will be appropriate in such cases.

7.2.4 *Presentation and use of wind data.* Standard wind records which are usually stored as hourly mean speed and direction and maximum gust speed, may be used as follows.

a) As a basis for wave prediction (see Section 4).

b) To calculate averages and maxima, for example as shown in percentage frequency tables.

c) To prepare wind roses or similar diagrams which provide a pictorial summary of the frequency distribution of wind direction and speed measurements.

d) To calculate extreme wind speed values by plotting measured maxima on a probability scale and extrapolating along the line of best fit. A return period or recurrence interval can thus be defined for a given wind speed such that it is likely to be exceeded on the average only once in that period. Data can also be plotted on a linear percentage exceedance diagram to illustrate the probability of a given measured wind speed either being greater or less than a certain limit.

e) To prepare persistence diagrams to show the expected duration and number of occurrences of particular wind speeds that are likely to be experienced.

7.2.5 *Alternative wind information.* If no measured wind data are available, the atmospheric pressure distribution may be used to calculate a smoothed picture of the wind field. Such methods are very approximate and are generally only used for weather forecasting and wave prediction. The associated calculations should be carried out by specialists.

7.3 Precipitation

7.3.1 *General.* The effect, type and intensity of precipitation should be assessed when considering the following aspects of design.

a) *Drainage design.* Estimates of maximum expected rainfall from a 50 yearly or 100 yearly storm should be made.

b) *Dead weight loading.* The accretion of snow or ice will exert load on a structure. The amount and expected duration of snowfalls should be considered.

c) *Cargo handling.* The annual variation in rainfall is important with reference to the type of cargo handled and the storage facilities in a port complex.

d) *Penetration.* A high frequency of driving rain may necessitate special protection for buildings.

e) *Construction delays.* Frequent rain will increase construction time substantially, especially earth-moving operations.

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7.3.2 *Direct point measurement.* Rain gauges measure the rainfall over a small known area, historically 19.6 in^2 for a standard 5 in diameter synoptic gauge and more recently either 150 cm^2 or 750 cm^2 depending on the time between inspection of the collected precipitation and the degree of resolution required for measuring small quantities. It is therefore unlikely that the measurements will be directly representative of a larger area due to topographic changes and man-made influences. This has to be considered both during calculations of total available water and in particular whenever historic rain data are used for a purpose other than that for which they were originally collected.

7.3.3 *Exposure and measurement.* Care has to be taken in the exposure of rain gauges; they are usually located on flat ground with the top of the collecting funnel 0.3 m above the ground and well clear of any obstructions.

The simplest gauges consist of a collector with knife-edged circular rim and a funnel leading into a receiver. The precipitation collected can be measured directly by transfer to a vessel graduated for use with the particular type of gauge and size of collecting area. Alternatively, a recording device can be used such as a tilting siphon or tipping bucket. These gauges generally underestimate rainfall when compared with a standard gauge but can give useful information on time of occurrence and intensity. The rainfall events are generally recorded on clockwork drum charts or on automatic data loggers using a recording medium such as magnetic tape or are telemetered direct to the user, as in flood warning networks, for example.

7.3.4 *Water balance.* Where it is impractical to install a rain gauge network it may be possible to calculate the rainfall by considering the total water budget of a catchment including river flow rates, evapotranspiration and infiltration measurements.

7.3.5 *Historic data.* Authoritative rainfall data may be obtained from the local meteorological office or from local water authorities, although some calculations may be necessary before the information can be applied to any particular site.

7.4 Air temperature and humidity

7.4.1 *General.* Estimates should be made of minimum and maximum air temperatures and the variations in relative humidity which are likely to be encountered during the life of the structure.

7.4.2 *Air temperature.* To provide suitable design data four temperatures should be measured at regular intervals, i.e. minimum and maximum, and wet and dry bulb. The thermometers should be exposed inside a Stevenson screen with the bulbs at 1.25 m above the ground.

7.4.3 *Relative humidity.* This is defined as the actual water vapour pressure expressed as a percentage of the saturated water vapour pressure at the same temperature. It is a measure of the dampness of the air and is important in cargo storage and in certain aspects of building construction, particularly concrete curing. Relative humidity is calculated from wet and dry bulb temperatures or can be measured directly using hygroscopic material.

7.4.4 *Historic data.* Information on averages and extremes of daily, seasonal and annual temperatures and relative humidity may be obtained from the local meteorological office or in published tables for selected sites.

7.5 Visibility

7.5.1 *General.* Poor visibility may have severe consequences in inshore waters and estimates of the expected duration should be made. The reduction of atmospheric transparency and hence visibility is caused by two predominant factors:

a) a suspension of extremely small dry particles, called haze;

b) suspended microscopic water droplets or wet hygroscopic particles, known as mist.

Fog is a term conventionally applied when the horizontal visibility at the earth's surface is reduced to less than 1 km.

7.5.2 *Measurement.* The method of measurement generally employed for synoptic purposes is the observation of objects at known distances, for example conspicuous trees, church towers, surrounding high land or, at night, lights specially erected for the purpose.

Visibility, in common with many other meteorological variables, often changes sharply near the coast between the widely different regions of sea and land. At coastal stations of the UK Meteorological Office, however, the visibility over the land is recorded as standard even if this is different from the visibility over the sea. The latter is recorded as a remark if it can be estimated. Caution should be used when studying visibility reports from a station not directly on the coast as the phenomenon known as sea fog may not be experienced more than 3 km to 4 km inland and erroneous data can therefore be extracted for navigation and piloting purposes.

Recent research has been conducted into the automatic measurement of visibility to remove observer bias and provide an input for unattended measuring stations. Techniques investigated include measuring the attenuation of a light source of known output over a prescribed distance and the amount of back scatter from a light source, but no single method has proved successful for all applications.

7.6 Atmospheric pressure

7.6.1 *General.* The pressure of the atmosphere at any point is the weight of the air which lies vertically above a unit area. The atmospheric pressure is recorded in millibars[1)] and is generally corrected to mean sea level, to remove the influence of altitude on the measurements.

The pressure distribution is used as a basic input parameter in the preparation of weather, and hence sea state, forecasts. In particular, the influence of pressure on water level is of importance. A high or low pressure will decrease or increase, respectively, the depth of water to cause what is known as a storm surge, either negative or positive (see clause **10**).

7.6.2 ***Measurement***

7.6.2.1 *General.* The techniques described in **7.6.2.2** to **7.6.2.4** may be employed.

7.6.2.2 *Mercury barometer.* The pressure of the atmosphere is balanced against the weight of a column of mercury. The two main types are the Fortin and the Kew-pattern barometers.

7.6.2.3 *Aneroid barometer.* An aneroid barometer consists of one or more thin metal capsules, partially evacuated of air. The change in the thickness of the capsules is magnified and typically represented by a needle and dial. Precision aneroid barometers are sensitive to 0.1 mbar with an overall accuracy of ± 0.3 mbar.

7.6.2.4 *Barograph.* Standard barographs are aneroid instruments. The variation in pressure alters the shape of the sensor and this change is magnified through a system of pivots and levers. The pressure is recorded continuously on a chart secured to a clock drum.

7.7 Solar radiation and hours of sunshine

7.7.1 *Measurement.* The intensity of solar radiation is usually measured with radiometers using a thermopile element which generates a millivolt signal output which can be directly related to the energy received at the sensor. Radiometers can be designed to be sensitive in particular wavelengths of interest, for example incoming short wavelengths or outgoing long wavelengths or in the specific bands of ultra-violet or infra-red.

The duration of bright sunshine is generally defined and recorded by the length of the burn or burns produced by focussing the sun's rays on to a special card and reported as hours of sunshine.

7.7.2 *Uses.* The incoming, reflected and emitted radiation is of prime importance in thermal balance equations and can be used to estimate the cooling capacity of an area of water and the potential evaporation.

The life expectancy of bacteria released to the sea is thought to be highly dependent on the intensity of solar radiation particularly in the ultra-violet wavelengths. The calculation or measurement of this mortality is important in sea outfall design for effluents such as domestic sewage. The effect of light intensity on marine fouling is well illustrated by the vertical sequence of species found with changing depth on immersed structures.

8 Bathymetry and topography

8.1 *General.* Bathymetric surveys are produced by taking a series of measurements of water depth at known locations over the area of interest. Information on the various techniques available and guidance on their application to maritime engineering is given in this clause. Guidance on location control is given in clause **6**.

Published bathymetric charts provide the designer with very limited data for his purposes. Charts are produced for safe navigation and features which may be of particular significance to the engineer may not necessarily be shown. Charts are only as accurate as the data from which they were compiled. Very often much data will have been taken from old surveys not conducted to modern standards or at scales not suited to modern requirements. Possible changes due to siltation, dredging, dumping, shipwrecks or other causes should be considered and proving surveys should be carried out to check existing charts where there is any doubt as to their reliability.

[1)] 1 mbar = 100 N/m^2 = 100 Pa
1 000 mbar ≈ 29.5 inHg

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8.2 Echo sounder

8.2.1 *General.* The prime method of measuring water depth is by echo sounder, which will record a continuous profile of the seabed as the survey craft moves over it. Only purpose-built hydrographic echo sounders should be used if accuracies acceptable for engineering purposes are to be obtained. The typical hydrographic echo sounder will produce a large scale paper trace providing a permanent graphic analogue record of the seabed profile. The instrument will be fitted with a range of controls to enable the operator to adjust and calibrate it to suit the conditions in which it is operating and built to a specification that enables it to maintain reliable and accurate recordings. Echo sounders can give misleading results in areas of very soft mud which sometimes occur in estuaries and in these cases other methods should be used, depending on the application.

8.2.2 *Calibration.* The echo sounder measures depth by sending out a short ultrasonic pulse of sound and measuring the time taken for this pulse to reach the seabed and be reflected back to its source. The accuracy of depth measurement therefore is dependent on the allowance made for sound velocity through the water.

Calibration is normally effected by lowering a target to a set depth below the sound source, i.e. the transducer, of the echo sounder and comparing the recorded depth against the actual measured depth of the target. The echo sounder can then be adjusted to record true depth. This method, commonly known as a bar check, is effective for calibration down to depths of about 20 m.

Calibration should be checked using the bar in the area of survey and at intervals throughout a working day as both sound velocities and functioning of the echo sounder itself are subject to fluctuations with time and position.

8.2.3 *Transmission frequencies.* The capability of an echo sounder is much affected by the transmitted frequency. Low frequencies transmit energy efficiently over long distances and will often penetrate through soft seabed materials, producing echoes off underlying strata.

Low frequency equipment, however, generates wide beam widths (see **8.2.4**) and is of necessity large and cumbersome. High frequency transmissions can be generated with narrow beam widths from light compact equipment. However, energy is not so efficiently transmitted, being quickly reduced in water, and not generally capable of penetration into the seabed.

Hydrographic echo sounders designed for use in shallow to medium water depths operate typically at a compromise frequency of around 30 kHz. Higher frequency echo sounders operating at between 150 kHz and about 210 kHz are also used and are useful in shallow water. Some echo sounders operate on dual frequencies of say, 33 kHz and 210 kHz, thus combining penetration through very soft sediments with an identification of the depths of these upper sediments themselves.

8.2.4 *Beam width.* The echo sounder transducer generates a beam of acoustic energy which will cover the seabed over an area dependent on the beam width and the depth. Thus, with a narrow beam width less seabed is covered but definition of seabed features is better. A high frequency, narrow beam width echo sounder would be particularly useful when it is required to obtain details of a rocky, precipitous seabed. This benefit is of course largely lost if the survey craft itself is pitching or rolling. On the other hand, a wider beam, lower frequency echo sounder would be better in flat or gently undulating seabed conditions, providing greater coverage with the additional benefit of some penetration of very soft sediments.

Beam width is not directly dependent on frequency, but on the design of the transducer. It is much easier and cheaper to design and build narrow beam transducers operating on high frequencies than low frequencies. However, not all high frequency echo sounders necessarily generate narrow beam widths and manufacturers' specifications need to be consulted.

8.3 Side-scan sonar. Working on the same basic principles as an echo sounder, side-scan sonar systems transmit a fanshaped beam of acoustic energy perpendicular to the track of the survey craft. The reflected signals from rock outcrops, sand waves, pipelines and any other projections on the seabed are recorded as changes in density on the continuous paper roll record produced. Changes in the nature of the seabed materials may also be detected.

Side-scan systems can be either hull mounted or towed, the advantage of the latter method being that the sound source is closer to the seabed, enabling changes in seabed texture to be determined more easily and the transducer is partly decoupled from the ship's motion. A hull-mounted system has the advantage that there is less chance of damage to the equipment and in inshore waters is generally adequate for locating rock outcrops and underwater obstructions and the position of the transducer is known more precisely.

Although some indication of bathymetric changes can be gained from analysis of the side-scan records, it is essentially a search device. Complete coverage of the seabed by echo sounder would usually involve an excessive amount of work and is not generally justifiable. Use of side-scan sonar enables the surveyor to examine the entire seabed between sounding profiles and to run additional profiles where side-scan records show significant changes in the bathymetry taking place. Side-scan sonar systems operate typically at 50 kHz to 100 kHz with a range capability of up to 400 m on each side of the vessel track.

8.4 Direct measurement. Direct measurements are occasionally required when the echo sounder records are doubted. This can occur when sounding over a particularly soft seabed or when large quantities of weed or kelp are present. It might also be necessary when establishing the least depth over a rock or obstruction. Such measurements are usually made by hand leadline, graduated pole or sweeping with a horizontal wire (see **8.6**).

8.5 Spacing and direction of sounding profiles. The appropriate spacing of sounding profiles will depend partly on the purpose of the survey and partly on the depth and nature of the seabed.

Where the bathymetric information is required for the study of wave and current effects or for the study of navigation channels where the average depth is greater than 1.5 times the draught of the largest ship expected, a chart scale of the order of 1 : 10 000 is generally sufficient. Lines of soundings should be spaced at about 100 m intervals with a locational fix spacing not greater than 300 m.

Surveys required for the study of wave effects should cover the transitional zone relative to the wave periods expected at the site.

For navigation channels, sweeping surveys may be considered as an alternative to detailed sounding surveys where the depth is greater than 1.2 times maximum draught (see **8.6**).

In rock and coral areas, more detailed surveys are advisable for navigation channels if the average depth is less than twice the maximum draught of the ship.

Where the bathymetric information is required for the siting of structures, measurement of dredging and checking navigation channels in shallow water, a detailed survey should be carried out to produce a chart to a scale of between 1 : 500 and 1 : 1 500.

Typically, a survey for a maritime structure would be made at a profile spacing of 10 m to 25 m in and around the proposed position of the structure and 50 m in the approach areas with locational fixes taken at approximately three times the profile spacing. Side-scan sonar could be used in conjunction with the echo sounder to obtain a qualitative record of the seabed between profiles, revealing any features undetected by the sounder.

The direction in which profiles are run is again dependent on the purpose of the survey. The usual purpose is to delineate depth contours as accurately as possible. This is achieved by running across the anticipated contours as near as possible at right angles. It is good survey practice to run additional profiles at right angles to the chosen direction at a wider spacing, for example 5 to 10 times. These lines will serve the purpose of check lines and also reveal the presence of features such as sand waves which, because of the original choice of direction, may not have been identified.

8.6 Wire sweep. A wire sweep survey may be necessary for determining the least depth over an obstruction or to prove the absence of obstructions in a particular area.

The former can be achieved by suspending a horizontal bar fore and aft from the vessel, which drifts over the known obstruction. After each pass, the bar is lowered or raised by its calibrated support wires and, by noting the bar's depth setting and applying the tide correction, the least depth over the obstruction can be determined.

As an alternative to side-scan sonar, for ensuring an area free of underwater obstruction, a fine-wire sweep survey can be conducted. The fine wire is suspended between two boats and weighted to maintain a predetermined depth. The boats proceed at 1 m/s to 1.5 m/s (2 knots to 3 knots) along the necessary course lines to ensure full sweep coverage of the area.

8.7 Reduction of soundings. Due to the vertical movement of the water level, soundings obtained have to be reduced to a standard reference plane by subtracting the actual height of the water level above this plane at the time of sounding. The rise and fall of the water level is observed on a tide pole or tide gauge during the sounding operation and heights plotted against time. This gauge should be as close to the area as possible. From the curve showing the variations in water level, soundings can be reduced to a suitable reference plane or datum (see also clause **10**).

Care should be exercised to ensure that soundings are recorded in synchronism with the tide readings. This is most important where a high rise and fall is expected.

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8.8 Coastal topography. Depending on the survey area and the scale of presentation, the final bathymetric plan should portray the coastline and prominent features, in order to allow immediate visual recognition.

It is general practice to show the high and low water contours on bathymetric charts. Should the bathymetry be conducted over the high water period it is usually possible to determine the low water contour from the sounding results. To obtain the high water contour it would be necessary to carry out levelling, by land survey practice, over the area not already covered during the sounding operation.

Although the bathymetric survey may initially be required only for navigational or wave study requirements, the results of many inshore bathymetric surveys may subsequently be used for structural planning purposes in conjunction with the results of the local land-based topographical survey.

Therefore the bathymetric plan should preferably be plotted on the same grid system as the land survey and the relationship between the vertical reference planes of each should be clearly recorded (see clause **6**).

9 Geotechnology

A thorough understanding of subsurface conditions is an essential preliminary in any maritime construction project. Section 6 includes guidance on the type and extent of geotechnical studies required, including geological, geomorphological and geophysical aspects.

10 Water level

10.1 *General.* The underlying long-period fluctuations in general water level result from astronomical tides which are generated primarily by the cyclic variations in gravitational attraction of the moon and the sun on the water masses of the earth. Superimposed shorter period fluctuations and non-periodic variations may be caused by such factors as wind, atmospheric pressure, wave effects, local run-off and evaporation. Tidal ranges vary widely around the world and are affected by geographical factors. The shallow waters surrounding the British Isles have the effect of increasing the height of the tidal wave considerably; in the Severn estuary ranges can exceed 15 m. Tidal ranges in the open ocean, however, are often less than 1 m.

Predictions of extreme water levels are required in several aspects of the design of maritime structures, including overtopping, hydrostatic pressures and the level of action of waves, currents, mooring and berthing loads. Values of rates of rise and fall may also be required in relation to soil pore water pressures, flood relief valve discharge capacities and for the prediction of tidal flows.

10.2 Tidal predictions. Admiralty Tide Tables list daily predictions of times and heights of high and low waters at a selected number of standard ports [1]. These predictions are of universal application and are usually based on continuous observations of the tide over a period of at least 1 year at that standard port. The tables also list data for secondary ports, enabling predictions to be calculated from the predictions listed for standard ports.

Tide tables and almanacs are also published by harbour authorities and others. It should be noted that whereas Admiralty Tide Tables give heights referred to chart datum, i.e. the datum of soundings on the latest edition of the largest scale Admiralty chart, it is not unusual for locally produced tide tables to be based on a different datum. Certain commercial companies also offer tide prediction services.

NOTE Commercial and harbour authorities often obtain their predictions from the Institute of Oceanographic Sciences, Bidston, Birkenhead, England when information is required in a more detailed or specific format than that required for the Admiralty Tide Tables.

10.3 Meteorological effects

10.3.1 *General.* Tide predictions are based on average meteorological conditions. Variations in predicted heights will occur with changes in the meteorological regime. The main effects are described in **10.3.2** to **10.3.5**.

10.3.2 *Atmospheric pressure.* Changes in level due to variations in atmospheric pressure seldom exceed 0.3 m. However, when combined with other effects such as strong winds and intensified by geographical constrictions, this effect can be important.

10.3.3 *Wind.* A strong wind blowing on shore will tend to pile up water against the coast, resulting in a water level higher than the predicted tide height. Winds blowing along a coast tend to set up long waves which travel along the coast raising the sea level at the crest and lowering the sea level in the trough. Grouping of waves from distant storms can produce variations of mean sea level within the group, resulting in a long period, low amplitude wave travelling at the same velocity as the group, which, when it approaches the shore, can cause a higher sea level and thus allow the waves to run further inshore before they break. A combination of pressure and wind effects can cause the phenomenon known as a "storm surge".

10.3.4 *Seiche.* The passage of an intense depression may cause oscillations in sea level. The period between such waves may be anything from a few minutes to 2 h and the height from a few centimetres to 2 m to 3 m. The shape, size and depth of some harbours makes them very susceptible to such waves, increasing their height often to destructive proportions.

10.3.5 *Water levels.* Water levels in estuaries may be raised by river flow originating from surface water run-off or artesian sources. Special factors such as opening of sluice gates upstream and seasonal flow patterns should also be considered.

10.4 Tidal observations

10.4.1 *General.* Where predictions are not already available, records of the water level should be taken at the proposed site over the minimum period discussed in **10.5**. Various methods of automatic tide recording are available (see **10.4.2**). The apparatus should filter out, by means of a stilling well or other means, short period level fluctuations due to wind or wash waves.

Even in situations where predictions are already available, a permanent tide gauge is necessary to provide a reference for other data-gathering operations (see **6.2**).

10.4.2 ***Types of gauge***

10.4.2.1 *General.* The types of gauge described in **10.4.2.2** to **10.4.2.8** may be considered. With the exception of **10.4.2.2** all are automatic.

10.4.2.2 *Tide pole or staff.* The tide pole or staff is a simple graduated pole set up vertically in the water so that the changing level of the sea surface may be read on it. A tide pole should always be installed where possible, regardless of any other automatic gauge which might also be deployed. It provides a calibration and checking standard for automatic gauges.

10.4.2.3 *Float operated.* A float installed in a specially designed vertical stilling well is connected directly to a chart recorded by a wire. Careful installation of this system can be rewarded by excellent tidal records.

10.4.2.4 *Pressure operated.* The height of the tide exerts pressure on a submerged diaphragm which actuates a recording mechanism. Such systems can be connected to a surface recorder using an air-filled tube or the whole system can be a bottom-mounted unit, the latter being also used offshore. Electronically controlled pressure transducer systems are also available and are very satisfactory, particularly for long term use where a power supply is available.

10.4.2.5 *Bubbler gauge.* A constant flow of compressed air is released via a bubble chamber, to an open-ended tube set at a known depth below LAT. The pressure of the water head above the open tube will vary with the tide and thus alter the pressure in the bubble chamber. This latter pressure variation can be recorded on a drum chart.

10.4.2.6 *Electrical capacitance gauge.* An electrical capacitance gauge system uses the varying electrical capacitance of a wire mounted vertically in the intertidal zone. Care has to be taken in areas of changing salinity for corrections in the conductivity of the sea water.

10.4.2.7 *Electrical resistance gauge.* An electrical resistance gauge is a similar system to the capacitance gauge, which uses the changing resistance between two parallel electrical wires to determine water level.

10.4.2.8 *Inverted echo sounder.* An echo sounder is laid on the seabed and transmits towards the surface. A continuous record of the water level is obtained. Care should be taken to prevent movement or burial of the transducer. Ultrasonics can also be used by locating the transducer above the water level.

10.5 Tidal analysis. If a sufficiently large data base is obtained, an analysis of the records can produce the astronomic tidal harmonic constituents which may then be used to predict astronomic tidal heights for the future. These predictions assume that no other changes occur at the relevant site and their accuracy depends on the size and accuracy of the original data base. It is possible to obtain the basic harmonic constituents for a site from only 11 days' tidal data but this requires favourable circumstances. Continuous records covering 28 days to 30 days will give a much more reliable analysis and subsequent predictions. A full tidal analysis from a minimum of 1 year's continuous data can produce up to 110 tidal constituents which can be used to predict accurate astronomical tidal heights.

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Such analyses and predictions are explained in detail elsewhere [2]. It has to be stressed that predictions can only give astronomic tidal heights and that actual water levels can differ significantly due to meteorological effects (see **10.3**).

11 Water movement

11.1 General. The largest scale global water movements are the essentially permanent ocean currents. These currents are the result of the response of the ocean and atmosphere to the global distribution of solar energy and the resultant flow of energy from the tropics to the poles. The surface ocean current systems correspond quite closely to the generalized global atmospheric circulation and shift seasonally with the passage of the overhead sun. Good local descriptions of these basic water movements can be found in the Admiralty Pilots series of publications [3].

Surface ocean currents have been well documented for over a hundred years because of their significant influence on navigation and trade. The deeper currents and counter currents, however, are relatively modern discoveries following the invention of reliable oceanographic instruments, such as current meters. The use of water sample bottles and in situ temperature measuring devices, such as the reversing thermometer, also considerably aided study of the deep ocean water masses as temperature and salinity provide useful identifiers of place of origin.

In the open ocean, the wind blowing over the seas induces a current in the surface water layer which moves at an angle of approximately 45° to that of the wind, to the right in the northern hemisphere and to the left in the southern. Due to the Coriolis effect on successively deeper layers of water, the resulting net wind-induced mass movement is similarly directed to the right or left but at 90° to the direction of the original wind. In sea areas adjacent to a land mass, prevailing winds blowing parallel to the coast can produce vertical water movement known as upwelling and downwelling or sinking. Short term wind systems of the order of 2 days' or 3 days' life can induce horizontal currents which may reach a speed of 0.5 m/s (1 knot) in a prolonged strong gale. These currents are difficult to predict as they take some time to develop and may continue to run for some time after the wind has dropped. As a general rule, these wind-induced currents tend to flow at a rate of 2 % to 3 % of the speed of the wind which set them in motion. Water currents associated with storm surges can move with the speed of tide-generated currents and are thought to be due to a combination of direct wind influence, the motion of the surge generating depressions and differences in sea level associated with the atmospheric pressure gradient (see clause **10**). This latter current component is an example of a slope current caused by the response of the water surface to either increased or decreased atmospheric pressure. In theory this response is equal to 10 mm for each 1 mbar change of pressure but in temperate latitudes the atmospheric pressure distribution changes so rapidly with the passage of depressions that the predicted steady state relationship can only be attained infrequently. Slope currents can also be generated by wind set-up, wave set-up or horizontal changes in temperature or salinity, causing variations in seawater density, i.e. density currents. Geostrophic or earth-turned currents can result from a sloping sea surface, created by wind or density, and the balance between the effect of gravity and deflection due to the Coriolis effect.

Water movements of purely astronomical origin are conventionally known as tidal streams. In practice, the total water movement in an area is a combination of the generally predictable ocean currents and tidal streams, with shorter term local wind- or pressure-induced currents and, in shallow water, wave-induced currents. Tidal streams in European waters are generally of the same type as the tides, namely semi-diurnal. In other parts of the world, however, the pattern of tidal streams can be entirely unrelated to the pattern of the tides and independent predictions are necessary. Tidal streams around the British Isles are shown in pictorial form in the atlases published by the Hydrographer of the Navy and include permanent estuary currents due to river flow [4]. In the absence of alternative information, in areas of semi-diurnal tides the tidal streams experienced during a spring tidal range may be considered as being twice as strong as those occurring during a neap range.

For a fixed structure in coastal water, the parameters of water movements important for design are the speed and direction of water movement at a point and how this will be affected when the water impinges on the structure. The permeability of the structure, its resistances to motion and the possibility of sediment scour or accretion should be considered. However, for the discharge of a liquid or solid effluent and its removal from an area of dilution within it, the water movement pattern over a much wider region may have to be determined.

Water movement on a scale larger than a particular water parcel, which tends to transport the water rather than mix it, is known by the term advection, i.e. due to currents. The terms diffusion or mixing are used to describe water motions of the same size or smaller than the particular water parcel. The smallest scale motions known as molecular diffusion can generally be ignored in this context as they are inefficient compared with the larger scale motions defined as eddy or turbulent diffusion. The distinction between advection and diffusion can also be demonstrated by averaging water movement over an area. The advection will be the net movement on which is superimposed a fluctuating flow consisting of equal and opposite exchange flows, the diffusion.

11.2 Measurement of advection

11.2.1 *General.* Indirect methods of calculating water movement are generally not suitable for inshore work and direct methods of measurement should be used. These methods rely upon either measuring the velocity of water at fixed points, for example by current metering, or following the trajectory of a labelled water parcel, for example by float or dye tracking.

11.2.2 *Current meter observations*

11.2.2.1 *General.* Although this method of measurement provides a direct reading of current speed, its application to mass transport in coastal waters is restricted because observations made at one point cannot be assumed to be representative of a larger area. Other limitations are practical ones, for example low current speeds may be beneath the threshold of the instrument and transmitted vessel or moorings movement may have a significant effect on the readings.

The operational principles of the many types of meter available are described in **11.2.2.2** and **11.2.2.3**.

11.2.2.2 *Direct reading current meters.* Direct reading current meters are operated, usually manually but occasionally via a telemetry link, from a moored vessel or existing structure. Water speed is generally calculated from a count made of the number of revolutions of an impeller or Savonius rotor during an accurately measured time interval. Horizontally mounted propeller or cup assemblies have been used as the revolving element but were generally found to be more fragile. Current direction is measured by the alignment of the hull of the meter or vanes relative to an internally housed, free moving compass card. Care has to be taken in operating current meters in the presence of waves, due to transmitted boat and suspension gear movement and wave action on the meter itself. In particular, the Savonius rotor instruments tend to over-register speed under wave influence because of their omnidirectional response. Using one relatively cheap instrument, it is possible to obtain rapidly a detailed velocity profile of the water column by lowering the instrument from near surface to near bed and making measurements at several intermediate levels.

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11.2.2.3 *Recording current meters.* Recording current meters are typically between 2 and 10 times more expensive than the direct reading current meters and record the velocity internally on a medium such as magnetic tape, photographic film or solid state memory. There is thus no requirement for constant vessel and operator attendance and good moorings should enable data to be collected under worse sea conditions than would be possible by an instrument suspended from a floating craft. Traditionally the recording current meters measure speed with either an impeller or Savonius rotor and record the number of revolutions during intervals of typically 5 min, 10 min, 15 min or 30 min, hence obtaining an average water speed over each sample period. In theory the instrument is permanently aligned with the direction of water movement by one or more large vanes and the instantaneous direction held at the end of a sample interval is recorded. In practice, however, wave orbital motions will affect speed and direction measurements made by the traditional type of current meter. In order to measure such short period motions, other types of equipment have been developed which resolve the current into two perpendicular directions so that measurements of direction are independent of the orientation of the meter. Two of the most widely used are electromagnetic and acoustic current meters. The former measure changes in an induced magnetic field and the latter the apparent variations in the speed of sound due to the water movement. Generally these meters will measure the current velocity at frequent intervals, for example 1 or 10 times a second, and either record a burst of data, for example 10 min every hour, or perform a vector average to remove the orbital currents before recording.

11.2.3 *Float tracking.* This method of tracing water movement involves the introduction of readily identifiable material which moves with the water, thus directly indicating its path. The size and shape of the floating object affects its reaction to the surrounding water movement and hence whether it faithfully follows that movement. The standard float assembly consists of a vaned drogue usually made out of canvas, wood, metal or plastics, joined by thin rope or wire to a surface buoy. The length of rope determines the depth of water at which the drogue will lie and follow the water motion. The drogue is typically 0.5 m to 1.0 m long and, when suspended at a depth of say 2 m, its path will be an expression of the average horizontal movement of a water parcel between 2 m and 3 m below the surface.

A major problem with float tracking is relocation. A large surface buoy or marker such as a flag or radar reflector will be subject to windage and, in the case of a buoy, to influence from the surface water currents in which it is floating. These top marks should therefore be kept as small as possible consistent with the need for good visibility. In order to aid relocation, small radio transmitters or underwater acoustic devices can be used together with radio direction finding antennae or acoustic receivers. Location by sonar has the advantage that the surface float can be smaller. These devices are most useful in the dark and easier to find than the flashing lights previously employed. Methods have been developed to remove the windage influence from the resultant track of a float but further work is necessary before the techniques are widely adopted.

A logship is a particular design of float used to show average horizontal motion over a greater depth of water, such as 6 m to 10 m. It consists of a long thin member with identification on one end and weights at the other so that it will float vertically just under the water surface. Typical materials used are telegraph poles, tree trunks, bamboo poles or lengths of aluminium. Logships are particularly useful in navigation studies and harbour layout planning since their lengths can be adjusted to be representative of the relevant vessel draught.

Floats can also be used to obtain point-specific information by releasing them at the bows of a moored vessel and timing their passage over the known distance to the stern. Alternatively, floats attached to an anchored vessel by a known length of floating cord can be allowed to drift astern. The velocity can be calculated from the time for the float to run to the end of its tether and the current direction can be found by taking the bearing of the float. In both of these applications it is likely that the presence of the vessel will modify the current speed and direction and hence float path. A crude estimate of current velocity can be obtained from the log reading and compass heading of a vessel moored so that it is free to lie with the current.

The standard drogue float can be modified to follow surface water movement by reducing the length of rope between the drogue and the top mark so that the top of the vane assembly is just on the sea surface. To follow movement in the top 100 mm or to estimate the true path of a surface phenomenon such as a grease or oil film, however, specially designed surface floats should be used. These can consist of disc shapes or spheres which float on the surface, neutrally buoyant bottles, cards or even finely chopped vegetables. A major problem is relocation and it is usually advisable to keep the floats constantly in sight. Aerial tracking and/or photography can provide a useful method of recording float positions but is too expensive for most applications. Drogues with visible top marks can be used to indicate the approximate position of the surface floats but these may not be entirely reliable due to differences between the paths of the identified water layers.

As stated above, the floats will not follow the same turbulent flow paths as the water itself and so will only indicate the average horizontal movement. Another limitation of float track information is the portrayal of a linear picture of water movement with no areal extent. However, float tracking can give a rapid and relatively cheap impression of general water movement. Several floats can be followed at any one time with repeated releases from one position, or multiple releases with drogues set at different depths. Tidal residuals can easily be determined by tracking a float over a complete tidal cycle and determining the distance between its release and recovery positions.

Floats can be deployed unattended if a net movement is required rather than knowledge of the path taken to arrive at the end point. Commonly the floats have messages attached asking the finder to record the time and place of the float recovery and to return the information. To reduce the chance element of the recovery beach-walkers can be employed to search for the landed floats. Similar methods can be used to tag bottom currents when boat following is impossible without, for example, acoustic links. The method can only yield minimum possible water speeds, as without other valid information it has to be assumed that the float took the shortest distance between the points of release and recovery and that the float was recovered as soon as it arrived at its final location.

11.3 Measurement of diffusion

11.3.1 *General.* Much research over the past 20 years has been carried out to develop various methods of tracing water movement, in particular for the disposal of effluent into the sea where the rate and extent of dispersion together with mass transport are important criteria. An indicator was sought with physical properties similar to those of the water to be labelled. Acceptable techniques which were developed were the use of dyes, radioactive materials and bacterial organisms. These techniques all have specific advantages and limitations and a problem can often best be solved by a combination of two or even three methods but before resorting to any form of artificial indicator, consideration should be given to the use of natural tracers or those not specifically introduced, as described in **11.3.2**.

Artificial or added tracers can be used to label an existing water body such as sewage or process water discharge or to indicate water movement in an area which is being considered as a receiver for an effluent. In the first case, the existing discharge position can often be identified and the tracer can be injected at this location. For example, an effluent could be labelled by the addition of the tracer at the final pumping station or collection point and, with either natural or artificially induced mixing, the total discharge would be tagged before it reached the sea. This is particularly valuable with diffuser outlets as their efficiency in initial dilution can be estimated. The tracer can be added as a single batch, in repeated discrete batches or continuously, depending on the mode of effluent discharge, the tracing facilities and the aim of the study.

Artificial tracers are probably most useful for estimating disperson rates and mass movement for a proposed discharge at sea before construction. The timing of the release can be organized as considered above although, because of its lower cost, the repeated discrete batch technique is often used for initial studies, such as determining the optimum location and length for an outfall. Provided that the pattern of circulation is steady or repeated regularly under tidal oscillations, the dispersion from a continuous effluent source can be calculated by integration of the results obtained from repeated batch injections of tracer. Alternatively, the batch releases can be used to provide a summary of the water movement and indicate the most important or critical period for effluent discharge and a continuous release can be made over that period for confirmation. For example, the minimum time or distance of travel to the land and the corresponding effluent concentration on the shoreline may be of special interest.

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In theory the tracer can be prepared at the same density as the proposed effluent and released at the same level, usually close to seabed, but unless volume flows are also reproduced density effects can still produce misleading results. Ideally the tracer should be introduced over the large area achieved by initial dilution processes and thereafter initial dilution should be ignored. Point sources can, however, be used to give conservative estimates of maximum concentrations of effluent after dispersion.

Batch releases of dye or radioactive isotopes can best be traced by a vessel keeping them constantly under surveillance and traversing the patch at frequent intervals, recording the tracer concentration and simultaneously fixing its position. The patch can be crossed and recrossed in zig-zag fashion, commencing and terminating when the detectors show zero tracer concentration above background. It should be remembered with dye tracing that the boundaries thus recorded will be wider than those estimated visually since the sensitivity of the measuring instruments is greater than that of the eye. Continuous releases cannot generally be sufficiently accurately followed by repeated crossing of the dye patch because of their wide spread, although an attempt should be made to outline plume dimensions. Alternatively, or in addition, a regular grid within the area of expected travel can be sampled or if sufficient craft are available they can moor at separate locations and measure the change of concentration with time at these fixed points. This technique can be extended by recovering samples for subsequent analysis if there are insufficient measuring instruments available. Grid sampling is the only method for use with bacterial tracing, as lengthy laboratory procedures are required before the number of organisms present can be determined.

Measurements of dye concentration and radiation levels can easily be made at any depth, by either lowering the detector or raising a sample of water. Alternatively, towed multi-level arrays can provide simultaneous measurement of dye concentration or radiation levels at several depths. Samples for bacterial testing can also be collected at depth but extreme care has to be taken to avoid contamination. Sealed sterilized bottles are lowered to the required depth, opened to take the sample and resealed to avoid transfer of water to or from the bottle on the return journey.

Although many studies of water movement have been carried out and documented, there are insufficient data available at present to provide a detailed model of either water mixing in coastal areas or more particularly of effluent mixing and dispersion, and this has hindered the cross-transfer and application of documented survey results. Neither is there sufficient information available to define exactly the role of the various physical parameters in determining the mixing rates. This has necessitated the continuation of the practice of carrying out specific site studies, particularly in areas of complicated bathymetry and current regime.

11.3.2 *Natural tracers.* Tracers are defined here as natural or normally occurring if they can be found in an area before any phase of new construction or site investigation is undertaken. For example, a river discharging into an area would create a particular salinity distribution and the freshwater plume could be traced rapidly using an in situ salinometer. Effluent from existing constructions may contain elements whose redistribution could be used as indicators of local water movement patterns. It may also be possible to evaluate distribution patterns of turbidity, silica and dissolved oxygen or the distribution of certain biological matter such as species of plankton, fish eggs or bacteria.

The discharge of domestic sewage presents an example of an existing controlled release of effluent which can be monitored to demonstrate present dispersion patterns or to assess the capacity of the receiving waters for further discharge. However, problems exist over the choice of indicator. Biochemical oxygen demand, dissolved oxygen concentration and nutrient load, for example, nitrogen or phosphorus concentrations, have been considered as tracers but as a generalization a bacteriological indicator such as *Escherichia coil* appears to be the most satisfactory for temperate latitudes. *E. coli* organisms are natural inhabitants of the intestine and are present in large numbers in domestic sewage, approximately 10^6 organisms per millilitre of untreated sewage being a typical count in the British Isles. Unfortunately, *E. coli* is not a specific identifier of a particular discharge as it may enter the sea via rivers, seabirds, bathers, from passing vessels and storm discharges.

The discharge of a heated effluent, such as cooling water from a power station, can be a useful indicator of local water movement and can be monitored easily at various depths with shipborne equipment. The distribution of surface water temperature can also be determined from the air using thermal imagery. The change in dimensions of the thermal plume would however be dependent on the difference between discharge and ambient temperatures, as well as the physical mixing and movement in the water and therefore may not be immediately suitable for use in modelling unheated effluent.

11.3.3 *Dye tracing.* The use of a colouring agent to label the seawater would appear to approach the ideal requirement for an indicator of water movement and in practice the technique has many advantages. In the simplest case, dye can be used as a visual indicator of the surface water movement in a similar way to surface floats, without the disadvantages of windage. Simple colouring agents such as vegetable dyes, inks or even emulsion paint can be released in the sea and tracked by recording the position of the mid-point or the perimeter at frequent intervals. Visual dye tracking can be carried out on a number of patches at any one time. This method does not give any indication of vertical mixing and can only be used whilst the patch can be seen and this would vary considerably with atmospheric and sea state conditions as well as the rate of dispersion. Aerial photographs can be of great assistance both in locating the axis of maximum dye concentration and as a permanent record of the visible extent of the dye.

More information can be obtained from dye tracing if the dye concentration can be quantified. A colorimeter can be used to measure the depth of colour in water flowing through the instrument on board a survey vessel, thus giving a continuous picture of the dilution of the dye. However, if a fluorescent dye or tracer is used, then a fluorimeter, which is more efficient in this application, can provide greater accuracy at low concentrations than direct colour measurement.

The fluorimeters used in dye tracing studies generally employ optical filters to reduce the measurement of fluorescence from materials other than the chosen tracer, but if the fluorescence in the water from these materials lies within the filter wavelengths it cannot be removed and is known as the background level. This level is generally in the region of 1 part of fluorescent equivalent in 10^9 to 10^{11} parts of water and varies with the chemical and physical nature of the water and with the wavelengths at which the dye absorbs and emits light. The characteristics of the dye can also limit the upper level of dye concentration measurement by direct flow fluorimetry and accurate determinations can only be made if the sample is first diluted, thus precluding direct flow work.

The fluorescence of various dyes is temperature, salinity or pH dependent and the choice of dye to be used should be made with care. Some dyes such as fluorescein, suffer the disadvantage of being susceptible to photochemical degradation, whilst others such as rhodamine B are readily absorbed by silt particles and some suspended matter. The chemical constituents of dyes should also be considered both for operator hazards and for possible detrimental environmental effects. Results of tests on numerous fluorescent dyes can be found elsewhere [5].

The quantity of dye to be released will be determined by various factors such as upper and lower limits of detectability, expected rate of dilution, itself influenced by currents and sea state, and the desired time scale of the tracing. For example, 2 kg or 3 kg may be sufficient for short coastal experiments on dilution immediately after release, whereas 5 000 kg was used in a central North Sea exercise lasting 23 days. [6].

11.3.4 *Radioactive tracers.* Procedures for radioactive isotope investigations follow a similar pattern to those for fluorescent tracers in that a controlled release is followed by in situ measurements using sensitive radiation detectors operating directly in the seawater. Readings can be made in the vertical plane rapidly and easily by lowering the detector and either recording a total count for the water column or by halting the detector at various intermediate depths to record radiation levels.

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Radioactive isotopes are non-conservative tracers, that is to say the tracer concentration decreases with time, even in the absence of any dilution effect. Measurements made after the release have to be corrected by the decay factor to isolate the decrease in radiation level produced by physical dilution. Safety and practicability together with the study aims dictate the quantity of isotope which can be released. Isotopes with relatively short half-lives have to be chosen if experiments are to be repeated at frequent intervals in one area since accumulation and contamination may otherwise result. A typical water-soluble isotope used for short term disperson studies is Bromine-82 which has a half-life of 36 h. Other isotopes such as Gold-198 with a half-life of 2.7 days, Phosphorus-32 with a half-life of 14.5 days and Silver-110 with a half-life of 253 days, have been used for longer studies.

Certain special precautions and statutory regulations have to be observed whenever radioactive materials are used and studies involving isotope tracers should be supervised by trained and experienced personnel.

11.3.5 *Bacterial tracers.* Added bacteria can be usefully employed as sensitive tracers of water movement since single organisms can be recovered from water samples taken after considerable dilution. The sensitivity of the technique can be increased in the laboratory examination by the use of membrane filtration, which allows relatively large sample volumes to be examined.

Pure cultures of tracer bacteria are used to model the behaviour of naturally occurring organisms and in this context they have been used successfully in studies of the discharge or proposed discharge of domestic sewage. The intestinal organism *Escherichia coli* is often taken as an indicator of sewage pollution and the possible presence of harmful organisms (pathogens) such as *Salmonella* species. *E. coli* have a rapid mortality rate in seawater and this, together with the usually strong mixture experienced around the British coasts, explains in part why the British Isles are not surrounded by a sea of sewage. The mortality rate is usually expressed by the time which elapses for 90 % of the original bacteria to die and is called the t_{90} value. This time is dependent on many local factors including the length of time the organisms have been in the sea but it appears that the intensity and quantity of solar radiation are of major importance. Measured values of t_{90} in reasonably clear water typically lie in the range 1.5 h to 9.0 h and a t_{90} of 4 h is often quoted as a generalization for daylight hours. In turbid waters a value of 40 h may be more appropriate.

As noted in **11.3.2**, *E. coli* cannot be used as a specific identifier in existing sewage discharges as it would be impossible to identify the source of those recovered. A harmless, easily identifiable organism which is not indigenous to the area is therefore desired, preferably with mortality characteristics similar to those of *E. coli*. The organisms *Serratia indica* and *Serratia marcescens* with t_{90} of the order of 50 % to 100 % greater than that of *E. coli* have been used as such tracers. *Serratia* are pigmented saprophytic organisms producing bright red colonies which can be readily enumerated. *Serratia* can be cultured either for batch release or on a longer-term basis for continuous discharge and the number of organisms which can be prepared for release is typically of the order 10^{12} per litre. Spores of a variety of *Bacillus subtilis* have been used for longer term experiments in conjunction with the radioactive isotope Bromine-82. However, it was noted that the spores could be detected in lower concentrations than could the bromine because of the greater sensitivity of the bacterial method.

Successful studies of water movement have also been made with bacteriophage, viruses which infect bacteria. Any particular bacteriophage will act only on one type of bacterium or closely related species and this parasitic behaviour forms the basis of the laboratory examination of samples. For example, a bacteriophage specific to *Aerobacter aerogenes*, with a t_{90} of around 15 days, has been successfully used in estuarine waters to study the mixing of two river systems in order to locate a suitable site for a sewage effluent discharge [7].

The main disadvantages of bacterial tracers are on the practical side of the survey. Because it is necessary to collect discrete water samples, unless a secondary tracer is employed simultaneously there is no definite indication as to whether the samples will contain any bacterial tracer. Considerable care has to be taken to avoid contamination of the samples both during collection, handling and testing, and a suitable laboratory is essential together with personnel trained in microbiology. There is always a substantial interval between the time when the sample is taken and when the results are available and this compares unfavourably with the rapid dye and radioactivity tracing. For example, the results of tests for *E. coli* would not be available for 18 h and those for *S. indica* not for approximately 44 h. Despite the delay in obtaining results, the advantages of modelling bacteria with a similarly behaving tracer are obvious.

11.4 Presentation and analysis of field data. The effort and cost of data collection can be significant. It is important that the method of interpretation is considered during the design of the field work, to avoid spending time and money on data which are irrelevant or unnecessary. In particular, the method of position fixing (see **6.3**) and its accuracy relative to the requirements of the tracing method should be considered. Interpretation of the data can range from visual intuitive examination of a few results to sophisticated computation of a large quantity of information. It is important to remember the aims and ultimate objectives of the study both when considering the method of data presentation and during interpretation.

Charts play a major role in the presentation of water movement information irrespective of which method of tracing is adopted. At the simplest level, the chart will show only the position of measurement. A pictorial representation of the measured data can be a useful addition, such as a scatter or central vector diagram for current observations or a record of tracer concentration. Charts are obviously essential for presenting data such as float tracks or for showing vessel tracks during dye traverses. Several spot readings can be contoured to show similarities and trends and this can be a useful technique for the discussion of radiation levels or bacterial counts. A summary chart showing water movement in broad terms as derived by the survey can be a useful aid to understanding the current regime of the area. Additional information on other variable environmental conditions such as sea state, wind and tidal levels should be readily available for reference.

12 Waves

Exposure to wave attack can have a profound influence on the selection of sites for maritime structures and the consideration of designs and construction methods requires detailed knowledge and understanding of the wave activity and persistence, in average as well as extreme conditions.

Detailed consideration of wave form, generation, recording, analysis and prediction is included in section 4.

13 Water quality

13.1 General. The interaction between water quality and maritime structures can be considered from two opposite view points. The effect of water quality on the safe and efficient functioning of the structure should be evaluated and this would require the gathering of the usual data on temperature, corrosive elements, suspended solids, marine growth, etc., but the influence of the structure on the water quality and other features of the surrounding environment should also be considered. This is particularly important when deciding on construction methods, material sources and operational details. In addition, the water around a coastal structure may be of considerable economic importance and the effects of construction on fish or shellfish stocks, potential freshwater or mineral extraction and recreation activities should be considered.

13.2 Temperature

13.2.1 *General.* In coastal regions there is generally a well defined seasonal temperature variation, although throughout the year the water column tends to be isothermal due to strong turbulent mixing. Significant stratification may exist, however, in areas where there is a thermal effluent or in estuaries with high freshwater discharge.

Temperature variation has effects as follows:

a) *Ice formation.* Sea icing may become a problem when the water temperatures fall below – 2 °C. However, floating ice masses formed elsewhere can survive when the water is considerably warmer. When considering icing on a structure, wind strength and air temperature are of prime importance; however icing is unlikely to occur until minimum sea surface temperatures fall below 6 °C.

b) *Corrosion.* Higher temperatures will increase the rate of iron oxide formation and may have a significant effect on bacterial corrosion.

c) *Marine growth.* The rate of encrustation on a structure and the species of organism present will depend on the temperature of the environment, higher temperatures promoting more vigorous growth.

d) *Effluent dispersion.* The density of sea water is a function of its temperature and salinity and is a fundamental parameter to be considered when modelling the behaviour of an effluent immediately after release.

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13.2.2 *Measurement.* Different applications require different methods of measurement. Surface temperature variations over a large area can be determined using infrared techniques from either an aircraft or satellite. Variation at a point can be monitored by comparison of multiple passes of the remote sensing apparatus. On a smaller scale or where depth profiles are required, continuous or repetitive measurements are taken using thermal sensors such as thermistors, resistance bulbs, thermocouples and mercury-in-glass thermometers.

13.3 Chemistry. The chemistry of the water should be determined at an early stage of the site investigations, with particular attention being paid to potentially corrosive elements such as chloride and sulphate ions (see also **49.12**). Coastal water is generally fully saturated with oxygen at the surface but if there is little vertical mixing the oxygen content will decrease with depth. Under normal circumstances this decrease is unlikely to have a significant effect until anaerobic conditions are reached but the local distribution and seasonal variation should be considered when siting outfalls to discharge effluents which could act as reducing agents.

The important chemical parameters are usually analysed directly or measured with selective ion electrodes, either in the field or from specific collected and stored samples in the laboratory.

13.4 Turbidity. Turbidity is generally caused by suspended clay or silt particles, dispersed organics and micro-organisms. A lower water temperature will increase the amount of sediment that can be transported in suspension due to the viscosity change.

Turbidity should be considered in harbour design, with special reference to sediment movement and siltation studies and when abstracting water for commercial purposes, particularly with regard to wear on pumps and blockages.

The most rapid changes in turbidity generally occur during dredging operations and the consequences should be borne in mind during the planning of the initial site preparation and maintenance dredging. Mixtures of fine sand and muds with organic matter tend to support a greater variety and number of benthic, i.e. bottom-living, organisms than coarse, clean, unstable sand, gravels or soft muds and so dredging in muddy sand will have a greater ecological impact. Detailed consideration should also be given to the area of spoil disposal, not only for the economic disadvantage of dispersion with possible redeposition back in the dredged area but also for the likelihood of deleterious effects to the resident flora and fauna.

13.5 Marine life

13.5.1 *Effects of marine life on structures.* Many forms of marine organisms including algae, molluscs, crustacea and others will attach themselves to a maritime structure. The organisms can cause blockage of intake and discharge pipes, impose or increase mechanical stresses (see **47.2.2**), accelerate degradation, retard flow or simply impede inspections for maintenance or certification purposes [8]. Methods of controlling marine growth include the use of antifouling paints and mechanical removal by hand scraping or water or air jetting. As well as surface-attaching species, structures with timber elements may be affected by boring organisms (see **60.2**). The presence of molluscs on the surface of a steel structure can inhibit corrosion and their removal is not always beneficial (see **68.1**).

13.5.2 *Effects of structures on marine life.* Shore-connected construction may result in long term changes in the maritime environment, but in nearly all cases the significant pollution aspects of these structures are of short duration, limited to the construction and installation activities. The more serious potential for pollution associated with these structures generally arises from their use and operation rather than from their presence in the maritime environment [9].

If a major development project is planned, then an environmental impact assessment may eventually be necessary which should include the environmental, social and economic effects of the proposed development. Methods for evaluating these effects in an impact assessment are still fairly rudimentary and most make use of value judgements. For details of these methods specialist texts should be consulted [10, 11].

13.6 Pollution

13.6.1 *General.* If it is likely that there will be a discharge of potential pollutants from a maritime structure, either during normal operation or due to accidents, a baseline survey should be carried out to define conditions before discharge and a monitoring programme established to identify changes in the environment during discharge. Baseline surveys generally include data on the physical environment and analyses to determine existing levels of particular chemicals in sediments, water, air and marine organisms. The monitoring programme may repeat the same analyses at regular intervals, or following particular occurrences likely to cause changes, such as accidental spillages.

13.6.2 *Examples of discharges*

13.6.2.1 *Oil.* Operational spillages can be minimized by taking adequate precautions, in particular the training of competent operators. In the case of accidental spillages, the authority responsible should maintain adequate stocks of cleansing equipment and have detailed knowledge of water movement in the local area so that personnel and supplies can be efficiently deployed.

13.6.2.2 *Trade effluent.* The discharges and their effects on the environment should be monitored and pollution kept to an acceptable level.

13.6.2.3 *Domestic effluent.* The pollution from sewage is predominantly bacteriological and visual. Pre-construction surveys should be carried out to study stratification, water movement and bacterial mortality and hence to decide on a suitable outfall location.

13.6.2.4 *Heated water discharge.* The effect of the increased temperature on marine life can be considerable, increasing the metabolic rate and decreasing the species diversity.

14 Sediment transport

14.1 General. In any operation involving the alteration of the inshore hydrodynamic regime, the subsequent effects on sediment movement have to be considered. It is in the inshore zone that energy generated at sea is transmitted to the land in the form of waves, tidal currents, etc. and it is the sediments, rocks and man-made structures of the area that are responsible for attenuating this energy. The sediment of the littoral and sublittoral zones is moulded into a topography and as long as the balance of forces is maintained, the system will be in dynamic equilibrium. If one of the system components is altered the remainder of the system will have to adjust in order to achieve a new dynamic equilibrium. Such is the delicate balance of nature that what may appear to be a minor alteration can bring about large scale changes to the system as a whole often with expensive and dangerous consequences.

The natural parameters that define the rate and direction of sediment transport are the prevailing currents, both tide and wind generated, waves, bathymetry and the properties of the seabed or beach sediment. Currents and waves are the agents responsible for entraining, transporting and depositing sediment. Bathymetry imposes constraints on currents and waves and hence on erosion and deposition. The size distribution of the indigenous sediment generally reflects the competence of the transporting agents, subject to the availability of material.

A general appreciation of sediment transport in an area can be gained by studying old charts and photographs, including aerial views, and in the case of beaches, by carrying out a preliminary inspection. Historic information can then be compared with modern details and changes of depth and apparent movement of features such as offshore sand bars can be studied to indicate the transport of sediment. In a preliminary beach inspection, features such as spits and accretion or scour against existing structures will be indicative of the direction of the drift and intensity of sediment transport.

14.2 Sediment transport in currents

14.2.1 *General.* When a current flows over a bed of erodible sediment a certain minimum or threshold bed shear or current is required to initiate movement. For flat beds composed of fine or medium sand (grain diameter 0.1 mm to 0.6 mm) the threshold current appears to be about 0.15 m/s measured at 0.3 m above the bed, although that required to move silts (size < 0.06 mm) and gravels (size > 1.0 mm) has been found to be greater.

When the current exceeds the threshold value, the bed begins to erode, the displaced particles being carried over the bottom by sliding, rolling or saltation, i.e. bed transport, or by being lifted into the body of the water by turbulence, i.e. suspended transport. For fine-grained sediments, both modes are usually present at the same time and it is difficult to distinguish the boundary and the level at which suspended transport is initiated. It is generally considered that suspension becomes appreciable when $v_s/v_* \approx 1$ where v_s is the average fall velocity of the particles in still water and the friction velocity $v_* = \sqrt{(\tau_0/\rho)}$ where τ_0 is the shear stress exerted by the water on the bed and ρ is the water density.

Generally beds subject to erosion form small scale ripples and larger scale dunes which, while displaying a certain average wave length and height, vary in size, have flat upstream slopes and steep downstream slopes, are usually short crested and migrate slowly in the direction of the current.

14.2.2 *Measuring suspended sediment load*

14.2.2.1 *General.* Most satisfactory results are achieved if sediment transport is regarded in terms of sediment flux. This is usually measured directly in the field at several stations at frequent intervals, usually half-hourly, on both neap and spring tides. The flux is obtained by the integration of the product of the water velocity and the concentration of suspended solids over the depth.

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Devices for measuring suspended load are described in **14.2.2.2** to **14.2.2.4**. For the samplers described in **14.2.2.2** and **14.2.2.3**, the grain size distribution is determined by standard sieving, settling column or Coulter counter techniques and the subsequent numerical integration can then be performed.

14.2.2.2 *Depth integrating sampler.* This collects and accumulates a sample as it is lowered to the seabed and lifted back again. The sampler should be moved at a uniform rate in a given direction, but not necessarily at equal rates in both directions. This equipment is restricted in use to shallow depths by its capacity to store the water sample.

14.2.2.3 *Point integrating sampler.* This operates in almost the same manner, except that it has an inlet valve which can be opened or closed as required. Thus, it is possible in deep water, to point sample at known depth intervals and then integrate over the entire depth.

14.2.2.4 *Light extinction meter.* This is commonly, but incorrectly, called a siltmeter. It incorporates a light source and a photocell and variations of suspended solids concentration cause fluctuation in the amount of light reaching the photocell, which is connected to a meter. A calibration curve should be established so that meter readings can be converted to concentrations of solids for the particular material in suspension. Since it is not usually practical, because of the bed features, to measure closer than about 0.3 m above the seabed, where the concentration is high, an extrapolation to lower heights above the bed is often made, based on theoretical arguments.

14.2.3 *Measuring bed transport.* For fine sand and silt, bed transport is usually small in comparison with suspended transport, but may become relatively appreciable with increasing grain size. There are primarily two methods for studying the movement of bed load; those which use laboratory-derived bed load formulae (see **14.2.4**) and those which rely on field measurements (see **14.2.5**). Direction and rate of bed transport can be estimated by employing fluorescent and radioactive tracers and the bed load can be sampled by using sediment traps or specialized equipment such as the Arnhem or Dutch pressure difference bed load samplers. The practical difficulties of using these traps and samplers in the field and consideration of their efficiency has reduced their popularity, with a corresponding increase in the use of bed load formulae.

14.2.4 *Bed load formulae.* The main types of bed load formulae consider transport rates from relationships of shear stresses, flow and statistical considerations of lift forces. All equations rely to some extent on experimental data from flume studies to determine their various integral coefficients. For details of these equations and their applications, reference should be made to specialist texts [13].

14.2.5 ***Field tracer studies***

14.2.5.1 *General.* These generally make use of coloured tracers, radioactive tracers or, occasionally, heavy mineral tracers. Radioactive tracing has a notable advantage over the use of coloured materials as measurements can be made rapidly in situ without the need to retrieve a sample or count the grains. However, some vertical cores should be taken whichever technique is used, to determine the depth of movement of bed material and hence quantify the total bed transport. Cores may also clarify the confusion which can be generated by the ability of the radioactive technique to detect buried, labelled particles and the resulting uncertainty as to whether a low level of radiation is due to a low source on the surface or a higher level from a buried source.

Problems of depositing the labelled material are common to both techniques as in theory it should be fully integrated with the bed throughout the movable depth. An initial tidal bias can also be introduced depending on the time of release, although this can in part be overcome by injecting half the tracer at high water slack and the other half at low water slack. These problems can both be minimized by allowing time for the labelled sediment to mix thoroughly with the seabed and by ignoring the first few days' results.

All tracing experiments indicate only the movement that has occurred during the period of the experiment. For radioactive tracers that have a relatively short half-life this may only be a period of a few weeks and coloured sand may also be dispersed after a short time. Unless weather and wave conditions are similar to those of the long term average, non-typical results may be produced. Generally the direction of sediment movement is much easier to determine than its magnitude which may be considerably in error, depending on circumstances. It should be noted that the techniques described can also be employed to study suspended sediment transport of non-cohesive grains. Considerations of flocculation and aggregated size confuse and hence generally restrict the use of radioactive tracers for silt and clay sizes.

14.2.5.2 *Colouted tracers.* Coloured tracers involve the coating of non-cohesive sediments with either a distinguishing coloured paint or a fluorescent substance. Indigenous sediment may be used for this purpose, but where it is not practicable to treat a large quantity of local sand, material of a similar density and size distribution may be substituded, provided that it will respond to the hydraulic processes in exactly the same way as the indigenous sediment.

The quantity of labelled material to be used will depend on the proposed duration of the test and the expected strength of the waves and currents but generally it has been found, in the case of long-shore drift investigations, that 50 kg to 100 kg per week of study is an appropriate amount.

The sampling routine depends on the environment being studied. Samples should be taken at a maximum spacing of 30 m on each side of the injection point and the basic requirement is that a surface sample of uniform area is retrieved. The method of achieving this will depend on the grain size. For instance, with sand-size material, a simple method is to press a grease-coated card into the sediment. In the laboratory, the grains are counted by eye either in natural light or, in the case of the fluorescent grains, under an ultra-violet light, to yield the concentration of tagged grains. The distribution of labelled sediment can then be plotted on a chart and contoured to show the displacement from the injection point.

14.2.5.3 *Radioactive tracers.* The usual radioactive tracer technique involves the preparation and irradiation of artificial sediment which has hydraulic properties similar to those of the bed sediment and which is then used to label the local material. Isotopes such as Barium-140, Lanthanum-140 or Scandium-46 are often employed. Care has to be taken when choosing an isotope to ensure that it will have no harmful effects on the environment and to match the half-life of the isotope to the duration of the experiment and the levels at which the isotope can be detected. It is generally not practical to label large particles in the same manner, as the acceptable levels of radiation for release would correspond to relatively few individual grains with reduced likelihood of relocation. The Hydraulics Research Station, Wallingford, Berkshire, however, has recently developed a method of labelling shingle which involves the sealing of the isotope, in the form of a short length of activated silver wire, into a hole in the pebble. With the development of a wide-sweep detector, it has proved possible to relocate a significant proportion of the pebbles and a typical quantity of 1 000 pebbles has proved adequate. The labelled sediment can then be traced with a sensitive radiation detector and sediment movement can be described by plotting and contouring radiation levels.

14.2.5.4 *Heavy minerals.* Heavy minerals represent the accessory and varietal minerals of igneous and metamorphic rocks and are operationally defined as those minerals which have a specific gravity greater than 2.85. They probably account for no more than 0.1 % to 0.5 % of terrigenous sedimentary rocks and providing there is a source rock relatively near to the study area, their distribution and relative quantities can be useful in indicating the general direction of sediment movement. If sediment cores are taken, the vertical distribution of the heavy minerals can also be determined and this could yield much useful information on past sediment movement. Heavy mineral analysis can be expensive and great care has to be exercised in the selection of indicator minerals with due regard being taken of the complexity and potentially long time scale of geological processes.

14.3 Sediment transport with waves

14.3.1 *Movement due to wave-generated currents.* Waves alone can initiate movement of sediment when the maximum orbit velocity they induce near the bed exceeds about 0.15 m/s, as for currents alone. For fine sand, the particles rock backwards and forwards and form ripples with crests parallel to the wave crests. These are typically more symmetrical and periodic than those formed under steady currents (see **14.2.1**). As the orbit velocity increases, the ripples become steeper and the sand is taken into suspension by eddies shed from the crests. In the presence of tidally induced currents, it appears that the oscillatory motions caused by waves act as an additional stirring agent at the bed and the concentration of suspended sediment is increased [14, 15].

Large eddy effects can be caused by wave-induced currents associated with diffraction at the sheltered side of the breakwaters (see **29.2.4**) and they are thought to be capable of redistributing sediment within harbours.

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14.3.2 *Littoral drift.* In the longshore direction, currents can be induced by the tangential momentum of waves breaking at an angle to the shoreline. Such currents are restricted mainly to the area inshore of the breaker zone. Typically their strengths are low which suggests low transport rates. However, the breaking waves create considerable bottom turbulence which lifts material up into the longshore current where it can be transported some distance, resulting in a high net sediment movement. Several empirical formulae have been proposed relating longshore drift of sediment to the breaker height and specialist texts should be consulted [16]. Littoral drift can also be measured using the tracer techniques discussed in **14.2.5**.

Littoral drift can be an important agent in the shaping of the local coastline and requires careful study whenever construction is contemplated which may disturb the dynamic equilibrium of the local drift regime (see **14.1** and **14.4** to **14.6**).

14.3.3 *Rip currents.* Generally there appears to be little convection of water between the offshore area and the breaker or surf zone, but along some shorelines rip currents, consisting of concentrated jets that carry water seaward through the breaker zone, can occur. It is thought that these may be caused by the interaction of wave set-up and edge waves (see **24.3**). Rip currents with speeds exceeding 1.0 m/s have been reported and it is noteworthy that they are the only currents easily capable of transporting sediment seawards. At present it is not possible to predict the strength or spacing of rip currents and there is insufficient information to indicate quantitatively how important they are as sediment transporting agents in the context of net movement.

14.4 Accretion and scour

14.4.1 *General.* Whilst the above techniques give an estimate of the rate and direction of sediment transport in an area, in many engineering problems it is desirable to produce a direct estimate of the amount of accretion or scour. Such a situation frequently arises in the planning of navigation channels. It should be noted that predictions of this type are uncertain and an estimate which is within 25 % of the final quantity may be considered as good.

14.4.2 *Trial trench method.* The most obvious method of estimating accretion or scour rates is by trial dredge. A series of trial holes are dug on the line of the proposed channel and the rate of infill determined from successive bathymetric surveys. Such trial holes should be approximately rectangular in plan and orientated with the length parallel to the channel. Ideally the hole should be dug to the same depth and breadth as the proposed channel and the length sufficient to avoid end effects, i.e. accretion at the ends which is not typical of the channel as a whole. Usually a length to breadth ratio of about 10 is sufficient, the infill rate being determined from surveys taken near the mid-section of the trench.

14.4.3 *Accretion/scour plugs.* This method is suitable for use in the intertidal zone at each site of interest but can be used in the nearshore zone if divers are employed. The technique involves the removal of several cores of sediment and replacement by an artificial distinguishable material with similar erosion properties. The amount of accretion or scour can then be estimated by recovering the core and surrounding sediment and measuring directly the amount of sediment deposited or eroded.

14.5 Bed-form migration. The direction of sediment transport and a qualitative appreciation of the rate of transport can also be gained by observing the migration of current-produced bed forms. Bed-form migration appears to be controlled by the time-velocity assymetry of bottom tide-generated currents and the sediment grain size. The method of study will depend on the required resolution. The most accurate measurements can be made using photogrammetric methods which, under favourable conditions, can be resolved to a scale of millimetres with complete areal cover. Precise levelling by divers, coupled with a tracer experiment, can give point measurements accurate to a scale of centimetres but this technique gives lower overall accuracies dependent on the line spacing. Conventional time-lapse underwater photography can also offer reasonable point or line accuracies under good visibility. Remote techniques such as echo-sounding and side-scan sonar surveys offer less resolution but, bearing in mind the qualitative nature of the results, are probably more practicable. These techniques can produce good results with the larger bed features such as sand waves.

14.6 Models. For certain operations, such as in the construction of breakwaters or causeways, it may be desirable to design a physical or mathematical model of the system in order to predict any changes to the sediment transport paths for various proposed schemes. With a physical model, the objective is to produce an accurately scaled model of the prototype. This is achieved by undertaking field observations and measurements which can be related to the model by a dimensional study of the prototype characteristics. The model is proved when it accurately reproduces known past and existing conditions. It can then be used to predict changes in the system brought about by future alterations to the environment.

A mathematical model operates by defining the processes and responses of the system in terms of mathematical relationships and, by necessity of the complexity of the model, is run on a computer. Validity is tested, as with the physical model, and prediction is performed by adjusting the relevant variables such as boundary conditions, wave height and current strength.

Both types of model have virtues and inherent limitations. The biggest disadvantage of the physical model concerns distortion resulting from the inability to scale certain system characteristics. For example, it is difficult to scale sediment movement since, if the sediment size is scaled, the viscous forces on the grains become relatively too large and the grains become so small that cohesive forces predominate. On the other hand a mathematical model may be an over-simplification or even misinterpretation of reality. Knowledge of the mechanisms of sediment transport and hence the governing equations is as yet incomplete; thus a mathematical model of sediment transport has to be an approximation. The best solution to the problems of modelling is probably a combination of the two techniques.

The accuracy of model predictions, whether mathematical, physical or a combination of both, varies widely, depending on both the complexity of the site situation, and the accuracy and quality of field data used in setting up and determining the validity of the models. Models are often indispensable for determining which of several proposals is the most suitable for site application as they can give a guide to the probable long term changes in the bathymetry. They are less able to determine how quickly such changes will take place and hence the rates of accretion or scour.

Section 3. Operational considerations

15 General

This section gives guidance on the general aspects of design life of structures, ship data, navigation in approach channels, ship handling in harbours and other operational considerations.

Many of the operational requirements of a maritime structure are specific to the particular function or functions of the individual structure and guidance on these aspects will be found in subsequent Parts of this code. However, in all projects, close liaison should be maintained with the client or operator so that the maximum operational benefits can be obtained in terms of safe and economic functioning of the finished installation. The following subjects should be considered for possible individual study in each case:

a) numbers, types, sizes and shapes of present and/or expected vessels;

b) provision of tugs, navigational aids and marine traffic control;

c) pilotage;

d) berthing and mooring patterns, practices, systems and load measurements;

e) requirements of cargo handling, roll-on roll-off (ro-ro) traffic, storage and other activities, including the need or otherwise to allow for future change, or flexibility in operational usage;

f) regulations affecting any of the above operations.

16 Design life of structures

The design life of a structure is taken to be its intended useful life and will depend on the purpose for which it is required. The choice of the design life is a matter to be decided in relation to each project since changes in circumstances and operational practices can make the structure redundant or in need of substantial reconstruction before the end of its physical life. However, the following values may be regarded as minimum requirements:

a) quay walls: 60 years;

b) open jetties: 45 years;

c) superstructure works: 30 years;

d) dry docks: 45 years;

e) shore protection works and breakwaters: 60 years;

f) flood protection works: 100 years.

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It should be noted that the design life will not necessarily be the same as the return period of the design condition since such an equality implies a 63 % chance of the design condition occurring during the design life (see **21.4**). This may not be operationally acceptable for some structures, particularly major protection works.

The design life is significant when assessing:

1) time-dependent factors running against the security of the structure such as fatigue loading, corrosion, marine growth and soil strength reductions;

2) probability levels for limit state design and for design condition return periods;

3) economic feasibility of the project and future developments.

In view of the variable and often unpredictable character of the forces to which maritime structures are subjected, it is frequently unrealistic to expect substantial cost savings to result from attempting to design them for short lives. Generally, greater overall economy will be achieved by choosing simple robust concepts and appropriate reliable construction procedures.

17 Ship data

17.1 *General.* Where possible, details and dimensions should be obtained from the relevant authorities, owners and operators for the actual vessels to be accommodated and those likely in the anticipated lifetime of the structure.

Vessel characteristics which should be considered include type, size and shape, ship handling requirements, cargo or passenger handling requirements and vessel servicing requirements.

Characteristic dimensions of passenger vessels, naval vessels and vessels of less than 20 000 t displacement vary considerably according to function, age and operational region. Approximate dimensions of the larger bulk carriers and container ships are given in **17.3**. These values are approximations and should only be used for preliminary planning purposes. General guidance on vessel dimensions may also be obtained from the registration particulars [17].

Ship handling considerations are discussed in clauses **18** and **19**. Guidance on cargo handling and servicing requirements is to be found in subsequent Parts of this code.

17.2 Tonnage and displacement. While naval vessels are customarily described by displacement tonnage, the size of other vessels is frequently quoted in terms of gross registered tonnage and deadweight tonnage. The latter values are significant for registration and in assessing the carrying capacity of the vessel. However, for computing berthing energies (see clause **41**) and other hydrodynamic calculations the displacement of the vessel is required.

For the purposes of preliminary planning the following relationships may be used to obtain displacement from gross registered tonnage or deadweight tonnage. These values are approximations and should not be used for detailed design unless confirmed by the actual vessel characteristics.

a) Fishing boats:

small: grt × (2.5 to 2.0);

large: grt × (2.0 to 1.5).

b) General cargo:

grt × 2.0; *or*

dwt × (1.6 to 1.4).

c) Passenger liners: grt × 1.1.

d) Container ships: dwt × 1.4.

e) Bulk carriers: dwt × (1.3 to 1.2).

Where two values are quoted in the above relationships, the first approximates to the smaller vessels of a given type.

17.3 Typical container ship and bulk carrier dimensions. For the purposes of preliminary planning, Figure 1(a) and Figure 1(b) may be used to obtain typical values of container ship and bulk carrier dimensions respectively. These values are approximations and should not be used for detailed design unless confirmed by actual vessel characteristics.

18 Navigation in approach channels

18.1 General. From the operational view point ideal accessibility to a harbour implies:

a) a straight, wide approach channel, the direction of which coincides with the direction of currents and of the highest waves;

b) a wide harbour entrance;

c) a large area within the harbour for turning and manoeuvring to jetties and quays.

Such an ideal layout can seldom be achieved, particularly for harbours on the open coast, because the dominant currents rarely coincide with the direction of the highest seas. Ports located in estuaries, where the hydraulic conditions are determined mainly by the tides, generally offer better protection for seagoing navigation and many of the larger sea ports are situated in such locations. Access problems may still arise, however, since vessels may be required to follow the sinuous course of a natural channel and finally cross the tidal currents to the harbour entrance or riverside quays. Often extensive dredging works have to be carried out to meet the increasing navigational demands of larger vessels and considerable maintenance dredging costs may be incurred to remove siltation both in the artifically deepened access channel and in the harbour itself. The demand for deeper channels and reduction in dredging costs is tending to bring about the construction of new port facilities for large vessels nearer to deep water at sea.

18.2 Studies. Where access channels are to be aligned or alternative routes found, the opinion of experienced masters and pilots is essential. Such advice can be supplemented by model and/or simulation studies.

In one method, model vessels having similar hydrodynamic behaviour to the full scale vessels are studied in small scale harbour models in which different designs of harbour entrance, breakwaters, quays, jetties and other structures, can be tested. The speed of the model vessel, its rudder settings and any bow and stern thrusters provided to simulate the action of tugs are usually radio controlled and the manoeuvres recorded by overhead photography. This type of experiment is of limited value, partly because the operator has a bird's eye view of the situation and his response is almost immediate and partly because a skilled operator rapidly learns to manoeuvre the vessel safely in a given situation. The experiments can, nevertheless, be useful in developing a strategy for coping with difficult navigational conditions of current, wave and wind.

Another method uses larger manned ship models which may be controlled by one or two operators, one of whom may be an experienced mariner. This method overcomes the problem of the operator's bird's eye view by presenting him with a more realistic view of the situation. The problems of a contracted time-scale are not entirely overcome, however, although the bigger the model, the smaller the problem.

The final and perhaps most comprehensive method of evaluating ship handling in or near a proposed harbour or waterway is by means of a ship simulator in which vessel behaviour, bridge layout and view from the bridge are all realistically reproduced. The central feature of these simulators is a computer which is programmed for the seabed profile and different ship types, with the aid of information obtained from full-scale observations and from model tests carried out in basins in which waves and currents can be reproduced. The simulator enables typical ships to be manoeuvred along proposed approach channels in real time and therefore overcomes the time-scale problems mentioned above. In addition such a facility allows a realistic assessment to be made of the safety of the waterway from the point of view of navigation.

18.3 Depth of channels. The required depth of channel is largely governed by the underkeel clearance to be allowed for the largest vessels using the port which, for economy of dredging, may be limited to passage only near high water. This sets a minimum transit speed at ports with long approach channels and increases the risk of encounters between such vessels.

The underkeel clearance to be provided should take account of squat. Squat is normally divided into two components, mean sinkage and dynamic trim. As a vessel proceeds along the channel, the displacement of water causes an increase in return currents along the sides of the vessel and between the channel bed and the underside of the vessel. This is offset by a lowering of the adjacent water level, causing the vessel to experience sinkage and change of trim. In addition sufficient clearance should be allowed to take account of the vertical movement of the vessel in heave, pitch and roll produced by waves.

Squat increases with the length of the vessel, with the square of the forward speed and with reduction in underkeel clearance. Formulae have been developed on the basis of theory, model test data and full-scale measurements which allow the squat to be estimated for ships sailing in shallow open water. These formulae have been used to produce the squat estimation chart shown in Figure 2 for vessels with block coefficients between 0.8 and 0.9, i.e. large bulk carriers [18].

Additional squat is experienced by each of two ships when they pass, the effect being accentuated with reduction of underkeel clearance and vessel separation as well as with an increase of speed, and additional sinkage is also caused by sailing in the proximity of a channel bank [19].

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The vertical movements under waves and swell depend on the wave height and direction and the ratio of the wavelength to the relevant characteristic dimension of the ship, i.e. length for pitch and heave and beam for roll. These factors determine the forces exciting the motion. The response of the vessel to these forces is mainly governed by the ratios of its natural frequencies in heave, pitch and roll to the encountered wave frequency and by the damping of the motion in these modes.

Usually investigations of the vertical movements of ships in waves have been concerned with their sea-keeping qualities in deep water and estimates have been based on model tests and theoretical predictions. A graph illustrating the results of such investigations is shown in Figure 3 [18]. Unfortunately there are practically no full-scale measurements of vertical ship movements in channels where the underkeel clearance is small and few experimental studies have been carried out for such cases. It is known that with small underkeel clearance both the natural period, due to increase in added mass, and the hydraulic damping in each mode of movement, are increased (see **31.2**). In view of the lack of information on this subject, it is recommended that model studies should be carried out in specific cases to determine the limiting wave conditions and dredged depths of the access channel, having regard to the largest size and type of vessel to be accepted (see **18.2**).

Where the bottom of the channel consists of mud or silt it is usual to define a nautical depth from the water surface to the level at which the density of the bottom sediment is equal to or greater than 1 200 kg/m^3 since research has shown that mud layers of lower density do not significantly impede the passage of a ship. It should be noted that, depending on its operating frequency, an echo sounder will identify as the seabed muds of significantly lower density whereas a sounding lead will sink until supported by muds of greater density than 1 200 kg/m^3.

18.4 Width of channels. The width of access channels is governed mainly by the steering characteristics of the vessel in response to the pilot and helmsman, considered as a closed control loop, when subject to external disturbances such as the hydrodynamic effects of bank suction, cross-currents, wind, waves and other traffic. Large vessels generally have relatively long time constants and may be rendered even more sluggish in their response to a given force applied by the rudder, due to the increase in the hydrodynamic forces and added mass of the ship when the underkeel clearance is small. Thus in negotiating channels with bends, large changes of helm and engine speed are common and even in straight restricted channels pilots tend to take a sinuous course. The efficacy of new channel alignments and dimensions, navigational aids and training methods can be studied in ship handling simulators (see **18.2**) in which the response of the vessel and pilot to changing conditions can be reproduced in real time and the burden on the skill of the navigator can be assessed.

In a survey of one-way ship traffic in 10 major UK and overseas ports with channel widths of 300 m or less at their narrowest sections, the ratio of the channel width to the beam of the largest vessel accepted was found to lie between 4 and 6 depending on local conditions. The ratio found in this survey is not regarded as an operational criterion in these ports, however, since no port, as far as is known, limits the size of vessel by any formal width standard [18]. For somewhat smaller vessels passing one another, the minimum width of channel appears to be about six to eight times the average beam of vessels depending on the exposure and straightness of the channels. For large tankers, of up to 300 000 dwt, with either straight or curved channels subject to cross-tidal currents, recommended ratios of channel width to beam lie between 5 and 7 [20]. It is strongly recommended that two-way traffic of large tankers should be avoided whenever possible.

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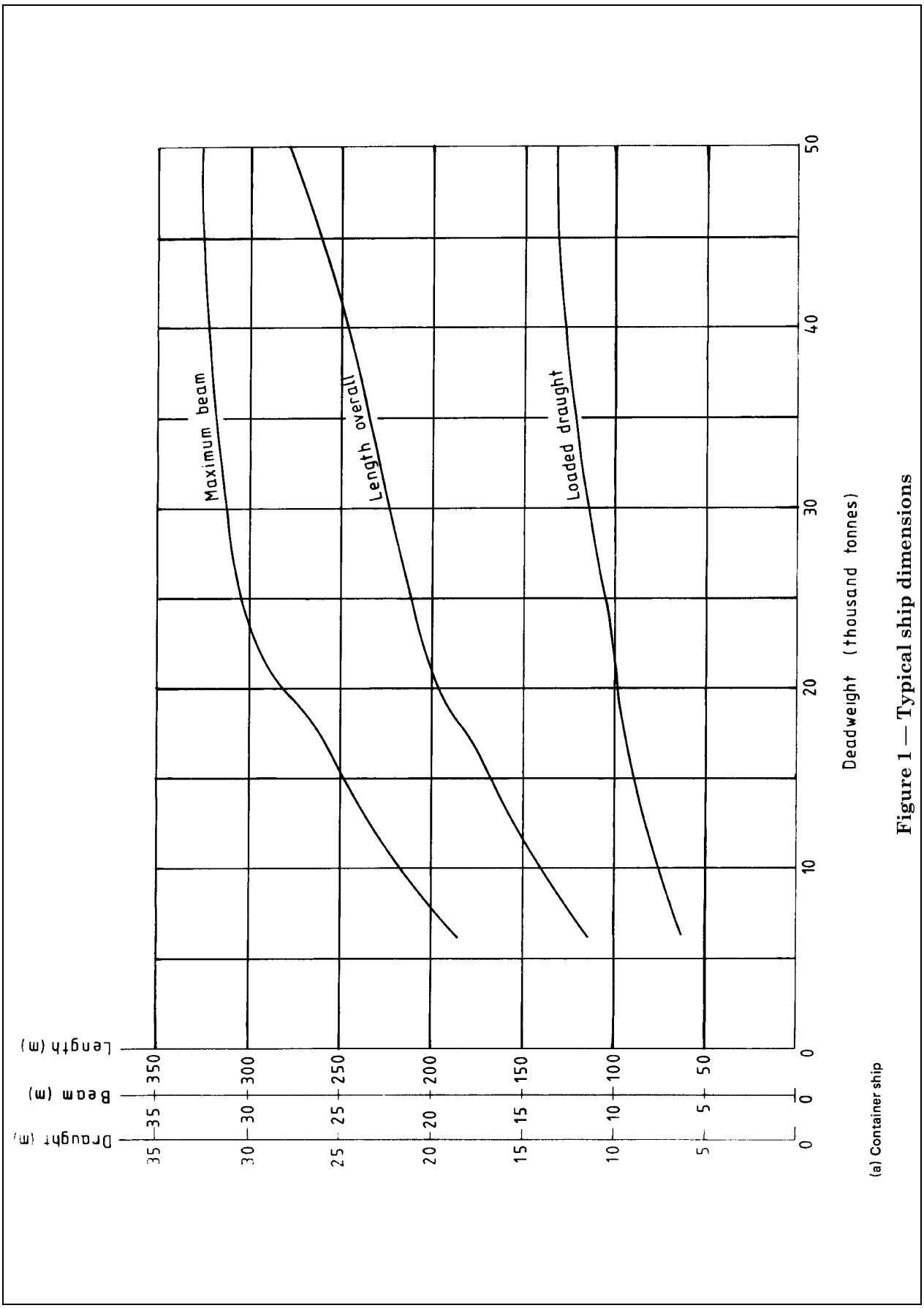

Figure 1 — Typical ship dimensions

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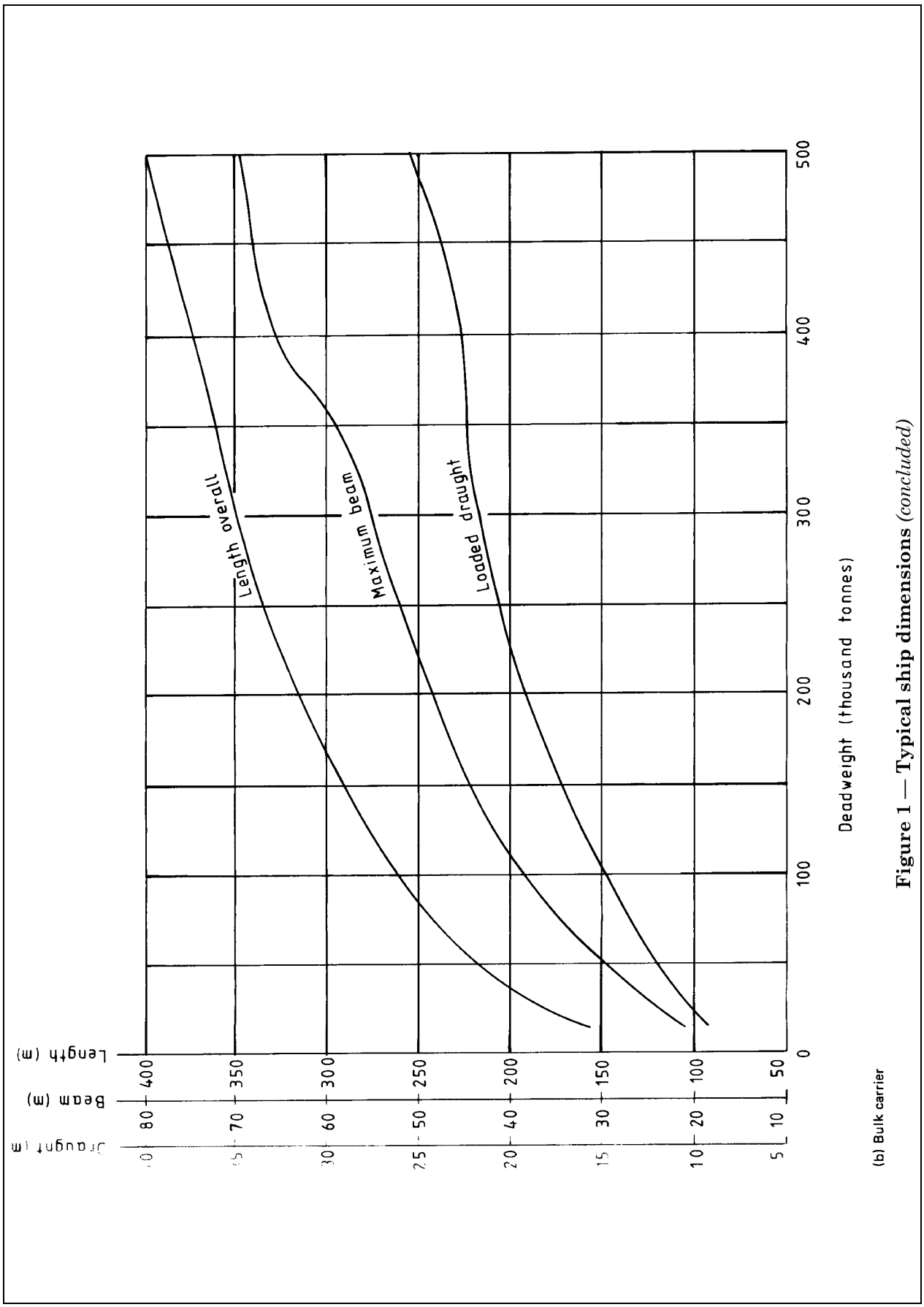

(b) Bulk carrier

Figure 1 — Typical ship dimensions *(concluded)*

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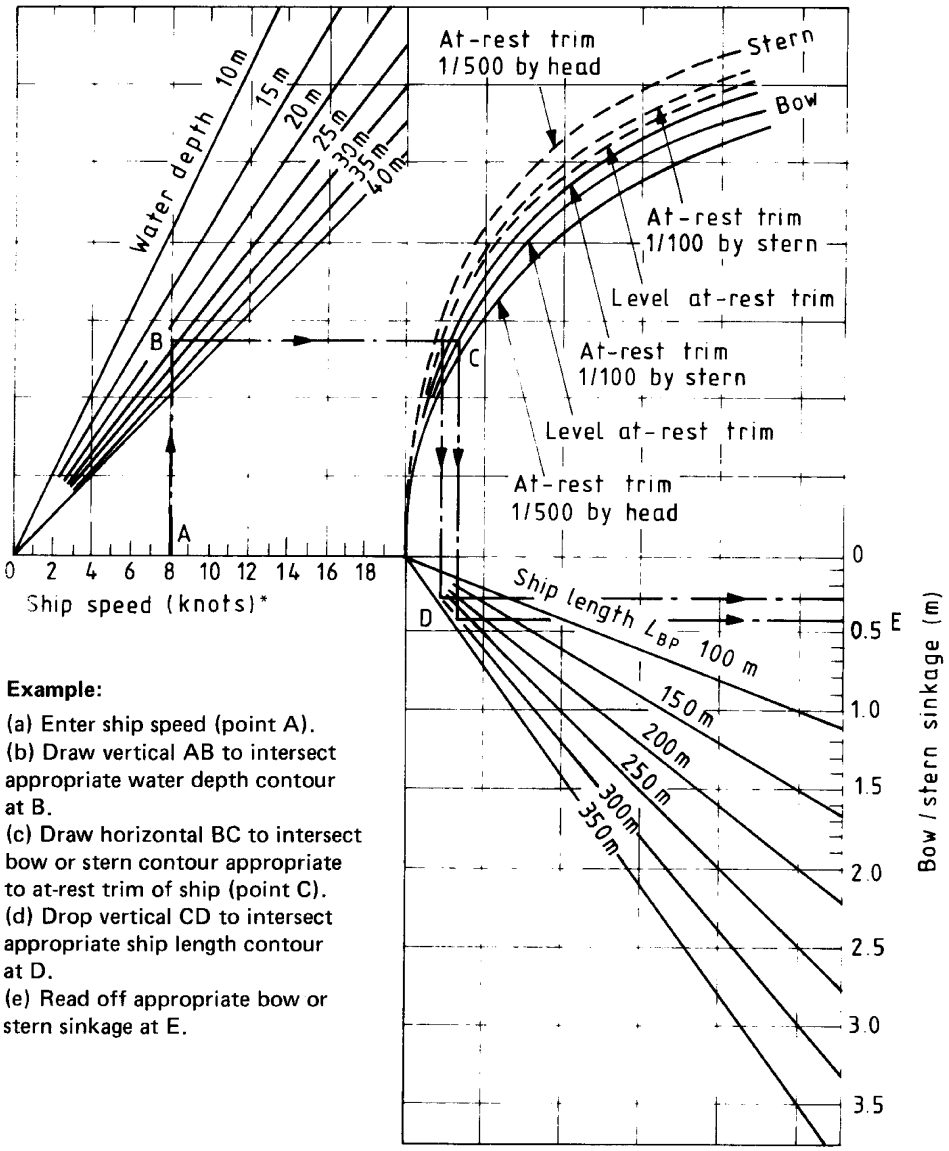

Example:

(a) Enter ship speed (point A).
(b) Draw vertical AB to intersect appropriate water depth contour at B.
(c) Draw horizontal BC to intersect bow or stern contour appropriate to at-rest trim of ship (point C).
(d) Drop vertical CD to intersect appropriate ship length contour at D.
(e) Read off appropriate bow or stern sinkage at E.

Applicable to ships with block coefficient C_B from 0.8 to 0.9, length to beam ratio $L/B_v \approx 6.0$, beam to draught ratio $B_v/d_m \approx 2.6$, and water depth to draught ratio d/d_m from 1.1 to 1.5.

*1 knot ≈ 0.515 m/s.

Figure 2 — Estimates of squat

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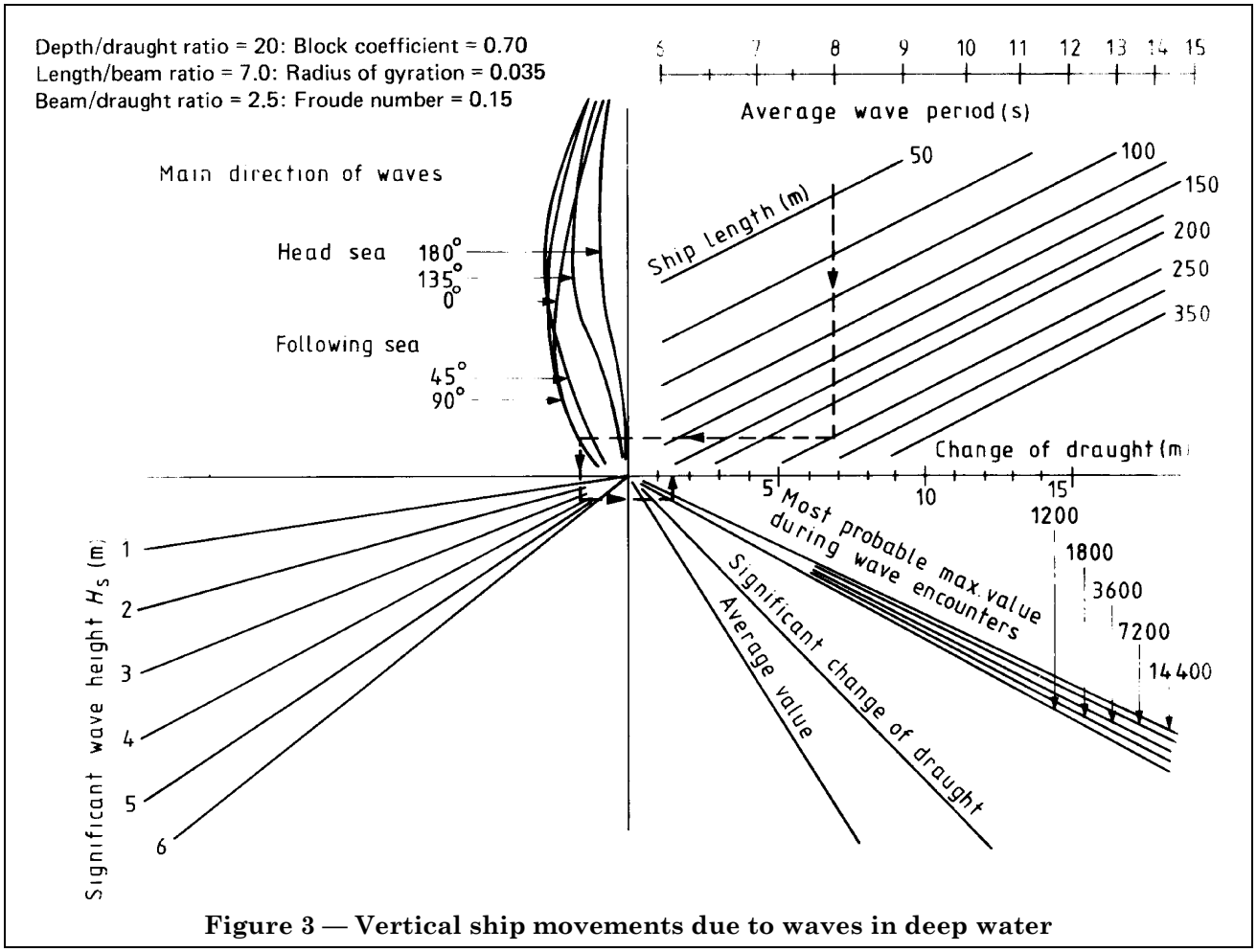

Figure 3 — Vertical ship movements due to waves in deep water

It can be argued that it is more appropriate to use vessel length to specify the minimum channel width since it is the changes in yaw angle, governed by the rudder and subject to changes in tidal cross-currents, which mainly cause deviations of course from the channel centre line. Since the length to beam ratio of large vessels lies generally between 5.5 and 6.5, the channel width to beam ratio given above can be interpreted in terms of length.

18.5 Other operational aspects. In addition to the hydro-dynamic effects discussed in **18.1** to **18.4**, other related operational aspects should be considered at an early stage.

Winds can impose operating limits on a port and its attendant waterways due to the increased difficulty of handling certain types of vessel in strong winds and to the impossibility of tugs and mooring boats working satisfactorily when the significant wave height exceeds a certain value.

The geometry of the dredged channel can have an effect on the speed at which a ship may be able to pass along it. This transit speed clearly is of importance to port operations as it affects the capacity of the waterways of the port as well as the day to day programming of ship movements.

Of similar significance is the decision to be taken regarding the advisability of two-way traffic and whether one-way traffic is appropriate for ships above a certain limiting size. When two ships pass they interact with each other to a greater or lesser degree, the severity of the effect being increased with reduced underkeel clearance, increased speed and reduced separation between the vessels. In general the overtaking encounter is best avoided, unless the vessels are extremely manoeuvrable, because the vessels remain in close proximity for an appreciable length of time which allows the large interaction forces to become well established. This restriction clearly has an effect on operations within the waterway, traffic in a given direction proceeding, of necessity, in a convoy system at the speed of the slowest vessel. In such a case the longitudinal separation between ships is of importance, affecting both safety of operation and capacity of the waterway. For one-way operations the longitudinal separation is largely determined by the distance required to stop a vessel in a controlled emergency stop. In two-way operation this separation should be further increased in the interests of safety to allow an oncoming vessel sufficient time after passing one convoy vessel to regain course before encountering the next.

19 Ship handling

19.1 Manoeuvring inside harbours. Ship handling should be fully considered in relation to the layout and dimensions of channels, turning circles and positioning of structures, taking account of the range of tides, currents and waves and the assistance available in the form of pilotage, tugs and navigational aids.

At the immediate approach to the harbour entrance, sea conditions may sometimes be rougher than elsewhere, due to the steepening of waves on an ebbing tide, offshore wind or reflection of waves from the harbour walls or breakwaters when these are steep sided. Accordingly, there will be a minimum velocity required to maintain safe steerage when entering the harbour but thereafter the vessel has to be brought to a standstill and the stopping length is frequently limited. It is customary for large vessels to carry out crash-stop trials at sea. However, such tests are not directly applicable inside harbours where the conditions are characterized by confined water, for both depth and width, low speeds, tug assistance and manoeuvres with rudder and propellor to hold course. While stopping lengths can be estimated they are probably best determined by model experiments (see **18.2**) if the area available is critical.

Inside the harbour, vessels frequently have to be turned and generous water areas should be provided for this and other manoeuvres. As a general guide, a turning circle with a diameter of four times the ship's length should be allowed where no tug assistance is available. The diameter may be halved where tugs are used regularly and may be further reduced if the critical vessels are equipped with bow thrusters. These average figures should be adjusted, if necessary, for local wind, wave and current conditions and for particular vessel or operator requirements, especially where the underkeel clearance is small.

19.2 Berthing. Full discussion of operational practices at a planned berth should be held with pilots, ships' masters and port operators whenever possible, with a view to determining the optimum layout for safe usage and the design parameters necessary for assessing berthing impact loads (see clause **41**).

Large vessels, such as bulk carriers and the larger container ships, are usually brought to rest a short distance off the jetty or quay and then manoeuvred on to the quay with the aid of tugs. Jetty-based electronic instrument systems can be provided to indicate speed and distance from the jetty during this manoeuvre, thus assisting the pilot to keep within the maximum allowable speed and the angle of approach for which the fendering system and jetty layout has been designed.

The employment of tugs may be compulsory in a port although the choice of how many will be the decision of the ship's master who will be guided by the pilot regarding local conditions and requirements. The number of tugs required for berthing depends upon the size of the ship, its power to weight ratio and the speed and direction of the current and winds relevant to the berth. Where no tugs, or insufficient tugs in terms of power or number, are used, the likelihood of excessive impacts is much greater. Where a ship is equipped with a bow thruster, this may reduce the number of tugs required to handle the vessel but can cause erosion problems at some berths.

For large bulk carriers, adequate pilotage and tug power should be provided as a matter of course. Berths for such vessels over 200 000 dwt should have velocity of approach monitoring equipment and should have a display to inform those on board the ship of the transverse approach velocity.

Ships which make a bow or stern approach to a berth under their own power, such as ferries and ro-ro vessels need special measures to ensure quick and safe ship operations, and full discussions with the relevant operators and authorities are essential.

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Predictions of berthing behaviour may be derived from measurements recorded at other berths in comparable situations or from experience of other structures. Records of such berthing impacts should be correlated, if possible, with ambient conditions of wave height and period, wind speed and direction, current velocity and tug assistance or technical aids.

Model studies may be made to determine the magnitude of the lateral force due to current and wind and the design of fendering and structures to resist impact loads which occur during the last stage of berthing. Berthing simulation techniques also allow various berthing strategies to be investigated before the ship arrives, or even before the berth is constructed.

Guidance on the assessment of berthing loads is given in clause **41**.

19.3 Mooring. When the ship has arrived at the berth and has been secured it is still subject to wind forces, current forces and movements caused by waves in the harbour and seiches (see **29.4**), as well as other long period effects caused by the suction of passing ships (see **18.5**). Therefore full discussion of mooring practices and mooring patterns should be held with pilots, ships' masters and port operators in order to assist the assessment of mooring loads and to ensure that the provision and layout of bollards, mooring hooks and other shore-based mooring equipment is appropriate to the requirements of the ships.

In some cases mooring loads may be limited by the practice of vessels either leaving the berth or taking on ballast during storm conditions. The reliance to be placed on such practices should, however, be examined critically before allowance is made in the design for their effects.

Guidance on acceptable wave conditions for moored boats is given in clause **30**, and guidance on methods of evaluating wave response for moored ships is given in clause **31**. Assessment of mooring loads is discussed in clause **42**.

Section 4. Sea state

20 General

When designing a maritime structure it will be necessary to obtain estimates of the expected extreme sea state for the site of interest. One method of obtaining such estimates is to use observations and calculations of high wind speeds and to apply them to a wave forecasting technique. Extrapolations of the wave height forecasts may then be carried out to obtain estimates of extreme wave conditions. Where possible these forecasts should always be checked against existing data sources, such as wave observations made from ships. The wave forecast will give a prediction of the offshore wave characteristics and it is then usually necessary to carry out a further study, incorporating wave refraction and attenuation due to the inshore seabed topography, to enable the inshore wave characteristics to be determined.

Estimates of extreme wave conditions may also be obtained by extrapolating a series of wave measurements made at the site. In general this requires a maximum of 1 year's wave records. Where possible the estimates based on wave forecasts from wind combined with wave refraction and attenuation calculations should be supplemented by site observations.

Inshore site observations are essential if reasonably accurate predictions are to be made in situations where the bottom topography is complex. For example, with offshore sand banks and bars the behaviour of waves is likely to be highly non-linear due to effects such as wave breaking, making the predictions of a wave refraction and attentuation study unreliable. If inshore wave data are not available for these sites then a physical model of the area incorporating a random wave generator may be used to estimate the inshore wave climate. However, care has to be taken with such models since wave height attenuation due to wave breaking and bottom friction is unlikely to be well represented at small model scales. Also, in situations where large vessels or structures are to be moored inshore or inside harbours which are in a relatively exposed location, it is necessary to measure long waves, with periods of the order of minutes, at the site since it is known that long period wave motions associated with wave grouping are frequently responsible for the large mooring loads which sometimes lead to large vessels and structures breaking away from their moorings. As these long-period wave motions are also non-linear, their calculation for inshore locations is unreliable at present, making site observation the only reliable method of prediction.

This section gives guidance on the above processes of investigation, prediction and extrapolation for both the offshore and inshore wave climates. Guidance is also given on the effect on sea state of maritime structures such as seawalls, breakwaters and harbours, together with methods of determining acceptable sea states for moored vessels. General information on wave characteristics is given in clause **21** in which definitions and derivations of terms commonly used to describe wave properties may also be found.

21 Wave characteristics

21.1 Wave forms. Waves have the ability to propagate energy to distant points but, as can be seen by the motion of a small floating body which is carried forward in the direction of wave advance on a wave crest and moves back in a wave trough, the water itself does not translate with the wave to any significant extent. In deep water the water particles on the surface move in almost circular orbits with a diameter approximately equal to the height of the wave. This orbital motion decreases rapidly with depth.

In shallower water, where the wave motion is attenuated by the restricted depth, the water particles move in orbits which approximate to an ellipse at the surface and to a horizontal straight line at the seabed.

Waves, especially those in deep water, are commonly idealized as sinusoidal waves in which increasing orders of complexity may be used to account for observed departures from the linear, or first order, theory which predicts a symmetrical profile about the mean water level and closed particle orbits. In increasingly shallow water the wave crests tend to become steeper and the troughs flatter and for certain applications in these situations cnoidal wave theory or solitary wave theory may give a better idealization of the wave behaviour than sinusoidal theory.

Unless noted otherwise, the expressions relating to wave characteristics used in this code are derived from first order sinusoidal theory. Where necessary, guidance on other wave theories may be obtained from modern references [16, 21].

21.2 Basic wave properties

21.2.1 *General.* For a given water depth, monochromatic waves may be described by the properties of height and period. Details of these and derivations of other related properties are given in **21.2.2** to **21.2.9**.

21.2.2 *Wave height.* The wave height is the height of a wave crest above the preceding wave trough and is denoted generally by H and, in deep water, by H_0.

21.2.3 *Wave period.* The wave period is the time for two successive wave crests to pass a fixed point and is denoted generally by T. In first order theory the period is assumed to be independent of water depth.

21.2.4 *Wave frequency.* The wave frequency is the inverse of wave period and is denoted by f.

21.2.5 *Wavelength.* The wavelength is the distance between consecutive wave crests and is denoted generally by L and, in deep water, by L_0. The variation of wavelength in shallow water is shown in terms of the deep water wavelength in Figure 11 (see **23.2**). Wavelength is related to the wave period and phase velocity (see **21.2.6**).

21.2.6 *Phase velocity.* Phase velocity, sometimes referred to as wave celerity or velocity of wave propagation, is the speed at which a wave propagates. This quantity is denoted generally by v_c and, in deep water, by v_{c0}. Phase velocity is related to the wave period and wavelength by the expression $v_c = L/T$.

First order theory gives the phase velocity as:

$$v_c = \frac{gT}{2\pi} \tanh\left(\frac{2\pi d}{L}\right)$$

where

d is the still water depth;

g is the acceleration due to gravity (9.81 m/s^2).

This gives the wavelength as:

$$L = \frac{gT^2}{2\pi} \tanh\left(\frac{2\pi d}{L}\right)$$

For values of $d/L > 0.5$ these expressions closely approximate to the following deep water relationships:

$$v_{c0} = \frac{gT}{2\pi}$$

$$L_0 = \frac{gT^2}{2\pi}$$

21.2.7 *Orbital velocity.* Since one orbit of circumference πH_0 is completed in one wave period T, the orbital velocity at the surface in deep water is $\pi H_0/T$. Values of the orbital velocities and particle accelerations, which decrease with depth below the surface, are required for assessing wave forces on submerged structural members and further guidance on this aspect may be found in **39.4.4**. Generally the orbital velocity at the surface is much smaller than the phase velocity, v_c, but when it just exceeds v_c,the water particles at the wave crest will catch up the preceding wave trough and lead to wave breaking.

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21.2.8 *Wave steepness.* The wave steepness is the wave height divided by the wavelength. Comparison of the expressions for orbital velocity and phase velocity indicate the existence of a limiting wave height in deep water which is less for short waves than for long waves. By solving the basic wave equations with their full non-linear surface conditions the extreme progressive wave of single period is found to have a steepness of about 1 in 7 in deep water.

21.2.9 *Group velocity.* A train of waves of single period travelling in still water will be seen to propagate at a velocity less than the phase velocity of the individual waves, therefore, in a limited train, waves will appear to be created at the rear and to move through the train to die out at the wave front. The velocity of propagation of the train is known as the group velocity and is the velocity at which the energy of the wave train travels.

The group velocity is denoted generally by v_{cg} and in deep water by v_{cg0}. First order theory relates the group velocity to the phase velocity by the following expression:

$$v_{cg} = \frac{1}{2}\left(1 + \frac{4\pi d/L}{\sinh(4\pi d/L)}\right) v_c$$

In deep water, this approximates to:

$$v_{cg0} = \tfrac{1}{2} v_{c0} = \frac{gT}{4\pi}$$

The variation of group velocity in shallow water is shown in terms of the deep water wavelength and phase velocity in Figure 11 (see **23.2**).

21.3 Sea state properties

21.3.1 *General.* Real waves may be viewed as being formed from a number of single wave components each with a well defined period of oscillation. As the components come into phase with one another they will give rise to a group of larger waves which, as described in **21.2.8**, travel at the group velocity. Repeated interactions such as these cause the real sea surface to have a very irregular appearance and attempts to describe this situation quantitatively lead to the introduction of the parameters detailed in **21.3.2** to **21.3.7**.

21.3.2 *Significant wave height.* The significant wave height is the average height of the highest one third of the waves, is denoted by H_s and has been found to approximate to the visual estimate of wave height that would be obtained from an experienced observer.

21.3.3 *Significant wave period.* The significant wave period is the average of the periods of the highest one third of the waves and is denoted by T_s.

21.3.4 *Zero-crossing wave period.* The zero-crossing wave period is the average period of all the waves with troughs below and crests above the mean water level and is denoted by T_z.

21.3.5 *Spectral density.* Energy in the sea is carried by a large number of individual waves which have different frequencies and which propagate in a range of directions. The spectral density is obtained by assigning an appropriate value of the square of the wave height to each wave component with a given frequency and direction. It is therefore a measure of the energy of the sea state expressed as a function of wave frequency and direction and is denoted by $l(f,\phi)$.

21.3.6 *One-dimensional spectral density.* As yet, limited data frequently prevent a reliable estimate to be given for the directional distribution of energy as required for the calculation of the full directional spectral density $l(f,\phi)$. When the energy at any one frequency is added together for all directions the one-dimensional spectral density, denoted by $S(f)$, is obtained.

21.3.7 *One-dimensional wave spectrum.* A graphical plot of one-dimensional spectral density against frequency. Reliable estimates of such spectra are now available and in common use (see **22.2.5**). Greater accuracy in the prediction of the response of structures to wave action has been obtained in both physical and mathematical models by the use of these representations of the sea surface instead of a uniform wave representation consisting of only one wave period.

21.4 Return period and design wave condition. The frequency of recurrence of a meteorological event is often specified by its return period, T_R, defined as the period which, on average, will separate two occurrences. It should be noted that this does not mean that exactly T_R years will separate two such occurrences. In fact, if the events can be taken to be statistically independent, then the probability, p, that a condition with a return period of T_R years will occur within a period of n years is given by:

$$p = 1 - (1 - 1/T_R)^n$$

Thus, for an event with a return period of 100 years, there is a 1 % chance of occurrence in any one year, even the one following a previous occurrence, and approximately an 18 % chance of occurrence in a 20 year period. For a time interval equal to the return period p is given by:

$$p = 1 - (1 - 1/T_R)^{TR}$$

$$\approx 1 - 1/e = 0.63$$

Therefore there is a 63 % chance of occurrence within the return period.

When carrying out extrapolations of the design parameters to obtain some estimate of their extreme values (see clause **27**) the question arises as to what return period to use for the design condition. Structures designed to withstand almost anything will in certain situations be necessary but in others they will work out to be more expensive than weaker structures for which the cost of periodical repair has been included. By taking costs into account in this way it may be possible to establish an acceptable level of risk of the design condition occurring within a given number of years. For example, if it has been established that over a 20 year period it is most economical to design a structure against a condition with a 20 % probability of occurrence, then the necessary return period for this condition can be estimated from the following:

$$0.2 = 1 - (1 - 1/T_R)^{20}$$

i.e. $T_R \approx 90$ years

In general it can be seen that the return period of the design condition will exceed the given period over which costs are to be optimized.

This type of cost optimization may only be used with confidence if the degree of damage that results when design conditions are exceeded is known. It is therefore most applicable to constructions such as rubble-mound breakwaters in which the damage can be expected to be gradual and in which model tests can help in establishing the rate of damage. In contrast some structures like vertical-faced seawalls may undergo almost complete destruction when the design condition is exceeded.

Where the consequences of failure are so grave as to be unacceptable at other than very low probabilities then such structures should be able to withstand design conditions with return periods of the order of 1 000 years. However, it should be realised that the accuracy of extrapolation of wave or meteorological data to such extremes may be affected by long term changes in the environment (see **27.2.1**).

22 Offshore wave climate

22.1 Wave generation. Waves commonly observed in the oceans are generated by the action of the wind. Although the primary physical mechanism by which the wind generates waves has not yet been identified it may be expected that the wind will transfer energy to those waves with a velocity of propagation slightly less than the wind speed.

In one proposed mechanism of wave generation, turbulent atmospheric pressure fluctuations with a wavelength and speed of propagation that match those of a water wave are shown to lead to wave growth. The initial wave pattern consists of two wave trains forming a rhombic pattern with a diagonal along the direction of mean wind. On this basis the shorter waves which travel obliquely to the wind appear first and grow to their limiting value and break while longer-period waves, propagating at less of an angle to the mean wind, grow more slowly but attain greater heights thereby tending to make the two wave trains coalesce with increasing distance. In this mechanism of wave growth the waves, once formed, are not allowed to react back on the atmospheric pressure fluctuations and so the mechanism is probably only applicable to the early stages of wave growth.

In another proposed mechanism the waves on the sea surface are assumed to be matched by waves on the bottom surface of the atmosphere and the transfer of energy is assumed to take place in layers of air in the shear zone at the particular height for which the wind speed equals the velocity of wave propagation. In this mechanism any interaction between the waves and atmospheric turbulence is ignored.

Although both the above mechanisms give growth rates which are lower than the observed rates of wave growth, taken together, they are consistent with observation to the extent that wave energy in the oceans is distributed over a range of wave periods and wave directions relative to the mean wind direction. They also serve to show why the sea surface takes on a very irregular appearance during a storm.

Recent observations of waves in the sea have shown that interactions between the waves are important in redistributing energy amongst the various wave components. It has been found that wave energy is transferred to periods both longer and shorter than the periods generated by the wind. Although such interactions amongst the waves only redistribte wave energy and so do not explain the discrepancy between the observed rates of wave growth and those predicted by the mechanisms referred to above, they are important in determining the shape of the one-dimensional wave spectrum (see **22.2.5**).

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22.2 Wave prediction

22.2.1 *General.* The methods of wave prediction described in **22.2.2** to **22.2.5** require estimates to be made of the extent of the wave generating area, known as the fetch, and of the wind speed which acts within that area for a given duration. Two general methods are then available for estimating wave parameters. The first method relies upon the use of prediction charts which give estimates of the significant height and period. The second method relies upon knowledge recently gained of typical one-dimensional wave spectra.

It should be noted that predictions are likely to be inaccurate due to the difficulties of defining the wind field and wave generation mechanisms accurately. The actual site may also be affected by swell (see **22.3**). It is therefore good practice to supplement wave forecasts with wave observations made from ships [22] and, whenever possible, with direct wave recordings (see clause **26**).

22.2.2 *Wind speed and duration.* The wind speed to be used, unless otherwise stated, should be the speed at 10 m above sea-level, averaged over the duration.

When forecasting for large ocean areas, meteorological synoptic charts which show isobars may be used to obtain estimates of wind speed, duration and fetch. In these cases the definition of the wind field, based on calculations which are best carried out by specialists, are found to be more reliable than direct measurements of wind velocity made from a moving ship.

Over smaller well defined fetches, coastal measurements of wind speed can be used. In these situations it is usual to increase the mean coastal wind speed by 10 % to obtain the equivalent wind speed over the open sea. It should be realized that if coastal wind data are used in this way then an assumption is being made that the wind speed and direction over the entire fetch is compatible with that at the coastal station. The resulting forecast will be unreliable in those situations where the fetch length typically exceeds half the radius of the cyclonic wind pattern.

22.2.3 *Fetch length.* The fetch used in wave forecasting techniques should ideally be restricted to one within which the wind speed does not vary by more than 2.5 m/s from the mean speed and the wind direction does not vary by more than 30°. Wave generation within a fetch may be reduced where the width of the fetch is much less than its length but evidence exists which points to this effect being small. The fetch length should therefore be considered to be equal to the straight line distance from the point at which the wave height is required to the upwind boundary of wave generation. The boundary may be provided by land or by meteorological conditions.

22.2.4 *Prediction by significant wave charts.* The more reliable method of wave prediction uses basic hydrodynamic theory and empirical data to predict average wave quantities in terms of the wind speed, the fetch length and the wind duration.

The range of earlier prediction charts has now been extended and the reliability of this method has been improved as increased data bases have become available. Typical deep-water wave prediction curves, which correlate well with the results of spectral techniques over a wide range, are shown in Figure 6. These charts may be used by entering with the value of wind speed and following it across until it intersects with either the fetch length or the duration, whichever comes first. The significant height and period may then be obtained at the point of intersection.

In deep water the wave energy is proportional to the square of the product of the wave height and period so the dotted lines of constant $H^2\ T^2$ shown on Figure 6 represent lines of constant energy. These may be used to obtain wave parameters from the cumulative effects of varying wind speeds histories; however care has to be taken to check that the fetch limitations are not exceeded in such cases.

For example, for a fetch of 120 km over which the wind speed averages 20 m/s from 1 000 hours to 1 600 hours and 25 m/s from 1 600 hours to 1 800 hours the significant height and period at 1 600 hours are given, for 20 m/s and duration 6 h, as 3.7 m and 7.6 s respectively. By following the constant energy curve upwards until the 25 m/s wind speed line is reached and moving along this line to the right an amount equal to 2 h, the significant height and period for 1 800 hours are found to be 4.8 m and 8.6 s, respectively, at a fetch greater than or equal to 97 km. Had the fetch for the higher wind speed been only 80 km for instance then the significant height and period for 1 800 hours would have been fetch limited at 4.4 m and 8.3 s, respectively.

22.2.5 *Prediction by wave spectra.* Recent studies, particularly those associated with offshore work in the North Sea, have enabled reasonable estimates to be made of the typical one-dimensional wave spectra in the fetch-limited situation which complement information previously obtained in the North Atlantic for the fully developed spectrum.

Examples of the two types of one-dimensional wave spectra are shown in Figure 7 and Figure 8 in which the spectral density, $S(f)$, is plotted against wave frequency, f. The graphs show in general terms how wave energy is distributed over the various wave periods in the sea and the area under the curve, which has the dimensions of metres squared, may be used to obtain estimates of wave height parameters. Analysis of empirical data has shown that the significant wave height, H_s, is given by the relationship:

$H_s = 4 \times (\text{area under the spectrum})^{½}$

The analysis of empirical data for those situations where the waves are fetch limited has resulted in the JONSWAP (Joint North Sea Wave Project) spectrum in which the spectral density is given by:

$$S(f) = \frac{\kappa_J g^2}{(2\pi)^4 f^5} \exp\left[-\frac{5}{4}\left(\frac{f_m}{f}\right)^4\right] \gamma^a$$

where

$$a = \exp\left[-\frac{(f-f_m)^2}{2\omega^2 f_m^{\,2}}\right]$$

$$K_j = 0.0662/x^{0.2} = 0.033\left(\frac{f_m U_w}{g}\right)^{2/3};$$

where

$$x = \frac{gL_F}{U_w^{\,2}} = \left(\frac{2.84g}{U_w f_m}\right)^{10/3};$$

where

L_F is the fetch length;

U_w is the wind speed 10 m above the sea surface;

f is the wave frequency;

f_m is the frequency at which the peak occurs in the spectrum and equals $2.84g^{0.7} L_F^{-0.3} U_w^{-0.4}$;

γ = 3.3;

ω = 0.07 for $f \leqslant f_m$ or 0.09 for $f > f_m$.

Figure 7 shows the JONSWAP spectrum for the case where U_w = 20.6 m/s (40 knots) and L_F = 29.81 km (16 nautical miles).

In addition to the foregoing relationships a non-dimensional parameter describing the surface variance was also determined from the JONSWAP observations. This may be used to calculate the significant height directly but gives values approximately 10 % less than those shown in Figure 9, which have been calculated as described above.

Figure 4 — *Figure deleted*

Figure 5 — *Figure deleted*

Empirical data from the North Atlantic Ocean have been used to define a fully developed one-dimensional spectrum known as the Pierson–Moskowitz spectrum in which the spectral density is given by:

$$S(f) = \frac{\kappa_P g^2}{(2\pi)^4 f^5} \exp\left[-\frac{5}{4}\left(\frac{f_m}{f}\right)^4\right]$$

where

k_P = 0.0081;

$$f_m = \frac{0.8772g}{2\pi U_{19.5}};$$

$U_{19.5}$ is the wind speed at 19.5 m above the sea surface.

Figure 8 shows the Pierson–Moskowitz spectrum for the case where $U_{19.5}$ = 22.66 m/s.

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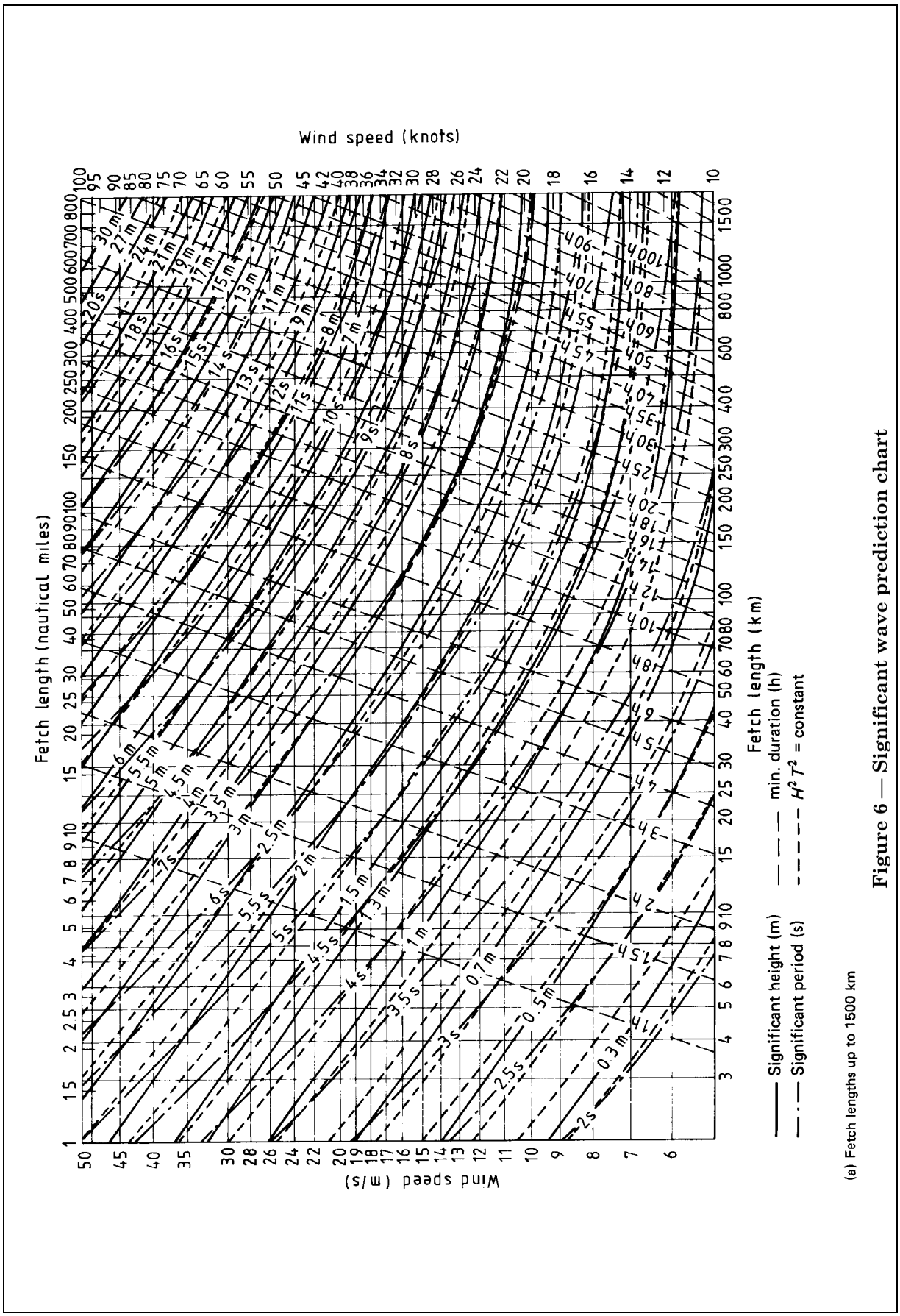

(a) Fetch lengths up to 1500 km

Figure 6 — Significant wave prediction chart

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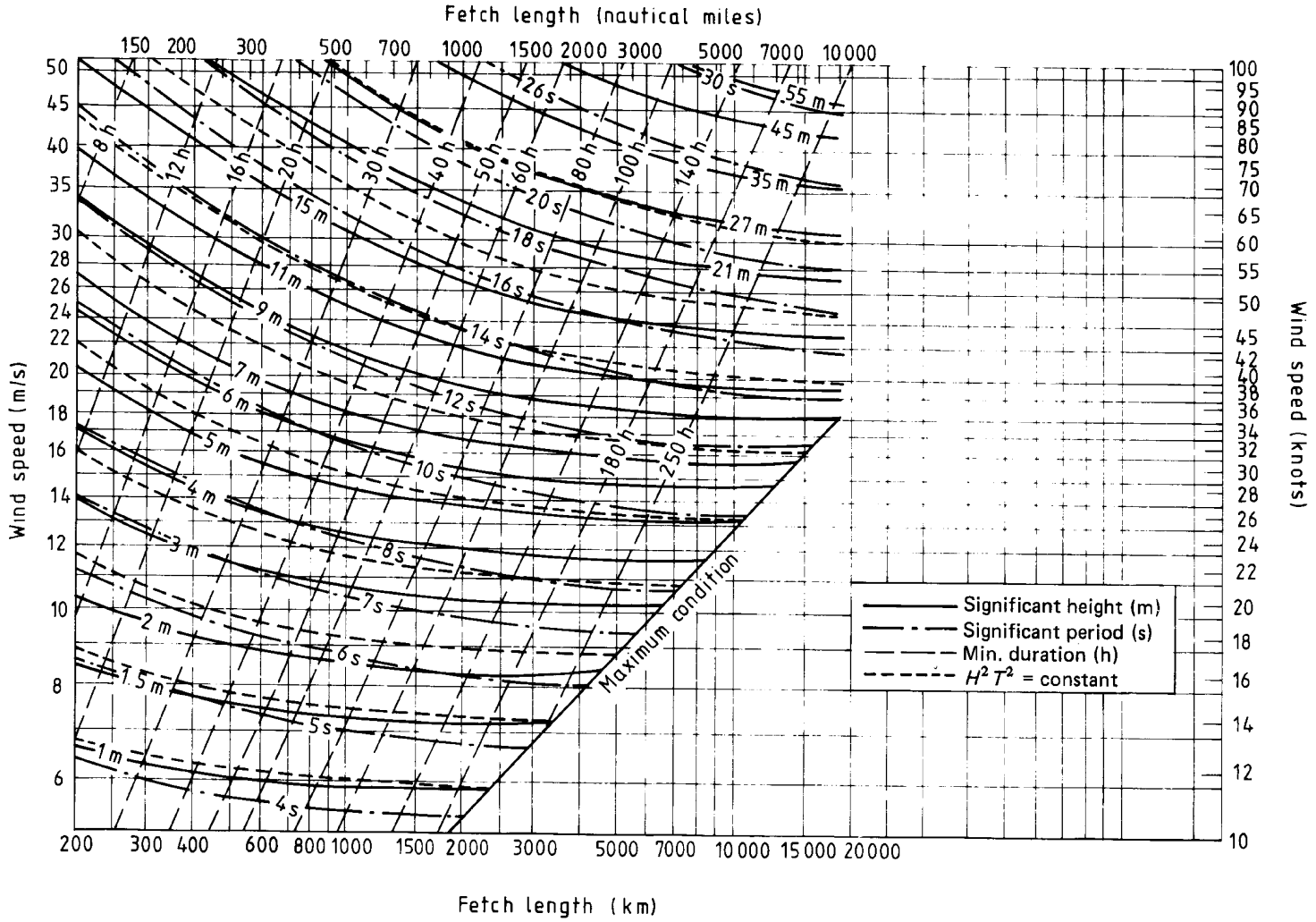

(b) Fetch lengths from 200 km to 20 000 km

Figure 6 — Significant wave prediction chart *(concluded)*

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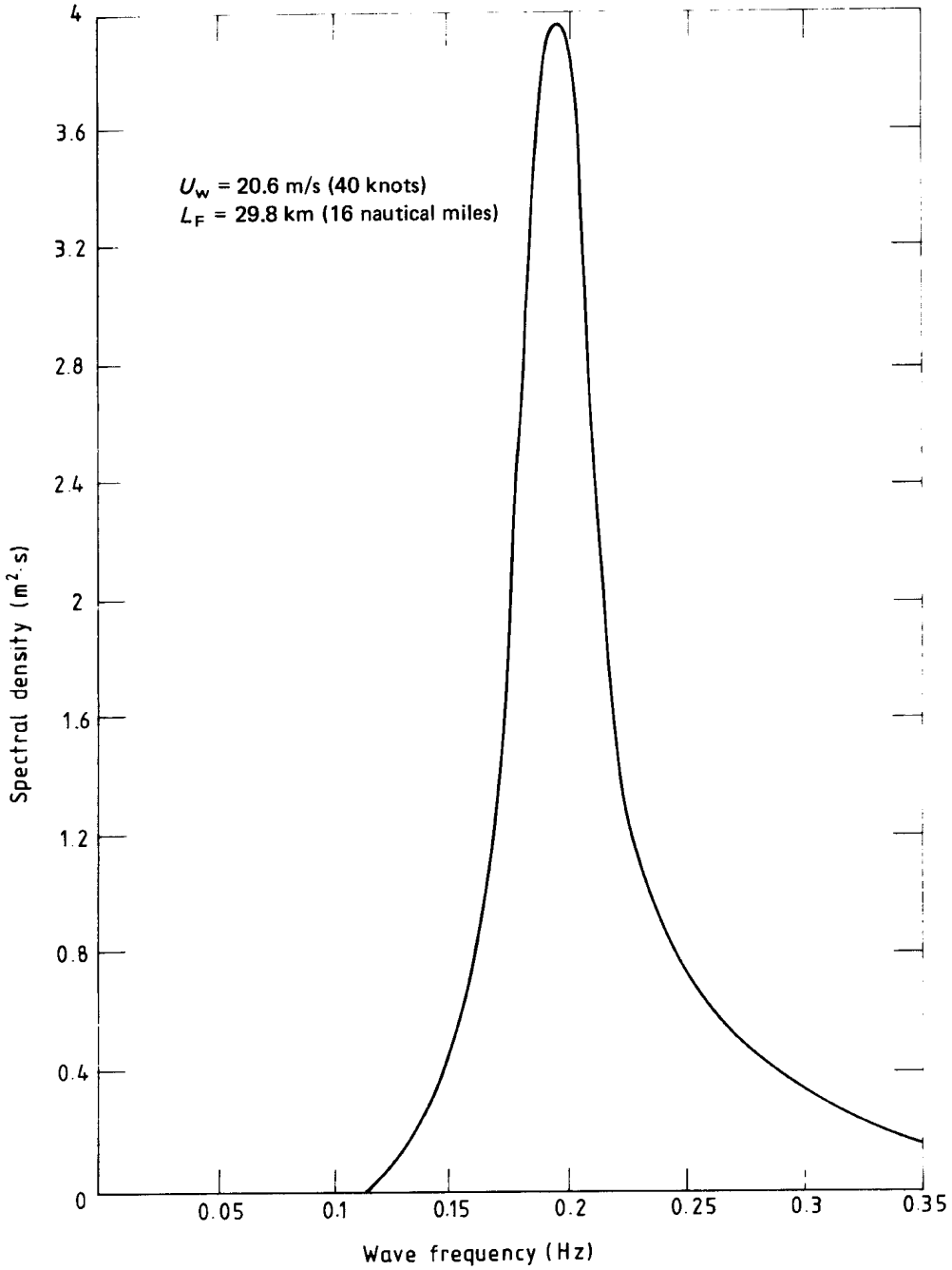

Figure 7 — JONSWAP wave spectrum

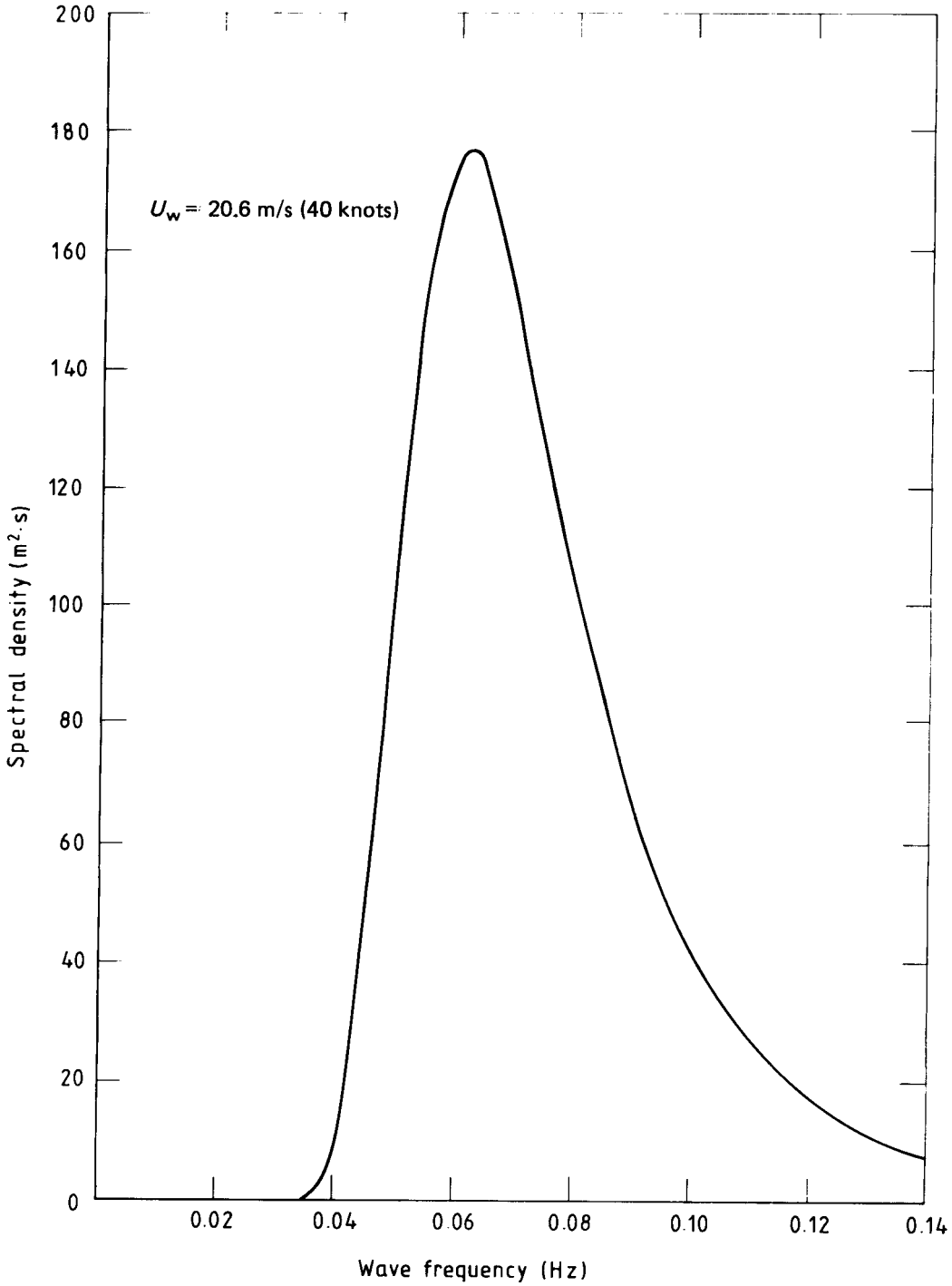

Figure 8 — Pierson–Moskowitz wave spectrum

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Since the wind speed U_z at height z m above the sea surface can be related to the wind speed U_w at 10 m above the sea surface by the expression $U_z = U_w (z/10)^{1/7}$ it can be seen that $U_{19.5} = 22.66$ is equivalent to $U_{10} = 20.6$ so the same effective wind speed has been used to obtain both spectra. Comparison of the two figures shows that the spectral peak frequency of 0.190 Hz in Figure 7 has moved down to 0.0604 Hz in Figure 8 and that, as expected in a growth situation, the amount of wave energy in Figure 8, i.e. the area under the spectrum, is considerably greater than the energy in Figure 7. It can also be shown that for $f = 0.190$ Hz the spectral density for the equilibrium, i.e. fully developed, spectrum is approximately half the value of the spectral density of the fetch-limited spectrum. These results are consistent with the fetch-limited data which indicated that interactions amongst the waves cause a migration of the spectral peak towards low frequencies with increasing fetch as well as giving a final equilibrium spectral density, for frequencies to the right of the spectral peak, of approximately half of the peak spectral density reached at shorter fetches. This latter effect, known as overshoot, has been observed in other empirical data at shorter fetches. Since the data used to obtain the fetch-limited spectrum was collected for fetches of up to 160 km the JONSWAP study was unable to verify whether the fully developed spectrum, where energy input from the wind is exactly balanced by energy losses, is achieved or whether the wave height can go on increasing with interactions amongst the waves causing a further migration of the spectral peak to even lower frequencies.

In the absence of any better information it is common practice, at present, to use the Pierson–Moskowitz spectrum for all those cases where the JONSWAP spectrum predicts a lower spectral peak frequency than the Pierson–Moskowitz spectrum, i.e. where $gL_F/U_w^2 > 2.92 \times 10^4$.

For shorter fetches than this, the JONSWAP spectrum is probably the most reliable prediction for the one-dimensional spectrum because it is based on the most comprehensive data obtained for fetch-limited situations. However, care is required in making predictions for fetches and wind speeds that fall just short of producing a fully developed sea state since the H_s value obtained from the JONSWAP spectrum will exceed that obtained from the fully developed spectrum. This difficulty may be overcome by the use of Figure 9 in which the contours of H_s have been adjusted to give a smooth transition between the two spectra.

22.3 Wave decay and swell. In the waters around the British Isles the most severe wave conditions are usually associated with storm waves and the wave forecasting techniques already described can be used to predict such conditions. In some situations, however, swell waves from distant storms are one of the more important features to be taken into account in the design of inshore structures. Because these waves have propagated out of their region of generation, the wave energy has subsequently spread over a large area making the waves lower in height and longer crested than storm waves. To be able to predict the heights and periods of these waves it is necessary to use published data on the decay of waves when they leave the generation region, in conjunction with the wave conditions at the end of the fetch [16]. However, knowledge of the wind conditions in the distant storm centre is rarely available so this method of prediction is not often used. In such situations it is preferable to install a wave recorder at the site of interest to obtain sufficient data so that an estimate of the extreme wave conditions can be made.

If time does not permit the installation of a wave recorder and the subsequent analysis of the data, then reference may be made to published statistical surveys of a number of years of visual wave observations made by shipping, in which wave heights, periods and directions are usually available for each month of the year and for the whole world [22]. Although individual observations taken by eye from a ship may be unreliable it is generally accepted that predictions based on a large number of observations made by different people does result in a useful estimate of wave conditions. These data will include storm waves as well as swell and should be used to check predictions of wave conditions based on wind data.

22.4 Extrapolation of offshore wave data. Having obtained offshore wave data from one or a number of the above methods it will be necessary to extrapolate these data to obtain extreme conditions appropriate to the design requirements. Extrapolation techniques are described in clause **27**.

23 Shallow water effects

23.1 General. When a wave group propagates into shallow water its characteristics are altered by the influence of the seabed, leading to changes in the velocity, length, height and direction of the waves although the period of the waves may be assumed to be constant.

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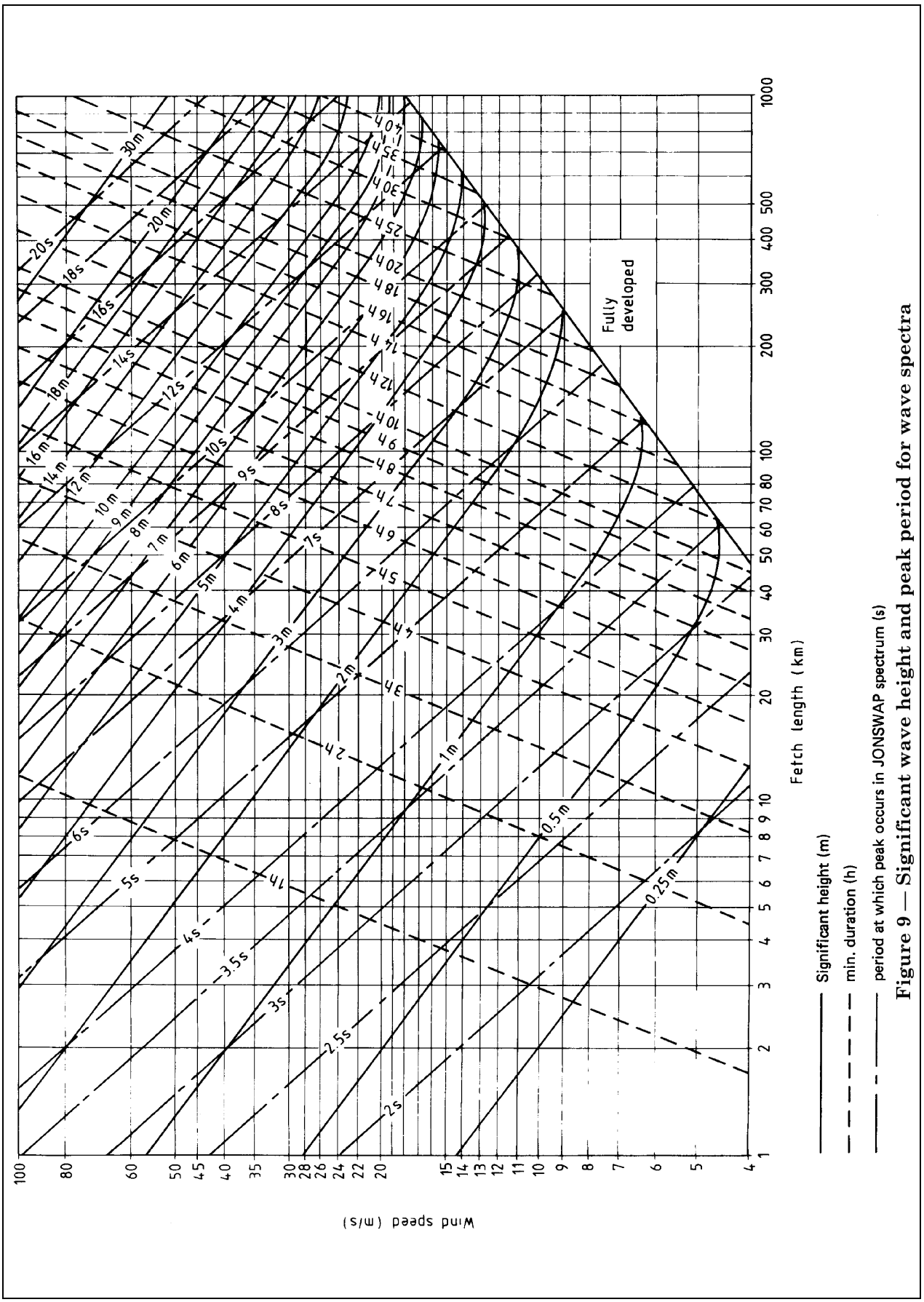

Figure 9 — Significant wave height and peak period for wave spectra

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Shallow water effects should be considered when the depth reduces to less than one half the deep-water wavelength.

Effects of refraction, shoaling, bottom friction and wave breaking are considered in this clause. Effects of diffraction and reflection are considered in clauses **28** and **29**.

23.2 Refraction and shoaling

23.2.1 *General.* Since the phase velocity of a wave decreases in shallower water, a wave front approaching the coast at an angle to the seabed contours will be refracted so that the wave crests will tend to align themselves parallel to the bed contours.

The effect is analagous to the refraction of light and the amount of deviation may be computed using Snell's law which effectively states that $(\sin \psi)/v_c$ is constant, where ψ is the angle between the wave front and the depth contour and v_c is the phase velocity of the wave. A schematic diagram of refraction is shown in Figure 10 which is representative of situations where the bed profile may be approximated to straight parallel contours.

For situations where the bed contours are irregular or curved in plan a series of wave orthogonals or "rays" may be constructed by progressive computation using Snell's law in order to provide a pictorial representation of the effects of refraction over the area considered.

The divergence or convergence of adjacent rays indicates a concentration or dispersion of the wave energy along the wave crest. When the angle ψ is zero, refraction will not take place; however the shoaling effect, in which wave energy is compressed or stretched in the direction of wave advance in response to variations in the group velocity, will always be present whenever waves move into shallow water.

Provided the changes in bathymetry are gradual and no energy is dissipated in wave breaking or bottom friction (see **23.3**) then the shallow water wave height will be governed by the conservation of energy flux such that:

$$H^2 v_{cg} b = \text{constant}$$

where

H is the wave height;

v_{cg} is the wave group velocity;

b is the length of wave crest between two rays, i.e. the wave ray separation.

The inshore wave height may then be computed from:

$$H = K_s K_r H_0$$

where

H_0 is the deep water wave height;

K_s is the shoaling coefficient given by $\sqrt{(v_{cg0}/v_{cg})}$;

where

v_{cg0} is the wave group velocity in deep water;

K_r is the refraction coefficient given by $\sqrt{(b_0/b)}$

where

b_0 is the wave separation in deep water.

Values of the shoaling coefficient, together with wavelength and group velocity, may be obtained from Figure 11(a).

The shoaling effect upon random waves, including the effects of wave breaking, may be estimated from Figure 11(b).

The refraction coefficient may be found by measurement of the relative divergence or convergence of wave rays obtained by refraction analyses.

It should be noted that abrupt changes in submarine contours can lead to wave diffraction and reflection which will cause energy transfer across rays and thereby invalidate the above expressions.

23.2.2 *Computational refraction methods and problems due to caustics.* A number of computer programs are available which are capable of calculating a large number of ray paths over an irregular seabed.

These programs use Snell's law to calculate the ray paths and can be applied by either sending rays from deep to shallow water or by sending out a fan of rays from the inshore point of interest. The former, and more conventional, method will give information on wave heights and directions over a length of coastline, whereas the second method can be used to obtain detailed information concerning wave height and direction at a specific site.

With a realistic representation of the seabed contouring, it is not uncommon to encounter situations where neighbouring rays cross one another which implies a very large wave height as the value of b becomes small. The envelope of crossing rays is called a caustic. The theory has been extended for uniform waves to take account of caustics for special cases of seabed topography and it is found that diffraction effects become important in the vicinity of the caustic. The resulting wave height near the caustic varies rapidly and requires considerable computational effort even for simple cases.

A promising method of dealing with caustics lies in the use of a wave spectrum in refraction calculations. The effect of even a small spread of wave energy is to smooth out the rapidly varying solution found for uniform waves in the vicinity of a caustic. Since waves in the sea always possess some spread of energy over direction and frequency the solution obtained using a wave spectrum can be expected to be closer to the behaviour of the real sea.

In such cases the expression for the conservation of energy flux (see **23.2.1**) is replaced by:

$v_c v_{cg} l(f,\phi)$ = constant along a ray path

where

l is the density of the directional spectrum, which is related to the one-dimensional spectrum $S(f)$ by:

$$S(f) = \int l(f,\phi)\mathrm{d}\phi$$

Thus, by sending out a fan of rays from the inshore site of interest and using Snell's law to track them out to deep water the inshore spectrum $l(f,\phi)$ can be expressed in terms of the offshore spectrum $l_0(f,\phi_0)$, by:

$$l(f,\phi) = \frac{v_{c0} v_{cg0}}{v_c v_{cg}} \int l_0(f,\phi_0)$$

where

ϕ is the starting angle of the ray at the inshore position;

ϕ_0 is the finishing angle of the ray in deep water.

By sending out fans of rays for a range of wave frequencies, this expression can be used to build up the inshore wave spectrum as a function of the offshore spectrum. In the absence of detailed knowledge of the directional spread, it is usual to assume l_0 to be the product of a function of frequency with a function of angle. The frequency function can be based on the appropriate prediction for the offshore wave frequency spectrum (see **22.2.5**) and the function of angle is usually taken to be a symmetric function with its peak at an angle corresponding to the mean offshore wave direction. The mean inshore wave direction, ϕ_m, can then be obtained from:

$$\phi_m \int S(f)\mathrm{d}f = \frac{1}{(\phi_2 - \phi_1)} \int_{\phi_1}^{\phi_2} \int \phi\, l(f,\phi)\, \mathrm{d}f\, \mathrm{d}\phi$$

where

ϕ_1 and ϕ_2 are the limiting ray directions, i.e. typically the directions for which rays begin to turn back shoreward just before reaching deep water:

$S(f)$ is the one-dimensional inshore spectrum given by:

$$S(f) = \int_{\phi_1}^{\phi_2} l(f,\phi)\, \mathrm{d}\phi$$

This method, which uses backward ray projection and a wave spectrum, allows the mean wave direction and the one-dimensional wave spectrum to be obtained at a specific inshore site after wave refraction and shoaling have taken place. This method has been incorporated into a computer program [23] and is in common use in situations where accurate predictions of the effect of refraction on inshore wave conditions are required. One of the more obvious effects of refraction on the wave spectrum is to reduce the directional spread as waves propagate into shallow water, which is clearly to be expected, since the wave crests tend to align themselves with the bottom contours.

23.2.3 *Entrance channel effects.* With the development of large-draught vessels such as supertankers, the need for a deeply dredged entrance channel sometimes arises. In such a situation Snell's law implies that waves approaching at certain angles will be refracted to such an extent as to be unable to cross the dredged channel.

It is clear that the critical angle of approach will increase for shorter wave periods, thus the range of angles of waves unable to cross the channel will be smaller for shorter wave periods and larger for longer wave periods.

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Because such deep channels are a relatively recent requirement there are insufficient field measurements available to establish the extent to which this simple refraction theory is applicable. It is to be expected that diffraction will become important along the edge of the channel on the side of wave approach because the crossing of neighbouring rays, as they are reflected back from the channel, indicates the presence of a line caustic along the channel edge and, in the case of extreme conditions, wave breaking may well occur in this region. Also, since wave energy is distributed over a range of angles, some wave components will always be able to propagate across the channel. All these effects will tend to give rise to some wave action in the channel but the resultant effect could still be a useful reduction in wave height in those cases where Snell's law predicts ray reflection for a significant range of angles and wave periods.

23.3 Bottom friction. By analogy with steady flow through a pipe, where friction losses effectively cause a resistance to flow along the pipe, waves can be expected to meet an effective resistance to their orbital motion near the seabed as they propagate into shallow water. This frictional force per unit area, F_b, takes the following form:

$$F_b = K_b \rho u^2$$

where

K_b is the bed friction factor;

ρ is the fluid density;

u is the horizontal orbital velocity at the sea bed calculated in the absence of bed friction.

By taking the product of this force with u and averaging over a wave period, an expression is obtained for the amount of energy dissipated per unit area in unit time and this may be equated to the power lost per unit length along a ray path. The resulting equation can be solved to give an expression for the wave height reduction factor, K_f, due to bed friction, which is plotted in Figure 12 for the special case of a seabed of constant slope s, expressed as the tangent of the angle between the bed and the horizontal. Any refraction effects have been ignored and the coefficient K_f does not include the change in wave height due to wave shoaling.

For typical parameters it can be seen from Figure 12 that it is only necessary to take bottom friction into account when information on wave parameters is required in very shallow water such as may be needed to determine the effect of wave action on beach erosion. For structures standing in, for example, 10 m of water, the effect of bottom friction on frequently occurring wave heights may be ignored in most cases. Bottom friction can, however, be an important factor in attenuating the height of severe wave conditions with long return periods.

If the use of Figure 12 indicates that bottom friction is of importance, then difficulties arise in obtaining accurate estimates of its effect due to the problem of assigning a realistic value to the bed friction factor, K_b.

In most situations where bed friction is important it is thought that the wave energy is being lost to turbulent water movement generated as the water particles oscillate over ripples on a sandy bottom. As these sand ripples are formed by the waves themselves with ripple heights that are probably a function of the wave parameters it can be seen that the bed friction factor can be expected to vary from storm to storm. In order to make estimates of the bed friction factor from field observations it is first necessary to extract all other shallow water effects, such as refraction and shoaling, from the data. The few studies of this kind which have been made indicate a bed friction factor that varies considerably with some values an order of magnitude larger than the often quoted figure of 0.01. Average values of 0.04 to 0.06 have been obtained. Some of these variations may be due to errors in extracting other shallow water effects from the data.

Clearly it is necessary, at present, to use field measurements of waves to obtain accurate predictions of the inshore wave climate in situations where bed friction is important. However, in the absence of site measurements a conservative estimate of wave height will be obtained by taking a bed friction factor of 0.01.

23.4 Wave breaking. In deep water waves break before reaching a limiting wave steepness (see clause **21**) but in shallow water it is necessary to clarify the different types of wave breaking that can occur. The breaker type depends on both the initial wave energy, which can be characterized by the offshore wave steepness (H_0/L_0) and the rate at which the height of the wave changes as it propagates into shallow water, which, ignoring refraction and bottom friction effects, will be dependent on wave shoaling and can be characterised by the beach slope, $\tan \alpha_s$.

When J, the ratio of the offshore wave steepness to the square of the beach slope, is in excess of about 5 then a spilling breaker, characterized by white water on its forward face and gradual breaking, will normally occur. This type of breaker generally occurs for a ratio of wave height to water depth in the range of 0.7 to 0.9.

When the ratio J is less than about 5 but greater than about 0.1 then plunging breakers, characterized by a curling over of the top of the crest which plunges down towards the preceding trough, will generally occur. This type of breaker can reach heights in excess of the depth and as the velocity of the water in the plunging crest is in excess of the phase velocity of the wave, very high loadings can be produced when these waves break on to a structure.

When the ratio J is less than about 0.1 then surging breakers tend to occur. As this type of wave steepens, its base surges up the beach slope resulting in the virtual disappearance of the wave crest. The transition between plunging and surging is sometimes called a collapsing breaker.

Clearly as the beach slope becomes infinite and tends towards a vertical face the ratio J will become zero and wave reflection will occur.

In general the transitions between the various types of breaker are gradual and the ranges given above for the ratio J can only be used as a very rough indicator.

Various empirical attempts have been made to determine a maximum breaker height in a given depth of water but the large scatter in the data makes the resulting relationships unreliable. In those situations where the inshore structure is subject to breaking waves, the following wave parameters, which will give an upper limit to the wave height, may be used, but only as a general indicator of possible wave conditions.

a) Where the prediction for the design significant wave height in the depth of water at the structure exceeds that water depth and the ratio J is less than or equal to 5, then assume a significant wave height equal to that water depth.

b) For more gentle slopes in front of the structure where the waves can be expected to spill, i.e. J greater than 5, and the design significant height exceeds 0.8 times the still water depth, then assume a significant wave height equal to 0.8 times that depth.

In general this procedure is conservative in that it is likely to lead to overestimates of wave height, particularly when combined with the extrapolation process described in **27.3**, and should not be used in wave breaking situations where accurate estimates of wave heights are required.

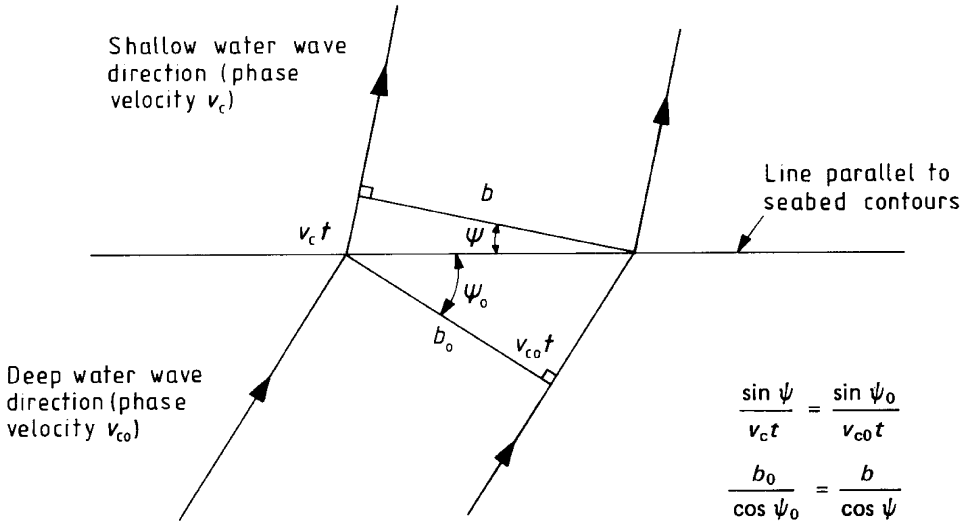

Figure 10 — Schematic diagram of wave refraction

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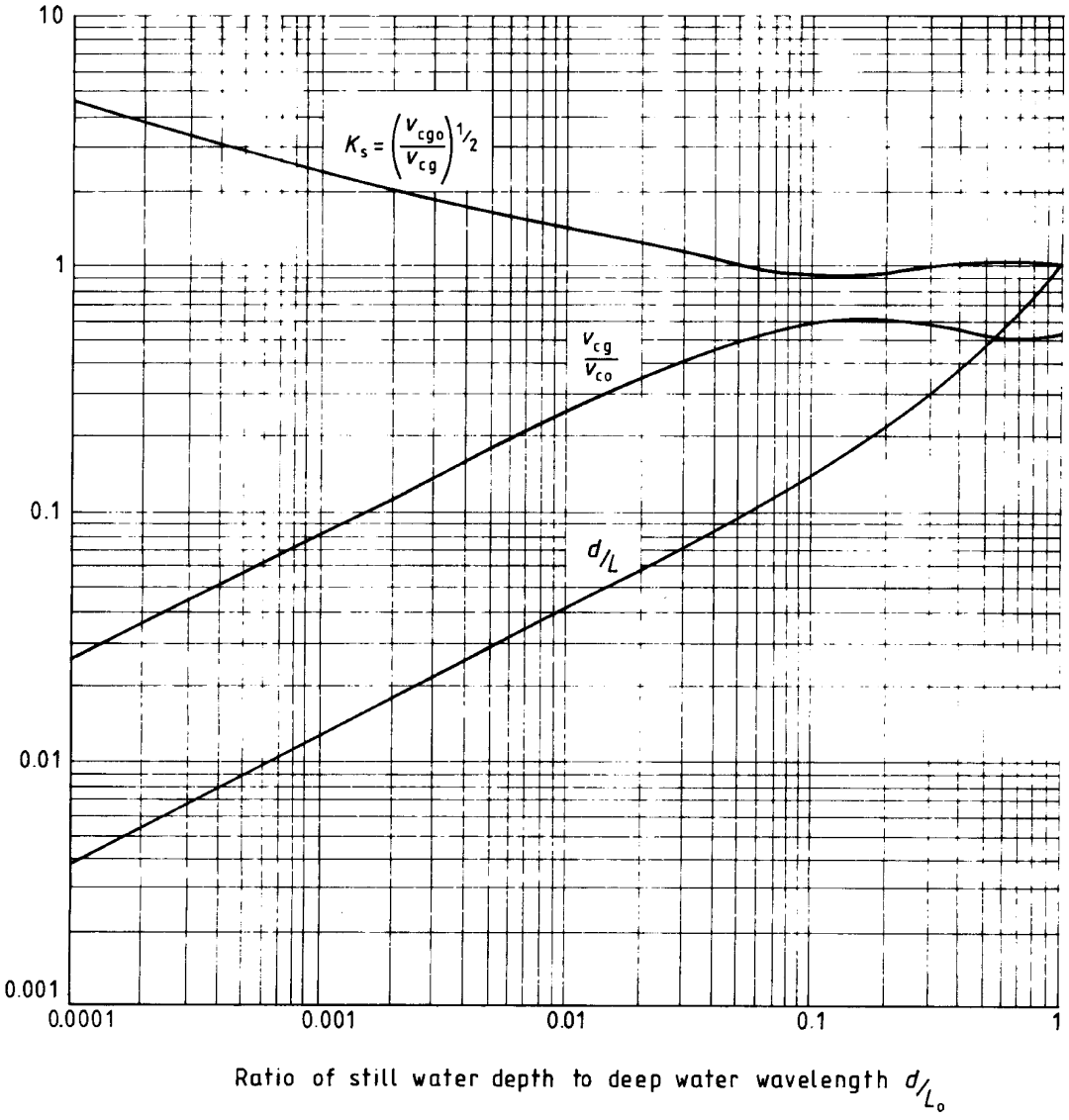

Deep water wavelength $L_0 = gT^2/2\pi$
Deep water phase velocity $v_{c0} = gT/2\pi$

(a) Variation of wavelength, group velocity and shoaling coefficient with water depth

Figure 11 — Wave shoaling and estimation of wave height in the surf zone

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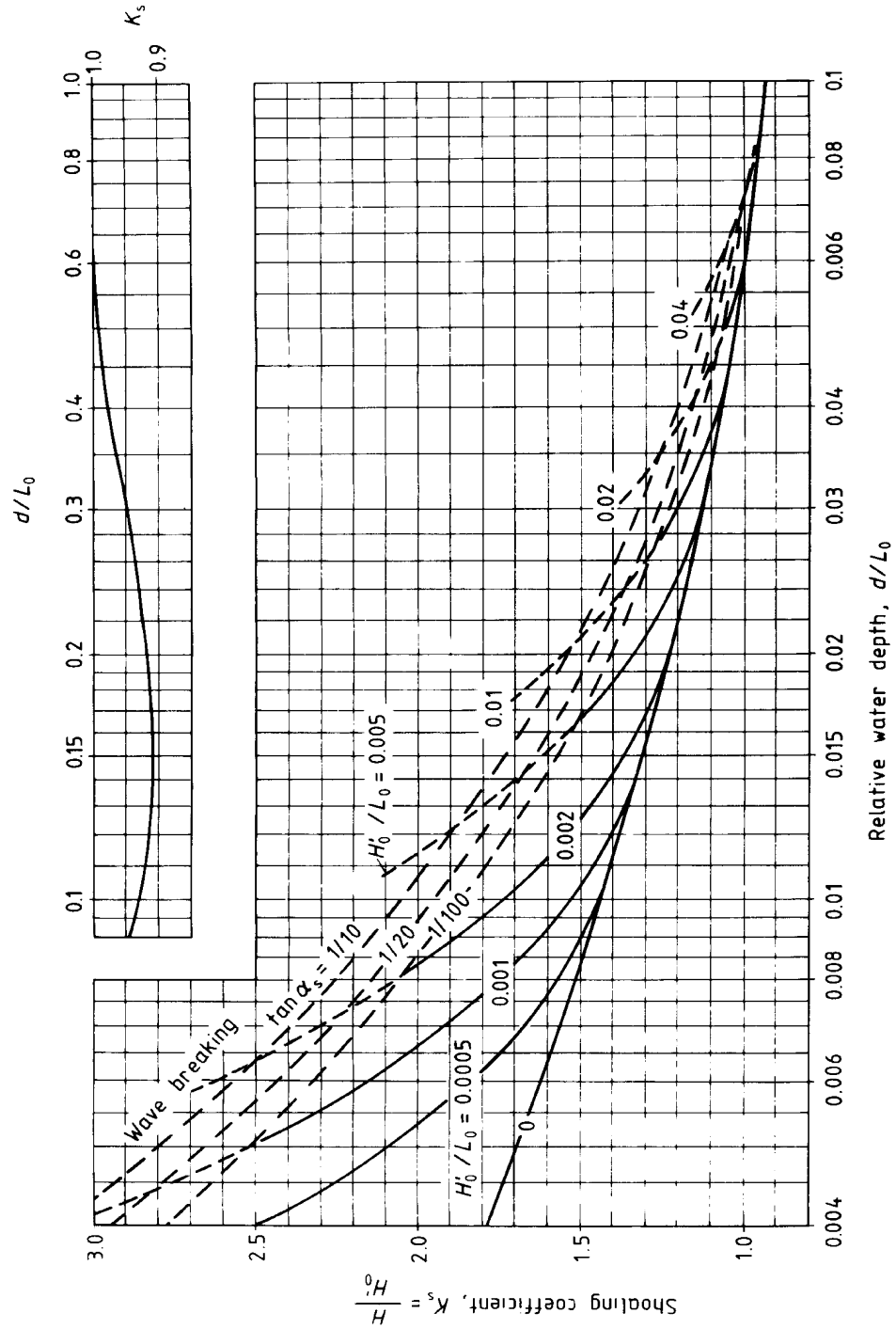

(b) Diagram of nonlinear wave shoaling

Figure 11 — Wave shoaling and estimation of wave height in the surf zone *(continued)*

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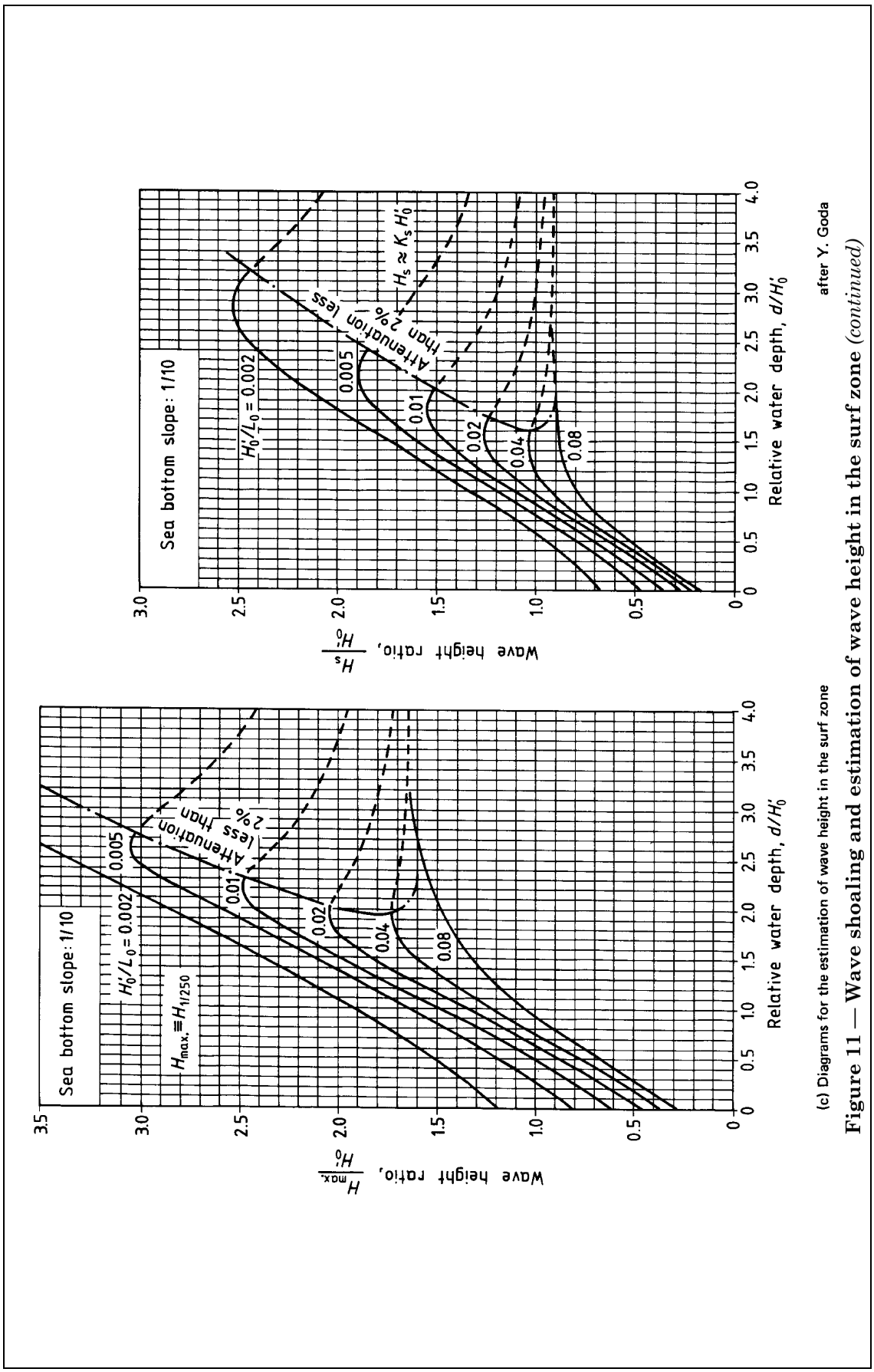

(c) Diagrams for the estimation of wave height in the surf zone

Figure 11 — Wave shoaling and estimation of wave height in the surf zone *(continued)*

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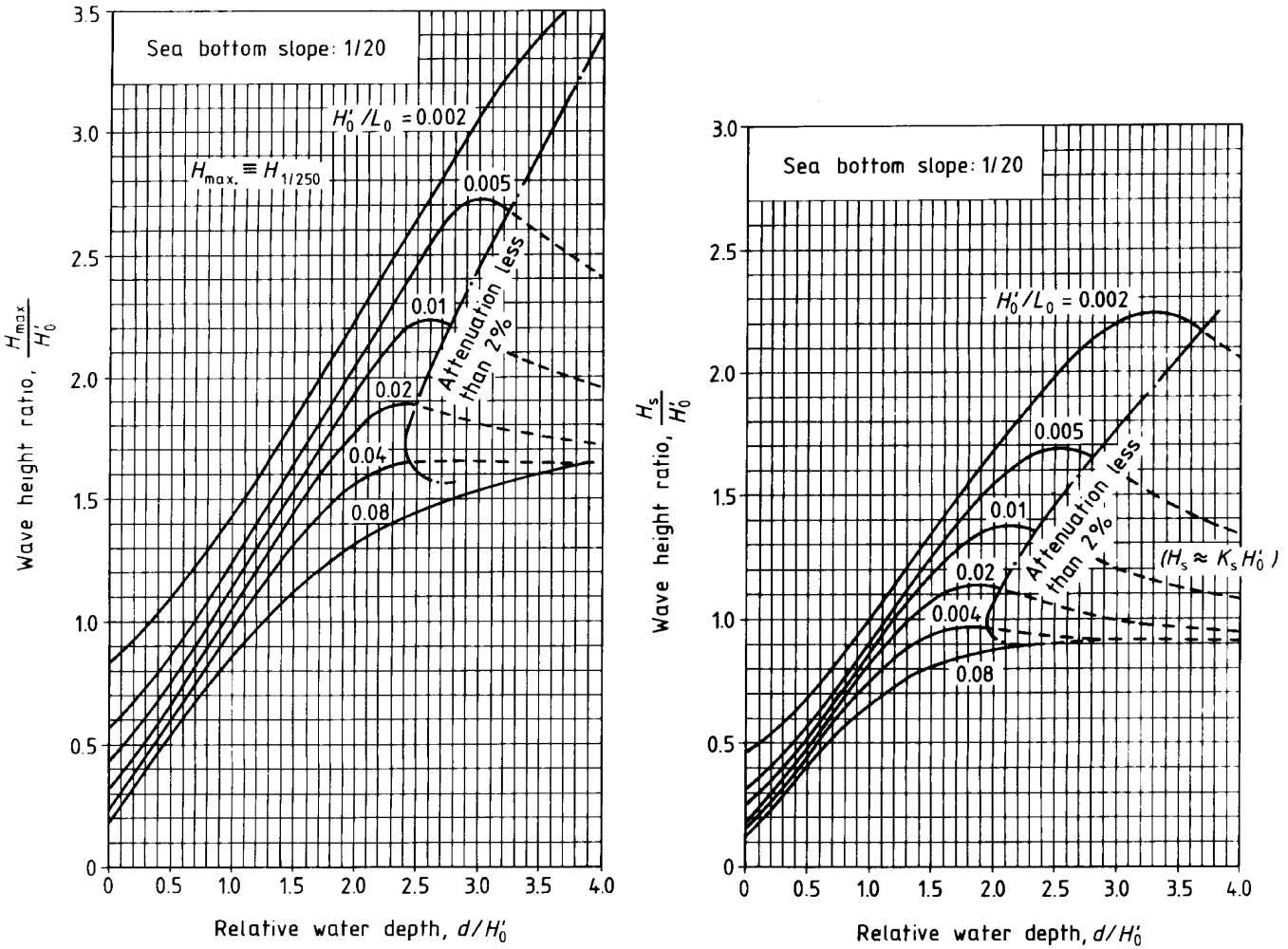

(d) Diagrams for the estimation of wave heights in the surf zone (sea bottom slope of 1/20) after Y. Goda

Figure 11 — Wave shoaling and estimation of wave height in the surf zone *(continued)*

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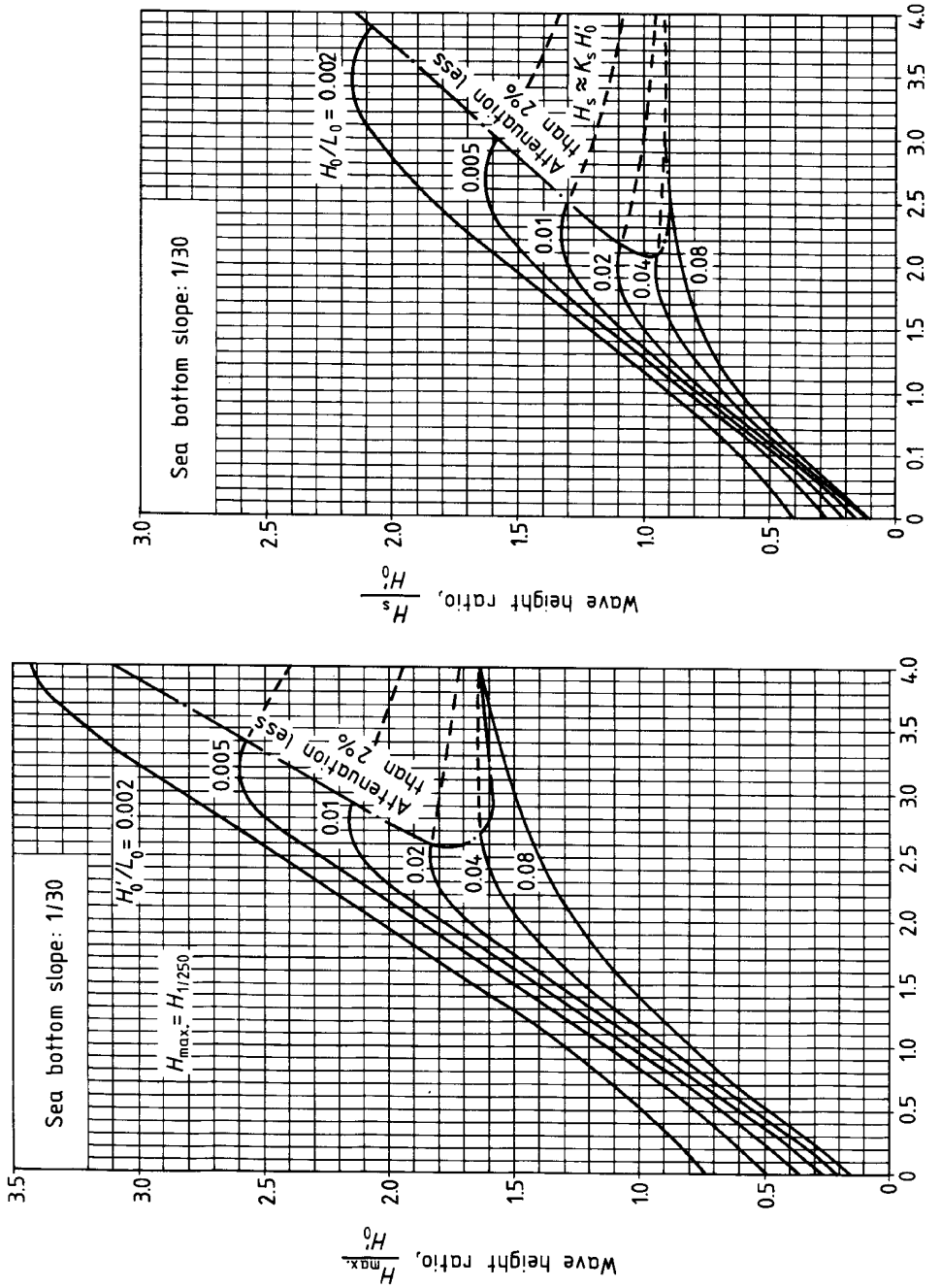

(e) Diagrams for the estimation of wave height in the surf zone (sea bottom slope of 1/30)

Figure 11 — Wave shoaling and estimation of wave height in the surf zone *(continued)*

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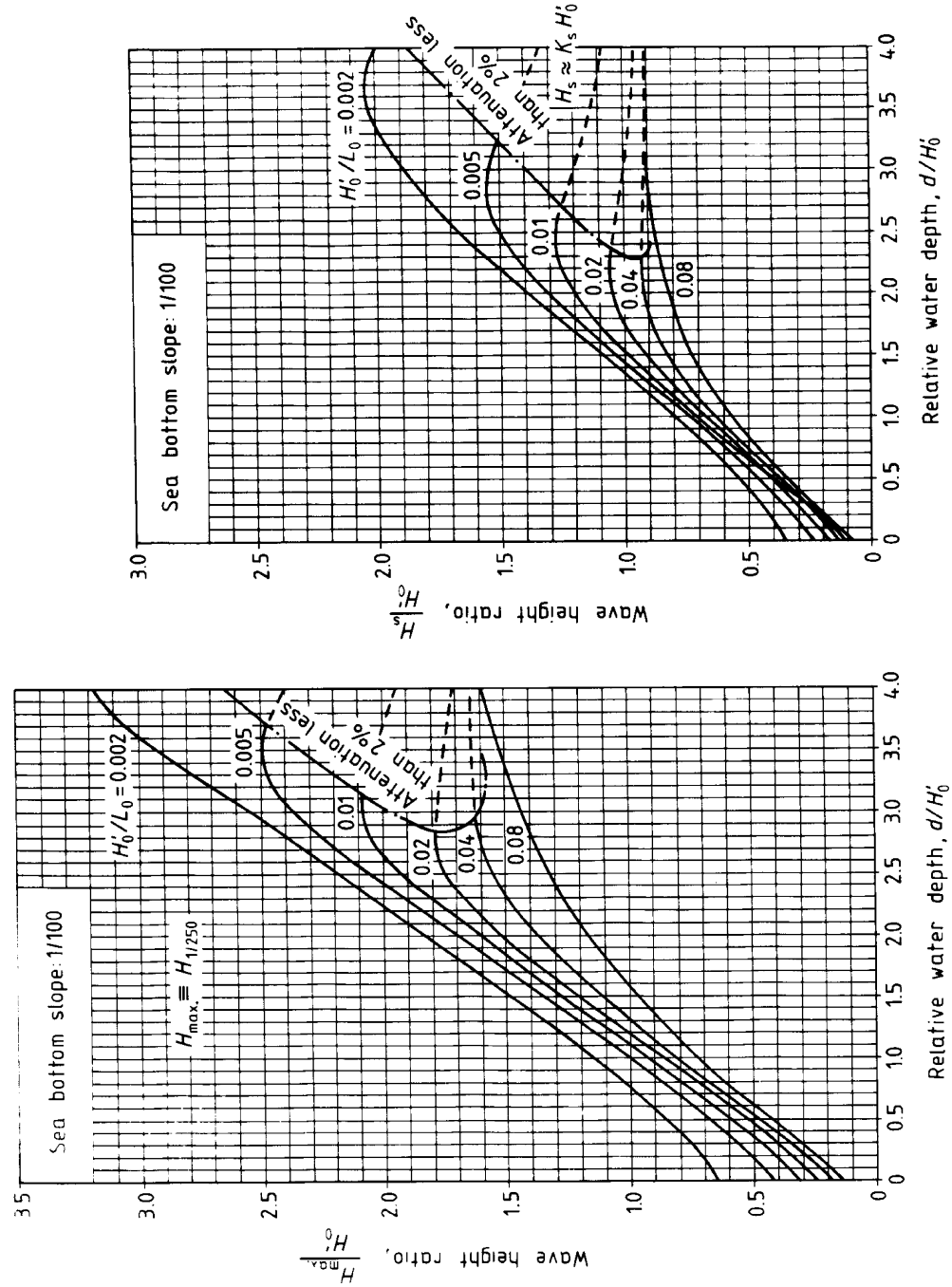

(f) Diagrams for the estimation of wave height in the surf zone (sea bottom slope of 1/100)

Figure 11 — Wave shoaling and estimation of wave height in the surf zone *(concluded)*

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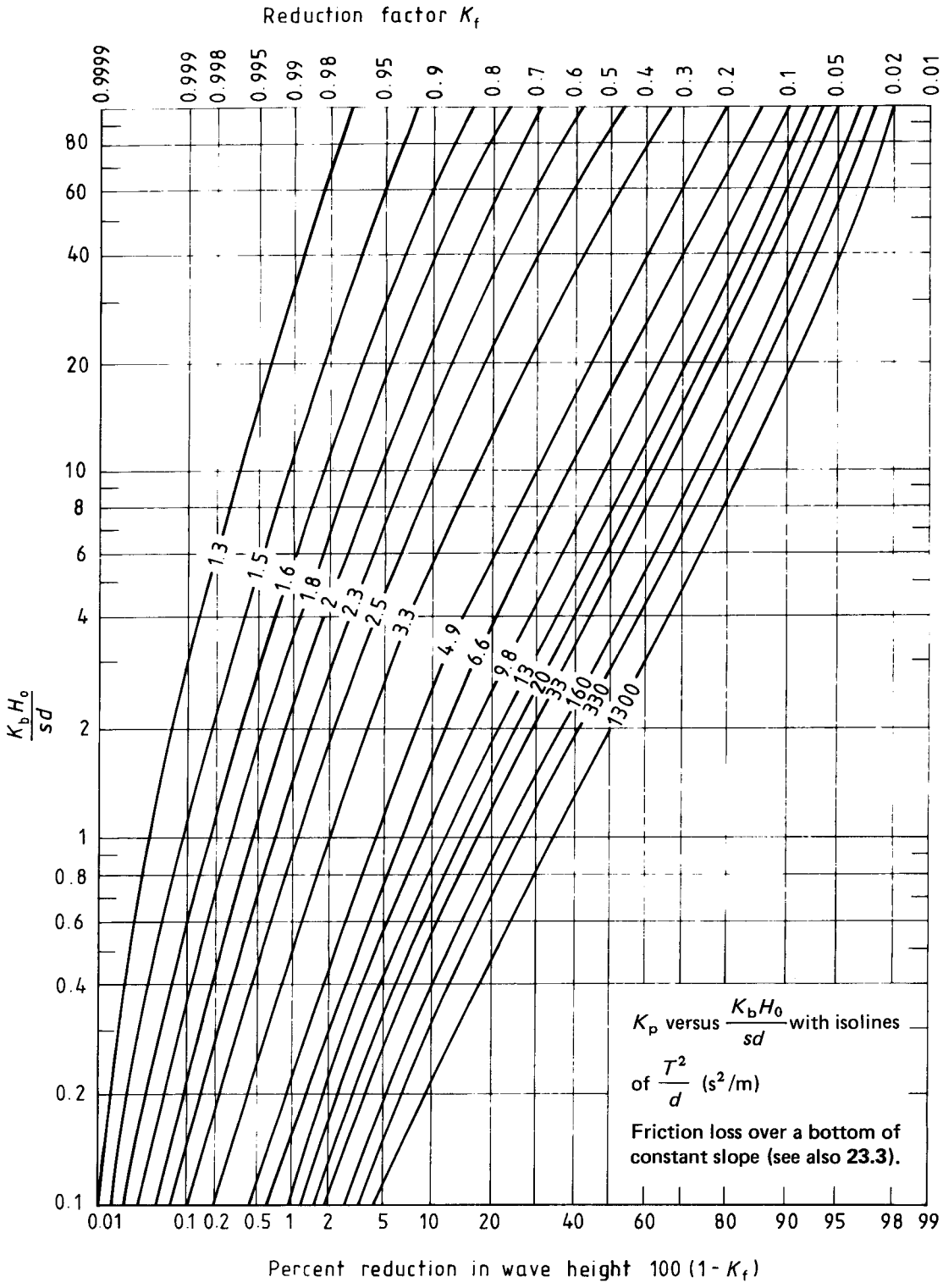

Figure 12 — Wave height reduction factor for bottom friction

Model tests with irregular waves plunging on a beach slope of 1 in 30 in front of a rubble-mound seal-wall indicate that the limiting significant wave height is approximately equal to the still-water depth with maximum wave heights up to about twice this depth. However, the mean water level in front of the wall was found to fluctuate, being higher when groups of high waves impinged on the sea-wall. This increase in mean water level accompanying high waves may be one factor operating in random waves that leads to maximum wave heights in excess of the still water depth.

The often quoted figure of maximum wave height being equal to 0.78 times the still water depth can be derived from theory describing individual waves but sufficient differences exist in models between results with random waves and results with individual waves to indicate that this is not an adequate estimate of breaker height in all situations.

A method has been developed in Japan from the combined results of model tests and prototype observations which may be used to estimate, for a random sea, wave heights in the surf zone and to seaward of that zone. The equivalent deep water wave steepness H_0'/L_0, the bottom slope and the relative water depth d/L_0 have been taken as parameters against which the maximum wave height and significant wave height, each normalized by the equivalent deep water wave height, are plotted. For the purpose of the remainder of this clause the maximum wave height shall be taken to be the mean height of the 0.4 % highest waves ($H_{1/250}$). An approximate relationship between $H_{1/250}$ and H_s is $H_{1/250} = 1.8\ H_s$. Figure 11(c) to Figure 11(f) are plotted for bottom slopes of 1/10, 1/20, 1/30 and 1/100. Each figure contains a dash-dot curve labelled "Attenuation less than 2 %". In the zone to the right of this curve the attenuation in wave height due to wave breaking is less than 2 % and the wave height can be estimated from the shoaling coefficient given in Figure 11 (b).

Equivalent deep water wave height is defined as the wave height at the point in question corresponding to the significant wave height in deep water and is given by:

$$H_0' = K_d K_r (H_s)_0$$

The period of the equivalent deep water wave is assumed equal to the deep water significant wave period.

$$T_s = (T_s)_0$$

Thus H_0' will, in general, vary and will be different for each geographical position considered.

The wave height may be estimated using the following equations:

If $d/L_0 \geqslant 0.2$ $\quad H_s = K_s\ H_0'$

If $d/L_0 < 0.2$ $\quad H_s$ is the lowest of the following:

$H_s = \beta_0\ H_0' + \beta_1\ d$ or

$H_s = \beta_{max}\ H_0'$ or

$H_s = K_s\ H_0'$

$$\beta_0 = 0.028 \left(\frac{H_0'}{L_0}\right)^{-0.38} \exp\ [20 \tan^{1.5} \alpha_s]$$

$$\beta_1 = 0.52 \exp\ [4.2 \tan \alpha_s]$$

β_{max} is the greater of the following:

$$\beta_{max} = 0.92$$

$$\beta_{max} = 0.32 \left(\frac{H_0'}{L_0}\right)^{-0.29} \exp\ [2.4 \tan \alpha_s]$$

$$\beta_0^* = 0.052 \left(\frac{H_0'}{L_0}\right)^{-0.38} \exp\ [20 \tan^{1.5} \alpha_s]$$

$$\beta_1^* = 0.63 \exp\ [3.8 \tan \alpha_s]$$

β_{max}^* is the greater of the following.

$$\beta_{max}^* = 1.65$$

$$\beta_{max}^* = 0.53 \left(\frac{H_0'}{L_0}\right)^{-0.29} \exp\ [2.4 \tan \alpha_s]$$

These equations may give estimated heights differing by several percent from those obtained from the graphs. In particular for waves of greater steepness than 0.04 in the water depth where:

$$\beta_0 H_0' + \beta_1 d = \beta_{max} H_0'$$

differences may exceed 10 % with a similar difference for $H_{1/250}$. There may also be a discontinuity in $H_{1/250}$ at $d/L_0 = 0.2$.

24 Long waves

24.1 General. Long waves with periods of the order of minutes and heights which are generally under a metre have been measured just offshore at many sites around the world. They are of importance to large moored vessels because such vessels have natural periods of oscillation at their moorings which are generally longer than those of storm or swell waves but frequently similar to those of long waves. The result is that the moored vessel can move in resonance with long waves and, since the damping of such slow movements is small, large mooring loads can be developed.

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There are three possible causes of long waves:

a) moving pressure fronts;

b) wave grouping effects;

c) tsunamis.

24.2 Moving pressure fronts. By analogy with the mechanism of wind wave generation it is thought that air pressure pulses associated with moving atmospheric pressure fronts are capable of generating long waves when they propagate at the requisite phase velocity. The resonant periods of motion of enclosed bodies of water such as lakes and harbour basins are also thought to be capable of excitation by pressure fronts moving overhead. In this case the excitation is similar to the ringing of a bell where an initial disturbance is set up which gradually dies away. If the damping of such motion is small the disturbance will still be present some time after the pressure front has passed.

24.3 Wave grouping effects. The second possible source of long wave generation is associated with wave grouping. When the various storm or swell wave components of the primary wave system come into phase with one another to form a group of large waves there is a corresponding increase in the kinetic energy of the orbital water particle movement. This leads to a reduction in the water pressure and, if the air pressure is assumed to be constant, the result is that a depression in the mean water level occurs beneath the group of large waves while a compensating rise in the mean level occurs between them. This surface perturbation induces a wave-like motion beneath the surface which enhances the original disturbance. This effect, known as set-down beneath wave groups, differs from a true long wave because it propagates towards the shore at the group velocity which is less than the phase velocity of a long wave of the same period.

Where the water depth to wavelength ratio is significantly less than one sixth, the amplitude of set-down increases so that its significant height, H_{sd} for a long crested primary wave system is given approximately by:

$$H_{sd} = 0.01\frac{gH_s^2T_z^2}{d^2}$$

where

g is the acceleration due to gravity;

H_s is the significant height of primary waves;

T_z is the zero-crossing period of primary waves;

d is the water depth.

On this basis, swell, which is typically long crested with a significant height and zero-crossing period of 1.5 m and 15 s, respectively, will have an associated set-down with a significant height of 0.34 m in a depth of 12 m. The average period of set-down will be associated with the inverse of the width of the one-dimensional wave spectrum. As yet there is little empirical information to support the above relationship for the height of set-down so that it should only be used as a general guide to its magnitude.

When the set-down reaches the coastline the primary wave system is generally dissipated but because of its long wave character the energy in the set-down is unlikely to be completely dissipated and long waves of similar period are likely to propagate back out to sea. Such seaward-going waves have been inferred from observations made just offshore of a beach where a correlation was found between the envelope of incoming swell and the trough of long period waves, but with a time lag between the two consistent with the time taken for the swell to propagate from the wave recorder position to the surf zone inshore, and for the long wave to return past the wave recorder. Such waves are generally called surf beats.

It seems likely that in many cases refraction will cause these seaward-going long waves to tend to turn back towards the land. Such effects may lead to the formation of edge waves. These waves, which propagate parallel to the coastline, have a height which decreases with increasing water depth so that the wave energy is effectively trapped by the coastline.

24.4 Tsunamis. These waves are generally caused by earthquakes. One of their main features is that, although their height in deep water may be relatively small, they grow to alarming proportions upon reaching the shore due to wave shoaling. For a given deep water height, the steeper the seabed slope, for naturally occurring slopes, and the less steep the tsunami, the higher the run-up. Resonance in bays and wave refraction can both produce local amplification of the tsunami height.

24.5 Conclusions. Observations of long waves just offshore generally show that their height is correlated with the height of the storm or swell waves. This indicates that, at sites not subject to tsunamis, wave grouping effects are the main cause of long waves. Thus they are likely to be important only for harbours and offshore terminals which are in relatively exposed locations or subject to swell waves with heights in excess of 1 m. Under these circumstances it becomes necessary to install a long wave recorder to obtain accurate estimates of long wave activity at the inshore site of interest (see clause **26**).

25 Storm surge

An important aspect in the design of maritime structures is the water level (see clause **10**). Storm surges can raise the water level to values above high tide levels. Therefore, some estimate of the combined effect of high tide, storm surge and wave action is essential, bearing in mind that the highest waves do not necessarily occur together with the highest water level.

Effects that can play a part in determining storm surge are wind set-up, reduced atmospheric pressure, rotation of the earth, coastline topography and storm motion. Of these the largest effect is generally produced by wind set-up. Wind blowing over the sea induces a surface current which can lead to a pile-up of water along the coastline. Clearly if the storm surge is forced to travel into a gradually narrowing area of sea between two land masses, the water level will be increased due to the funnelling effect of such a coastline. For seas of limited extent, storm motion effects may be capable of exciting the resonances of the sea basin thereby increasing the storm surge level. For large bays, it is possible for the natural modes of oscillation of the bay to be excited as well and this can further enhance the surge level.

Methods of predicting the increases in water level due to storm surge are given elsewhere [16] but, for the waters around the British Isles, it is common practice to subtract predicted tide levels from recorded tide levels at slack water to give positive or negative storm surges.

Since both wave height and water level are important parameters in many of the design factors for maritime structures, it is advisable, if possible, to establish from the available data whether large wave heights and abnormally high water levels are dependent or independent of one another. For the case where the two are inter-dependent it may be assumed that the significant wave height with a 50 year return period will occur with the abnormally high water level with a 50 year return period. If, however, the available data suggest that the two are independent and if advantage is to be gained from this independence, then more care is required in the specification of design conditions since it is possible for various combinations of water level and wave climate to lead to similar degrees of overtopping or to similar design wave heights for sea-walls and breakwaters. In such cases, account should be taken of all those various combinations which may give rise to similar design criteria.

In the absence of sufficient data for establishing whether or not high water levels and large wave heights are correlated, the assumption that they are dependent on one another should be made, although this may sometimes lead to conservative estimates of the height and strength of sea-walls and breakwaters required for a given degree of protection.

26 Wave recording and analysis

26.1 Existing data sources. There is a reasonable global coverage of wave data collected by volunteer observing vessels along the major shipping routes and stored by the UK Meteorological Office [22] but a marked scarcity of localized detailed information, particularly in coastal waters.

In order to facilitate the search for relevant instrumentally measured wave data, the Marine Information and Advisory Service (MIAS) at the Institute of Oceanographic Sciences has been designated as the Responsible National Oceanographic Data Centre for Waves and maintains an international inventory of such data.

The aim of MIAS is not necessarily to hold the raw data but to store and collate the addresses of institutions which do, together with the form of any data analysis that has been carried out and MIAS therefore may be consulted to establish whether data exist for the particular region or site of interest.

The use of any data not collected for the specific construction project should be treated with care, as water depth and exposure may well be different from the study site and shallow water effects, long-period oscillations and storm surges need to be considered.

26.2 Site observations. In the absence of suitable existing wave data, site measurements should be taken. Such site observations, which may be used alone or as a supplement to forecasting techniques to produce design wave parameters, are essential where:

a) complex seabed topography at the site would render transformation of forecast deep water wave characteristics unreliable;

b) meteorological data for wave forecasting at the site is inadequate or of doubtful quality;

c) the fetch is not readily amenable to forecasting procedures;

d) the presence of currents or tidal variations in the study area is likely to affect the wave characteristics significantly;

e) the presence of swell or long-period wave action is likely to be significant.

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Records can be taken visually and reasonable correspondence has been demonstrated between observers' reports and instrumentally measured wave heights and periods. The maximum wave height recorded from visual observations over a limited observation period may normally be taken as the significant wave height for that period. It is likely, however, that the maximum waves will be missed or inaccurately estimated even with frequent observation periods. Also the height of long period waves and swell cannot be estimated accurately by observer and this knowledge is sometimes important for harbour design and studies of moored vessels. A major advantage in visual observations is the ease with which sea and swell direction can be estimated (see also **26.3.4**).

Types of automatic wave recording equipment and their relative advantages are described in **26.3**. The instruments should be appropriate to:

1) the tidal range;

2) the expected maximum wave heights and periods;

3) the degree of exposure of the recording locations to marine hazards, especially proximity to shipping.

Whichever type is chosen, it is important that the instrument should be robust and suitable for the maritime environment, as well as being accurate and properly designed. Experience has shown that wave recorders are particularly liable to malfunction both due to inherent faults and independent interference such as disturbance by passing vessels. The system should therefore be checked regularly. Although many of the newly developed instruments appear to offer advantages over the older systems, a well tried robust instrument is generally more reliable and therefore likely to produce a higher data return.

Where wave recording is to be carried out, it is advisable to install the recording system at an early stage to enable the recording programme to be as long as possible. A minimum of 1 year's records is essential to enable reasonably reliable extrapolation of design parameters to be made. Recording periods of 10 min to 15 min at intervals of 3 h are usually sufficient for derivation of design storm or swell wave data.

In those situations where long waves with periods of the order of minutes are also being investigated (see clause **24**) it is necessary to record for at least 1 h to 2 h to obtain reasonable spectral estimates of long wave activity. As these long waves are generally correlated with high storm or swell wave activity, it is often sufficient to arrange for the data logging system to switch from the periodic recording mode to the continuous recording mode for 1 h to 2 h only when the primary wave heights exceed some particular value.

Extrapolation is sensitive to the inclusion of extreme waves in the records. The meteorological records (see clause **7**) should be checked to ensure that representative storms have occurred in the relevant fetch areas during the recording programme.

It is important that only a complete set of recordings should be used for extrapolation to obtain extreme wave conditions. Recorders are often damaged during storms and to obtain 12 complete months' records covering January to December it is often necessary to record for more than 1 year.

To determine the general offshore wave climate, the wave recording instrument should be sited in deep water, relative to the wavelengths, adjacent to the study area, if possible in a position from which wave orthogonals can be easily transferred to the points of interest inshore. The recorder should be sited away from local reflective and diffractive features and from uncharacteristically strong currents, which may affect the wave climate.

For the study of conditions peculiar to specific locations or effects, the wave recorder should be sited at or close to the location. Particular care is necessary for the siting when investigating resonance or interference wave patterns, in order to obtain truly representative records.

26.3 Wave recorders

26.3.1 *Surface-mounted systems.* Buoy-mounted accelerometers have come into wide use in recent years and the more robust models of this type have probably the highest rates of success of all the different types of recorder for waves with periods of from 3 s to 20 s. One disadvantage is that the instrument is not sensitive enough to measure the vertical accelerations caused by long waves with periods of 1 min or more.

Resistance and capacitance staff recorders are capable of measuring waves with periods ranging from seconds through to tidal cycles. Their main disadvantage is that they require a fixed structure for support.

The float recorder or modified tide gauge has proved useful in the past for making surface measurements but its response to storm and swell waves is non-linear. If the difficulties of mounting and regular cleaning out can be overcome, the recorder will give a continuous and accurate record of long wave activity for waves with periods longer than 30 s.

26.3.2 *Sub-surface systems.* The pressure transducer is probably the next most successful type of wave recorder after the buoy-mounted accelerometer. Pressure fluctuations measured at a particular water depth can be related to the value of the water surface fluctuations. Linear wave theory should give no more than ± 10 % error in the calculated water surface movement.

The transducers are capable of measuring waves with periods ranging from seconds through to tidal cycles but the large hydrodynamic attenuation of short waves with increasing water depth means that care is required in choosing the water depth for measurement and large tidal ranges may limit their use.

Inverted echo sounders mounted below the surface are also capable of measuring waves as they pass overhead but the measurements are unreliable for storm waves where the surface becomes aerated, causing scattering of sound instead of reflection back towards the seabed.

26.3.3 *Above-surface systems.* Radar and laser beams from satellites, aircraft or from ground-based stations may be used to measure waves. These techniques show promise for both wave direction and height measurement. The major advantages of these techniques are in the wide spatial coverage possible from one installation and the fact that no equipment has to be deployed in the sea. However, they are still under development and are not yet used for routine data collection.

26.3.4 *Directional systems.* Routine measurements of the directional properties of waves are not made at present but a number of methods are under development. Pitch, roll and heave buoys can give the mean wave direction accurately and an estimate of the directional wave spectrum. An array of sensors measuring wave height can also be used to obtain the directional spectrum. Some directional information can also be obtained from the back-scattering of radar or microwaves from sea waves. Electromagnetic current meters, capable of measuring the two horizontal components of the orbital velocity in waves, can be used to obtain directional information.

Until techniques for the automatic recording of wave direction are sufficiently advanced, resort should be made to visual observation, local knowledge and to inspection of synoptic weather charts.

26.4 Analysis of records

26.4.1 *General.* In the past, intermittent wave records were normally produced in the form of a pen trace on a continuous chart roll which would then be processed manually (see **26.4.2**). Chart recordings may also now be converted into digital data and analysed on a computer to produce the required parameters (see **26.4.3**).

The hand analysis of chart recordings is time consuming and is now being replaced by magnetic tape recording and direct computational analysis. It is wise, however, to retain chart recordings, where possible, as a back-up system because they may be used to identify malfunctions in the equipment.

26.4.2 *Manual analysis.* Chart recordings, usually lasting from 12 min to 20 min and taken every 3 h or 4 h, may be processed manually as described in this subclause.

A mean line is drawn through the wave record and the number of up-crossings, N_z, of the mean line in a 10 min interval is noted. An estimate of the zero-crossing wave period, T_z, is then obtained by dividing 600 s by N_z. The height of the highest crest, H_A, and the next highest crest, H_B, above the mean line are noted for the 10 min interval. Similarly the depth of the lowest trough, d_A and the next lowest trough, d_B, below the mean line are noted. An estimate of significant wave height, H_s, may then be obtained from the following expression:

$$H_s = \frac{H_{s,1} + H_{s,2}}{2}$$

where

$$H_{s,1} = H_1 \left(\frac{2}{\lambda}\right)^{1/2} (1 + 0.289\,\lambda^{-1} - 0.247\,\lambda^{-2})^{-1}$$

$$H_{s,2} = H_2 \left(\frac{2}{\lambda}\right)^{1/2} (1 - 0.211\,\lambda^{-1} - 0.103\,\lambda^{-2})^{-1}$$

where

$H_1 = H_A + d_A$

$H_2 = H_B + d_B$

$\lambda = \log_e (N_z)$

In the above expressions, H_1 and H_2 have to be corrected for the frequency response of the recorder and, in the case of pressure recorders mounted below the surface, it is also necessary to correct for hydrodynamic attenuation with depth. These correction factors are functions of wave period and it is usual to use the factors associated with the zero-crossing period, T_z.

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It should be noted that pressure recorders will tend to overestimate the zero-crossing period of the surface elevation because the shorter periods in the wave spectrum will undergo greater attenuation with depth than the longer periods. This problem can be overcome by carrying out a spectral analysis of the pressure record (see **26.4.3**) and correcting the resulting spectrum frequency by frequency for depth attenuation and for instrument frequency response, if necessary, to obtain an estimate of the spectrum of the surface elevation. The zero-crossing wave period may then be obtained from the expression for T_z given in **27.3.1**.

The analysis described above will give an estimate of H_s and T_z for each record which may then be extrapolated to give significant wave heights for the required return period and, when required, may be further extrapolated to give design maximum wave heights for various probabilities. Alternatively the average maximum wave height occurring between records may first be estimated by assuming that the wave train obtained from the 10 min record persists throughout the subsequent 3 h or 4 h interval and the resulting maxima then extrapolated to give the required design condition (but see **27.2.1**). Methods of extrapolation are described in clause **27**.

26.4.3 *Computational analysis.* Chart records may also be analysed using a curve-following machine which converts the chart data into digital data which can be processed on a computer to obtain the one-dimensional wave spectrum. Where the information is directly recorded on magnetic tape in digital form, spectral analysis can be carried out more rapidly.

27 Extrapolation of wave data

27.1 General. Extrapolation processes applicable to wave height predictions may be divided into the following two types:

a) extrapolation to provide an extreme wave height with the required return period from a representative set of statistically independent wave heights (see **27.2**);

b) extrapolation to provide a maximum wave height with the required probability of occurrence from a single height representative of the wave train, usually the significant height (see **27.3**).

In a) the initial information may comprise a set of significant wave heights or it may comprise a set of maximum wave heights already obtained from the significant heights by individual extrapolations of the type described in b); however it is important that the initial data set should be statistically independent, consistent and representative of typical conditions (see **27.2.1**).

Extrapolation of the associated wave period is usually achieved by consideration of the wave steepness (see **27.4**).

27.2 Extrapolation to extreme wave conditions

27.2.1 *Reliability of extrapolations.* Estimates of extreme conditions obtained by extrapolation rely on the year or years for which observations were used being typical. If storms were particularly severe or mild during the observation period then the extrapolations will give overestimates or underestimates of extreme values. However, the extrapolations should become more reliable if the original data are derived from a greater number of years. Similarly if the original records are incomplete through, for instance, malfunction of a wave recorder or failure to obtain regular data from a full 12 month cycle, then bias may be introduced.

The original data set should be statistically independent and care should be taken that this requirement is not overlooked in an attempt to increase a limited data base by including non-independent observations. Where a set of wave records for a full year are available it has been common practice to extrapolate the maximum height for each 3 h or 4 h interval between recordings by the methods given in **27.3** thus providing a data base in excess of two thousand values. This large range of values could be plotted and extrapolated as described in **27.2.2**. However, there is doubt about the validity of the results since the original values applying at every 3 h or 4 h interval in the year cannot be considered as statistically independent.

This difficulty may be overcome if the original data set is reduced to include only the maximum wave heights associated with the peak of a storm, since storms may be considered as statistically independent events. If the number of storms in the year is small the resulting extrapolation will again become unreliable, in which case it is reasonable to take the largest values of maximum wave height occurring at weekly intervals as being a statistically independent set.

Extrapolation techniques assume that the wave-generating mechanisms remain constant in the long term. For this reason extrapolations to return periods in excess of say 100 years should be viewed with caution since their reliability may be affected by long term changes in the climatic pattern.

27.2.2 *Wave heights.* The method consists of plotting the initial wave heights against their cumulative probabilities of occurrence, using an appropriate probability function, with the object of achieving a straight line graph which may then be extended to give an estimate of the occurrence of extreme conditions.

For a set of n_x values of representative heights H tabulated in increasing order of magnitude the probability that H is less than an individual value H_n (where n is less than or equal to n_x) may be denoted by $n/(n_x + 1)$; therefore the probability, p_n, that H_n is equalled or exceeded is given by $p_n = 1 - [n/(n_x + 1)]$.

Values of p_n may be calculated directly by the above expression for each individual height in a limited set of data, but for large sets of data it is more convenient to subdivide the arranged set of heights into a number of equal height intervals. For each height one count is recorded in the appropriate interval and one in each of the lower intervals. The total number of counts within an interval divided by the total number of observations gives the probability p_n of the wave height H_n being equalled or exceeded, where H_n is the height defining the lower limit of the interval under consideration.

A number of probability distributions have been found to be appropriate in different situations. It may sometimes be found that one distribution will fit the lower wave heights well while another distribution will fit the higher wave heights better, possibly indicating two different wave populations. In these cases the distribution with the best fit to the larger waves should be used for extrapolation. The following distributions may be appropriate.

a) *Weibull distribution.* Plot $\log_e\log_e(1/p_n)$ against $\log_e(H_n - H_L)$.

b) *Fisher-Tippet distribution.* Plot $-\log_e\log_e[1/(1 - p_n)]$ against $-\log_e(H_L - H_n)$.

c) *Frechet distribution.* Plot $-\log_e\log_e[1/(1 - p_n)]$ against $\log_e(H_n - H_L)$.

d) *Gumbel distribution.* Plot $-\log_e\log_e[1/(1 - p_n)]$ against H_n.

e) *Gompertz distribution.* Plot $\log_e\log_e(1/p_n)$ against H_n.

f) *Log-normal distribution.* Plot H_n against p_n on the appropriate log- probability graph paper. Otherwise plot y against $\log H_n$ where y may be obtained, with the aid of tables, from

$$p_n = 0.5 - (2\pi)^{-1/2} \int_0^y \exp(-t^2/2)\, dt$$

For distributions a), b) and c) the value of H_L, which represents a lower or upper limiting value of H_n should be chosen by trial to give the best fit.

The use of appropriate graph papers will reduce the necessary computation.

The resulting straight line plot gives the probability of certain wave heights being equalled or exceeded during the period over which the original set of data were obtained. Provided the original set are statistically independent and also representative of typical conditions then extrapolation to increasingly lower probability values, and hence longer return periods, may be made. Since, by definition, the return period is that period during which the event will occur only once on average (see **21.4**) it follows that the maximum height H_{n_x}, obtained from n_x values representative of a period of observation T_o, will have a return period equal to T_o and, by substituting n_x for n in the expression for p_n given above, a probability of $1/(n_x + 1)$. Therefore the wave height with a given return period T_R will have a probability of occurrence of T_o/T_R $(n_x + 1)$ on the probability plot which may then be used to obtain the relevant extreme wave height.

In practice it is desirable to use a minimum of a full year's set of records from which to abstract the necessary data since any shorter duration is unlikely to yield a representative set.

Figure 13 shows a typical example in which the maximum significant wave heights from each of 39 storms occurring over a period of 1 year have been plotted to a Weibull distribution enabling estimates of 7.3 m and 7.9 m to be extrapolated for the significant wave heights with return periods of 50 years and 100 years, respectively.

Extrapolations of wave heights measured at inshore sites may be made using the above techniques but since the wave climate may be limited by breaking criteria (see **23.4**), the Fisher-Tippet distribution, in which the wave heights are restricted by an upper limit, may provide the best straight line fit on the probability plot.

27.3 Extrapolation to individual maximum wave heights

27.3.1 *General.* Analysis of wave data has shown that reliable estimates of maximum wave heights can be extrapolated from the significant wave height if the zero-crossing wave period and the duration of the design condition are known. The assumption is made that the sea state is steady in the sense that the average wave height and period remain constant for the duration of the design condition.

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The extrapolations made to obtain the design conditions will generally give the duration to be associated with the design significant wave height. The number of zero-crossing waves, N, occurring over the duration of the design condition may then be obtained by dividing the duration by the zero-crossing wave period, T_z. If the method of extrapolation used to obtain the design wave height does not yield a value of the design T_z then the following estimate may be obtained from the area and second moment of area of the one-dimensional wave frequency spectrum $S(f)$, according to the following expression:

$$T_z = \left[\frac{\int S(f)\, df}{\int S(f) f^2\, df} \right]^{1/2}$$

For storm waves the JONSWAP spectrum should be used for fetch-limited situations and the Pierson-Moskowitz spectrum for the fully developed sea (see clause **22**).

27.3.2 *Maximum heights.* For a number of design conditions, all with the same average wave parameters and duration, the maximum wave height taken from each will differ because of the statistical nature of the sea. However, the average resulting wave height, known as the average maximum, will just be a function of the average wave parameters and duration of the design conditions. In fact, the average maximum is close to being the most probable value of the maximum wave height, and may be estimated from the following expression:

$$\overline{H}_{max} = \frac{H_s}{\sqrt{2}} \left\{ (\log_e N)^{1/2} + 0.2886\, (\log_e N)^{-1/2} \right\}$$

where

$\overline{H}_{max}$ is the average maximum wave height;

H_s is the significant wave height;

N is the number of zero-crossing waves in duration of design condition.

Alternatively, an estimate can be given for the maximum wave height with a given probability of being equalled or exceeded during the design condition, using the following expression:

$$H_{max} = \frac{H_s}{\sqrt{2}} \left\{ \log_e N - \log_e \log_e \left(\frac{1}{1-p} \right) \right\}^{1/2}$$

where

H_{max} is the maximum wave height;

p is the probability of $\overline{H}_{max}$ or H_{max} being equalled or exceeded during the design condition.

Values of $(H_{max}/H_s)^2$ for various probabilities and values of $(\overline{H}_{max}/H_s)^2$ are plotted on Figure 14.

In the case of inshore sites the above methods of obtaining maximum wave heights may be applied but within the limitation that the inshore significant wave height cannot exceed the water depth when $J \leqslant 5$, e.g. breakers plunging on a relatively steep seabed slope, and 0.8 times the water depth when $J > 5$, e.g. breakers spilling on a more gentle seabed slope, where J is the ratio of the offshore wave steepness to the square of the beach slope (see **23.4**).

27.4 *Extrapolation of wave periods.* Whether the wave information used for the extrapolation of wave height was obtained from forecasts from wind using Figure 6 or Figure 9, from visual observations of wave heights or from wave records, the data should contain information on wave period. Depending on the source of data this period could be the zero-crossing period, T_z, the significant period, T_s, or the period at which the peak occurs in the wave spectrum, T_p. Due to the shape of wave spectra the following inequalities apply:

$$T_z < T_s < T_p$$

It is usual to plot a scatter diagram of wave height against wave period on which curves of constant wave steepness may also be plotted. In general, this diagram will suggest a prevalent value of wave steepness which may be assigned to the design condition. However, if there is considerable scatter in the values of wave steepness it may be necessary to consider a range of values to be associated with the design wave height.

For a given design wave height, the lower values of wave steepness, i.e. longer wave periods, will generally give the greater wave force, but care is needed if the structure or its individual members can resonate at periods within the period range of incident waves, in which case the resonant period should be included in the design parameters (see section 5).

For storm waves, the wave steepness, $2\pi H_s/(g T_z^2)$, in terms of significant wave height and the deep water wavelength associated with the zero-crossing wave period, is typically within the range 0.04 to 0.06. For a fully developed sea state it is 0.05 irrespective of the significant wave height.

In the case of maximum wave heights, the periods associated with these maximum values can be expected to be close to the period, T_p, at which the peak occurs in the one-dimensional wave frequency spectrum and this period is generally significantly longer than the zero-crossing wave period, T_z. For a fully developed sea state $T_p/T_z \approx 1.4$.

Maximum significant wave heights for 39 storms in 1 year

Storm No	H_S	Storm No	H_S
	m		m
1	1.05	20	1.14
2	1.80	21	1.09
3	1.11	22	1.77
4	2.31	23	1.39
5	3.16	24	1.60
6	1.64	25	1.27
7	1.70	26	3.16
8	1.49	27	2.31
9	2.25	28	1.74
10	1.12	29	1.09
11	1.39	30	1.61
12	1.43	31	1.21
13	0.75	32	0.96
14	1.12	33	1.38
15	1.52	34	2.31
16	0.79	35	3.21
17	1.26	36	1.93
18	1.02	37	2.34
19	1.40	38	4.57
		39	1.23

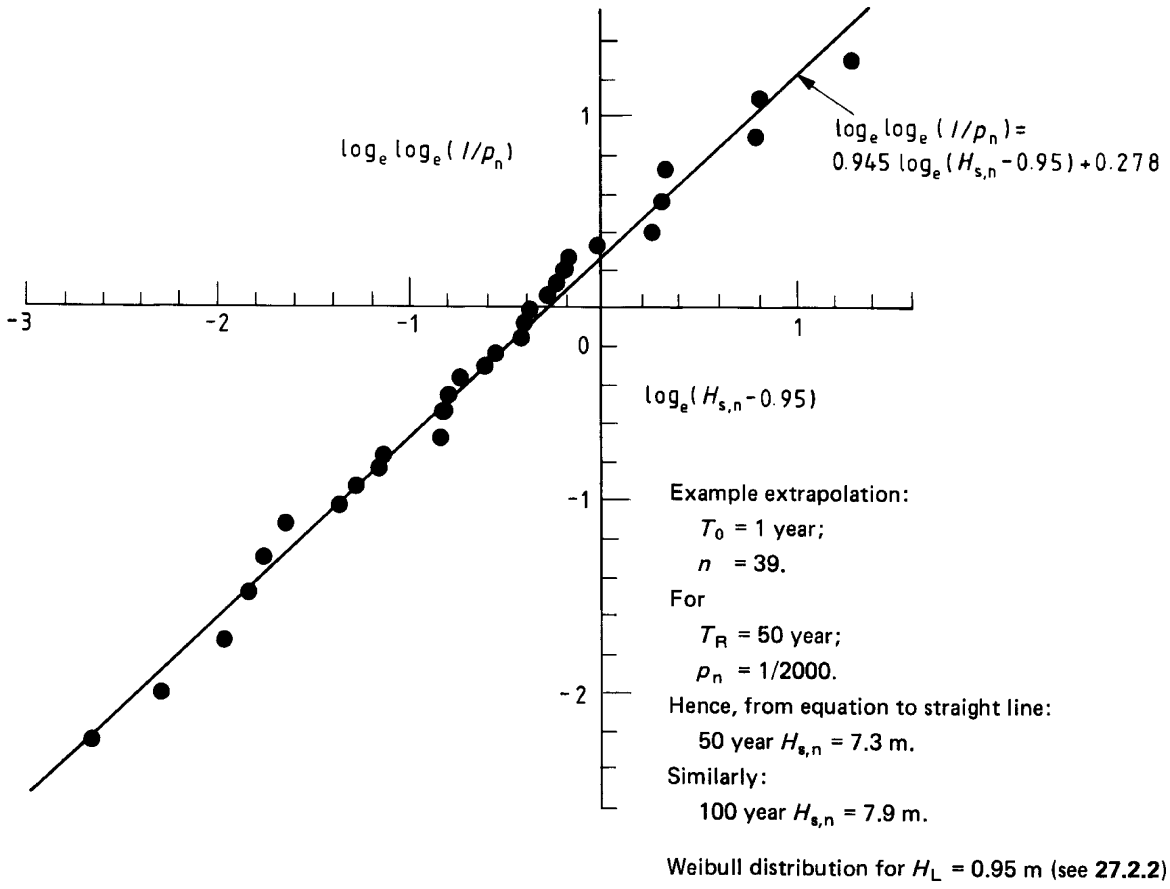

Figure 13 — Typical wave height extrapolation plot

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28 Effects of breakwaters and sea-walls on sea states

28.1 General. Sea-walls are used to protect the land from damage by the sea and they become necessary where there is little or no protective beach. Breakwaters provide protection against wave action for harbours.

Both types of protective structure can have vertical or almost vertical faces and are then highly reflective when subject to non-breaking waves. For harbour breakwaters, this creates cross seas which can lead to unfavourable conditions for ships entering or leaving harbour. In addition such structures, especially when subject to broken waves, can create bed currents and turbulence which leads first to severe scouring of material from the base of the structure so that some form of armouring is necessary and second to a general fall in the bed levels in the vicinity of the structure due to the increased littoral drift (see clause **14**). In contrast, a rubble-mound type of construction, which can be used for both sea-walls and breakwaters, is effective in dissipating energy and this helps to reduce cross seas at harbour entrances and scour in front of the structure.

In some cases breakwaters consist of vertical faced stuctures resting upon wide-topped rubble mounds. These act as effective absorbers for large waves which break before reaching the structure but are highly reflective for low waves. Similar behaviour is exhibited by vertical sea-walls preceded by a beach.

An important practical consideration in the design of breakwaters and sea-walls is the crest level needed to limit overtopping. Overtopping itself reduces wave reflection and the amount of overtopping obviously depends on the maximum ambient water level and the maximum vertical distance, known as run-up, which the waves can reach above this level. Storm surges can raise the water level to values above high tide level and the factors which can affect these levels are discussed in clause **25**. The maximum elevation to which the waves can reach depends not only on the height of the largest waves but also the rise in mean level accompanying groups of high waves impinging on the structure (see **29.2.4**).

28.2 Breakwaters in deep water. In deep water the seas incident on a breakwater may be expected to have the JONSWAP or Pierson–Moskowitz wave energy spectrum (see clause **22**), modified by the effects of shoaling and refraction as described in **23.2**. While some measurements of overtopping have been carried out in irregular seas of this type [24] the majority of tests have been made in models using regular waves of constant height and period. Results of these tests are summarized in Figure 15 and Figure 16, which refer to breakwaters with graded riprap slope protection and the more porous rubble-mound structures, respectively. These results do not apply if waves break before reaching the structure.

In Figure 15 the relative run-up or ratio of run-up to wave height is given as a function of the ratio of the slope steepness to the square root of the wave steepness, where α_s is the angle the structure slope makes with the horizontal, H is the wave height in front of the structure and L_0 is the deep water wavelength. In contrast, the results given in Figure 16 are expressed separately for a number of structure slopes in terms of the deep-water wave height H_0 and the wave period T.

Although these results apply to regular waves they may be generalized to irregular seas by the following argument. Since wave spectra are narrow (see Figure 7 and Figure 8) it is reasonable to assume that wave heights in an irregular sea will have a Rayleigh probability distribution from which it follows that approximately 13 % of the wave heights will exceed the significant wave height. Therefore, if the crest level of a rubble-mound structure is designed such that a wave height equal to the significant wave height is just prevented from overtopping then only approximately 13 % of the waves will overtop the structure. This is not considered to be excessive for a breakwater because it is thought to lead neither to a requirement for armouring to be heavier on the lee side than on the seaward side, nor to significant wave disturbance on the lee side. It is thus frequently possible to use the significant wave height as the design wave height for establishing a crest level for a rubble-mound breakwater.

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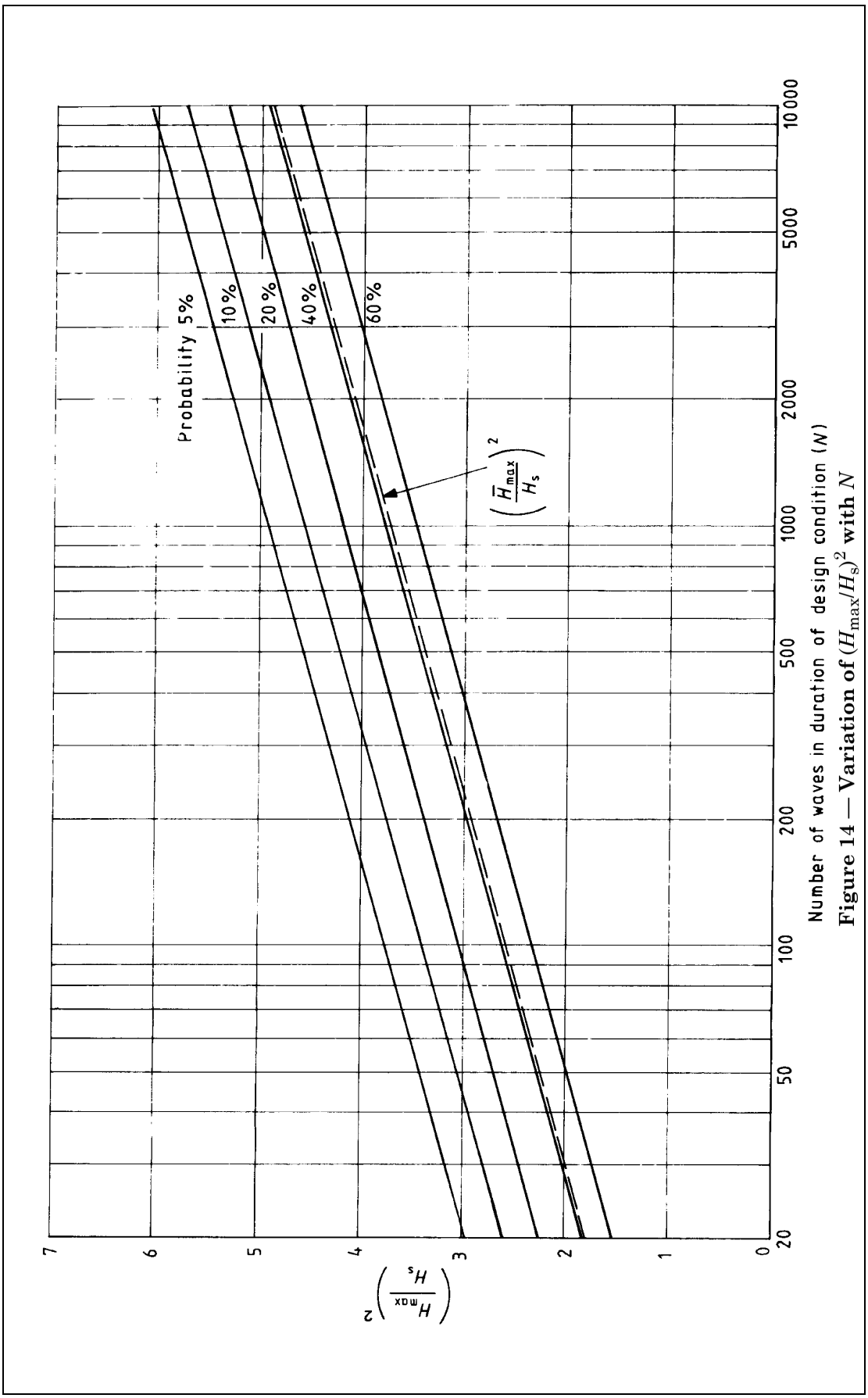

Figure 14 — Variation of $(H_{max}/H_s)^2$ with N

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28.3 Breakwaters and sea-walls in shallow water. When breakwaters and sea-walls are situated in shallow water the higher waves are forced to break and the wave spectrum approaching the structure is considerably modified. With relatively low offshore wave heights, only the largest waves break before reaching the structure and the majority either break or are reflected at the structure. In more severe conditions breaking occurs over an area seaward whose extent depends on the ratio of the local significant wave height to the depth and, to a lesser degree, on the mean wave period and bed slope.

As the waves travel inshore in diminishing depth the ratio of significant wave height to depth lies between about 0.8 and 1.0 (see **23.4**) and the significant wave height is almost entirely dependent on the local depth and slope, the tendency for wave heights to increase by shoaling being balanced by wave breaking, either by spilling or plunging, and bed friction.

When such breaking seas impact on breakwaters or sea-walls, part of the incident wave energy is absorbed by waves plunging or collapsing on the structure itself and part is reflected depending on the slope of the face of the structure and its porosity. Vertical structures are highly reflective for the lower waves within the sea state and these can cause breaking of incident waves below the depth limit. Plunging waves striking such obstructions lose energy by the expulsion of near vertical jets from the crests at the moment of impact, by turbulence in the flow at the crest and bed and by reflection. Breakwaters with flatter slopes can cause less reflection since incident spilling waves can also lose energy by running up the slope and plunging. Thus the geometry of the structure and its porosity can modify the heights of waves incident upon it. Because of the dependence of the incident wave energy on the depth, it is usual to test structures situated in shallow water by using models subject to irregular waves. Sufficient of the bed topography should be represented to ensure that the approaching waves break in the correct manner.

Since the height of the largest waves directly incident on an inshore structure is mainly dependent on the depth at the toe, run-up and overtopping will be greatest at high water and therefore the occurrence of extreme values may be more dependent on the incidence of tidal surges than on exceptionally severe storms (see clause **25**).

For a sea-wall, particularly if buildings or roadways are within a short distance, the acceptable overtopping is likely to be very much less than for a harbour breakwater. Close to the wall much of the overtopping water falls as a contiguous mass and its throw is comparatively independent of wind speed, whereas further away it falls as a cloud of spray which is more readily carried by the wind. Comparison of rates of overtopping measured in model and full scale have been published [24, 25], as have subjective assessments of the danger of different rates of overtopping to buildings and personnel located behind the wall [24].

29 Harbour response

29.1 General. A requirement in the design of harbours is the ability to estimate the degree of shelter which will result from any given layout of breakwaters. The wave pattern within a harbour with two breakwater arms on each side of the entrance is made up of crests forming arcs inside the harbour which decrease in height as they spread out, as if a source of wave energy existed at the entrance. Calculation of this type of effect, known as wave diffraction, is necessary if predictions of wave heights inside the harbour are to be made.

The response of harbours can be further complicated by the effects of wave refraction over a varying seabed (see **23.2**) and wave reflection from the interior boundaries of the harbour. If the reflection coefficient, i.e. the ratio of the reflected wave height to the incident wave height, is high these reflected waves could then undergo further reflection and for certain wavelengths these multiple reflections can reinforce one another giving rise to an amplification of the incident wave height. This effect, known as harbour resonance or seiching, can cause ships to range at their berths thereby developing high mooring loads and leading to lines and fenders being broken in severe cases. It is, therefore, good practice to build breakwaters, sea-walls, quay faces and reclamation areas with rubble slopes wherever possible as these can be useful dissipators of wave energy (see clause **28**). Natural beaches are very good dissipators of storm and swell wave energy so that a careful evaluation should be made before making any harbour modification which might decrease or eliminate such areas.

In **29.2** some simple diffraction solutions are given which ignore the effects of wave refractions and reflection. These solutions can, in general, only be used to obtain a first estimate of storm or swell wave heights inside large harbours in which the boundaries are several wavelengths distant from the harbour entrance and therefore unlikely to be good reflectors. Under these conditions there is insufficient reflected wave energy present in the harbour to alter seriously the wave diffraction pattern obtained ignoring reflection. This situation is not valid for long waves with periods of the order of minutes where even a beach can be expected to produce a noticeable reflection, nor for smaller harbours for fishing boats and pleasure craft which frequently have reflective boundaries, such as vertical faced quay walls, and which consequently may resonate at storm and swell wave periods.

The diffraction solutions described in **29.2** are modified in **29.3** to include the effect of wave refraction and, although these solutions can be further extended to include some wave reflection, the type of model described is not capable of allowing for harbour resonance and is therefore similarly limited to the calculation of storm and swell wave heights inside large harbours, but to a greater degree of accuracy.

In **29.4** some of the basic resonant modes of harbour oscillation are described and in **29.5** and **29.6** guidance is given on the use of models in harbour analysis.

29.2 Wave diffraction for a flat seabed

29.2.1 *General.* The solutions presented in **29.2.2** to **29.2.4** are applicable to regions sheltered by breakwaters within which negligible refraction and reflection takes place.

29.2.2 *Single breakwater case.* The solution for waves incident on a single breakwater arm is similar to the solution found for the diffraction of light around a semi-infinite screen. Figure 17(a) shows a typical diffraction diagram in which wave fronts and contours of the diffraction coefficient, K_d, the ratio of the diffracted wave height over the incident wave height, on the sheltered side of a single, vertical faced, impervious breakwater arm are plotted for the case of uniform waves approaching with their crests parallel to the breakwater. In diffraction diagrams of this type distance is normally expressed in units of wavelength so that it is necessary to work out (from Figure 11) the value of wavelength appropriate to the water depth and wave period.

Diffraction coefficients for a range of uniform wave directions incident on a single breakwater arm are shown in Table 1 in which the coefficient is given for various positions defined by polar coordinates and for various incident wave directions where angles and distances are as defined in Figure 17(b).

Diagrams of this type generally show that, along the "shadow" line drawn on the sheltered side from the breakwater tip parallel to the incident wave direction, the diffracted wave height is approximately half the incident wave height and, on each side of this line, the diffracted wave height is changing rapidly. The relatively rapid variation of wave height in this region is unlikely to occur to such an extent in real situations due to wave energy being spread in direction.

Although wave refraction outside the harbour will tend to reduce the directional spread of incident wave energy, the remaining spread will, in many situations, be sufficient to reduce the rate of variation in the theoretical diffraction pattern obtained for uniform waves given in Table 1.

29.2.3 *Double breakwater case.* Where a harbour entrance is formed by two breakwater arms, it is acceptable to apply the solution given above for a single breakwater arm to each breakwater in turn provided the width is 5 wavelengths or greater.

For entrance widths of less than 5 wavelengths a more exact solution, which makes use of a series of Mathieu functions, can be given. Results for seven angles of wave approach to entrance widths of 0.5, 1, 2 and 3 wavelengths are summarized in Figure 18 where an intensity factor $l_D(\theta)$ together with its relevant scale is given for each case. The wave diffraction coefficient, K_d, may be obtained at a position with polar coordinates (r,θ) by the following procedure (see Figure 19).

On the intensity factor plot appropriate to the entrance width and incident wave direction, a line is drawn at the required angle θ extending from the centre of the entrance. The distance along this line to the point where it cuts the intensity plot gives the value of the intensity factor $l_D(\theta)$ when measured at the relevant scale. The diffraction coefficient K_d at the position (r,θ) is then given by:

$$K_d = \sqrt{(l_D(\theta)L/r)}$$

If K_d is written in the form of the wave height $H_{r,\theta}$ over the incident height, H_{inc}, then the above relationship can be rewritten to give:

$$H_{r,\theta} = \sqrt{(H_{inc}^2\, l_D(\theta)L/r)}$$

from which it can be seen that the diffracted wave height for a given value of θ decreases with radius r just as if a source of wave energy $(H_{inc}^2\, l_D(\theta)\, L)$ existed at the harbour entrance.

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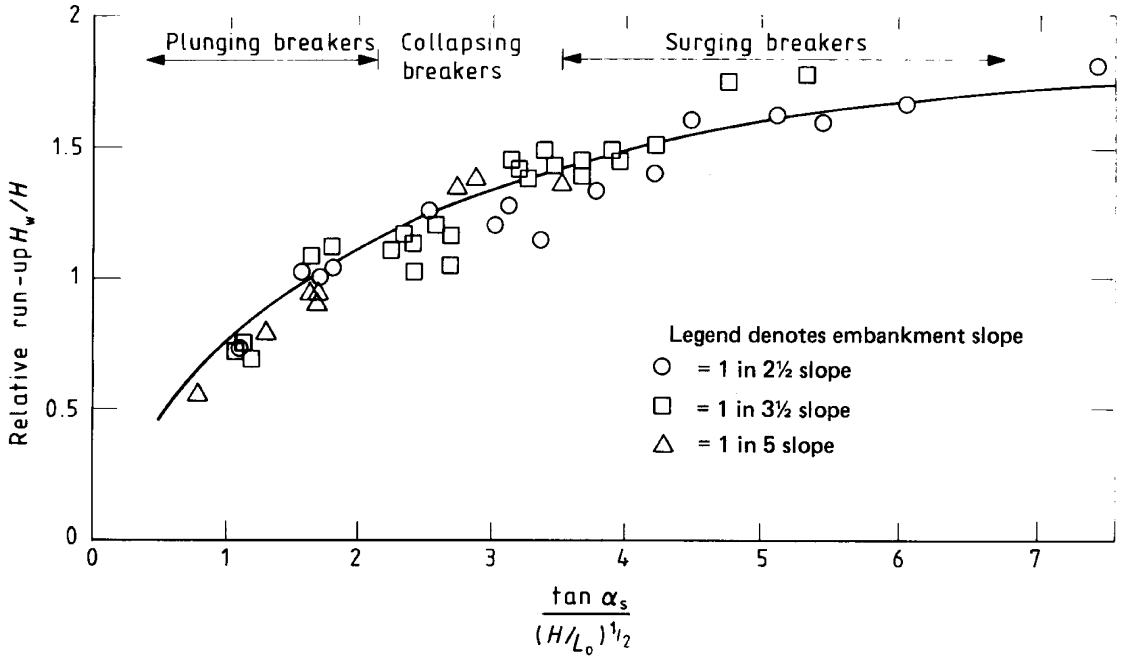


Figure 15 — Run-up on graded riprap slope

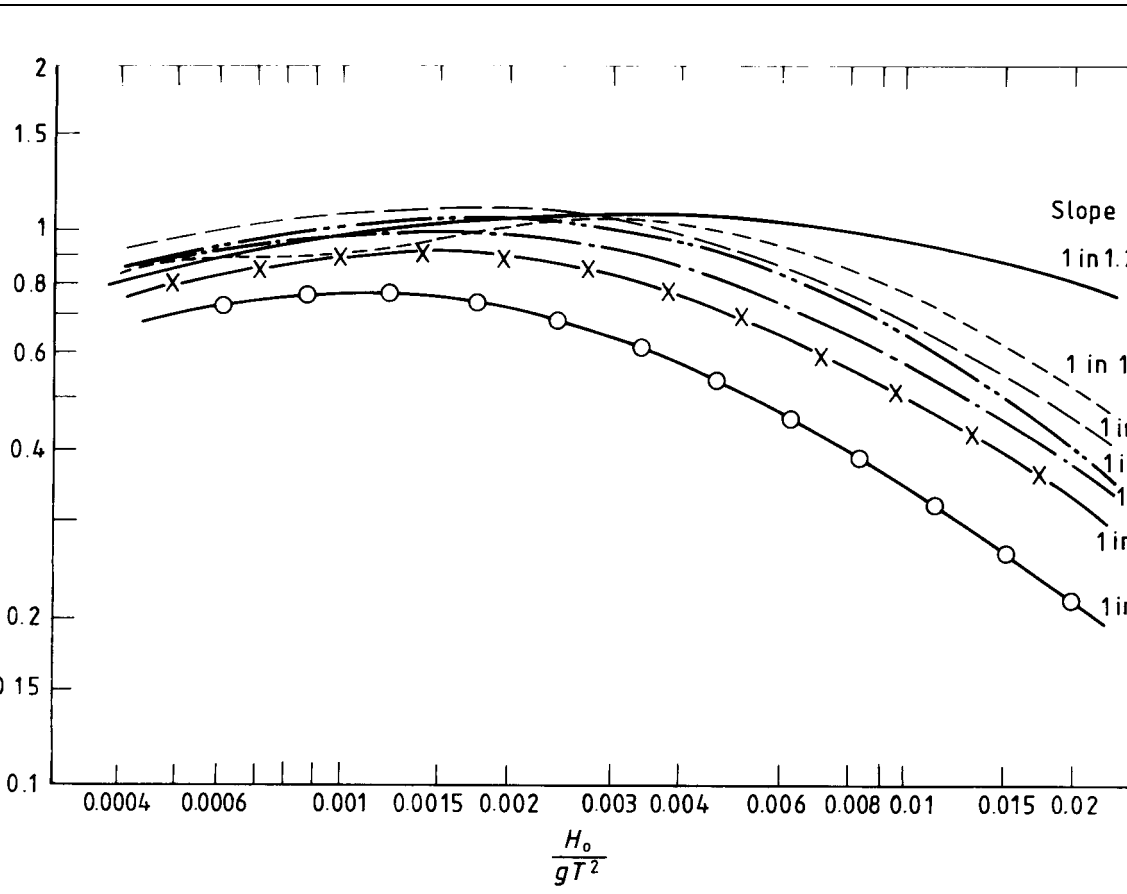
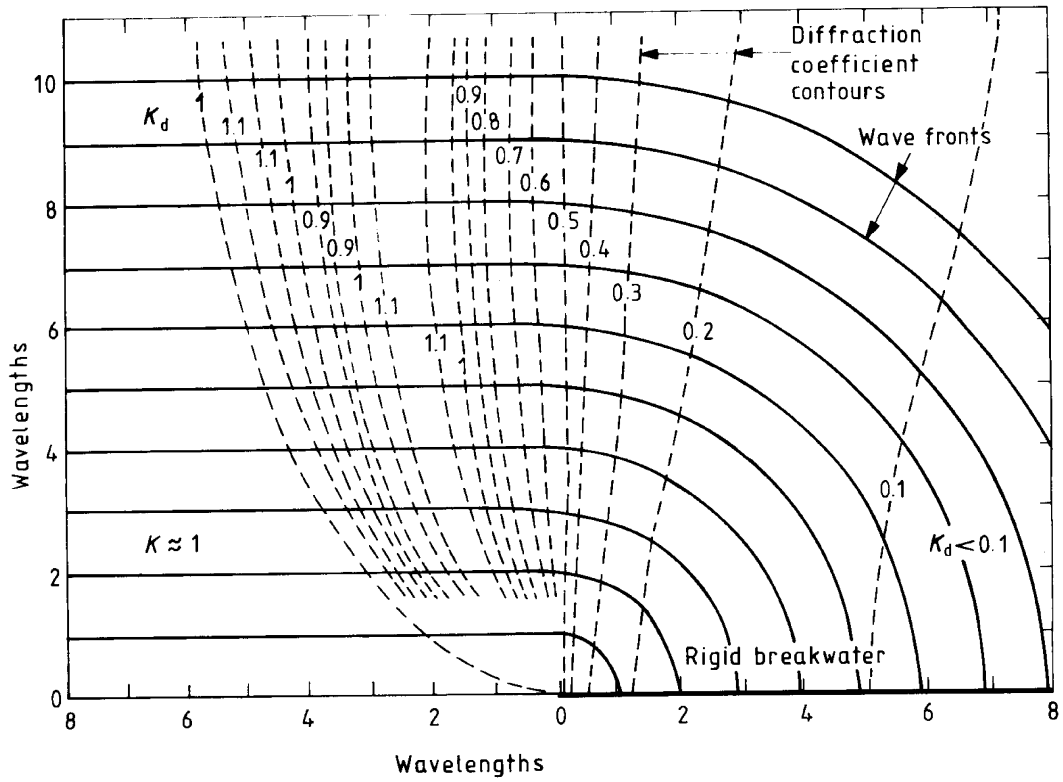
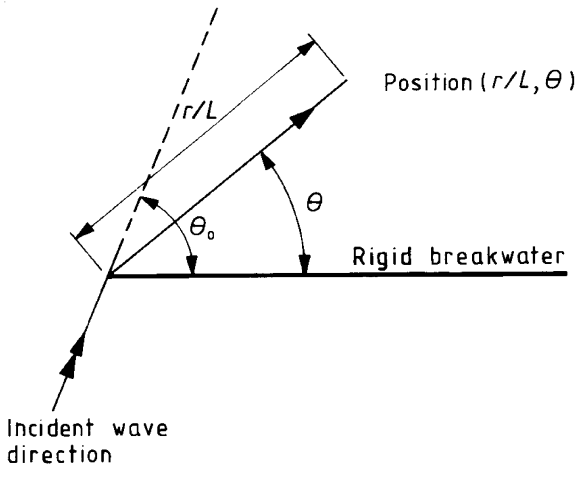


Figure 16 — Run-up on rubble-mound slope



(a) Diffraction diagram for $\theta_0 = 90°$



(b) Definition of terms for figure 17(a) and table 1

Figure 17 — Wave diffraction at single breakwater arm

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From Figure 18 it can also be seen that the energy source varies from one that will give a fairly uniform diffracted wave height at all angles θ for an opening of half a wavelength, to one that will give a diffracted wave height which is strongly dependent on θ for an opening of more than one wavelength. One result of this effect is that the building of a second breakwater arm to reduce the entrance width can increase wave heights at some positions inside the harbour. It is clear from Figure 18, however, that the diffracted wave height at positions immediately opposite the entrance ($\theta = 90°$) will be diminished when the width of opening is reduced.

29.2.4 *Currents induced by wave diffraction.* A secondary effect of wave diffraction at harbour entrances is the induction of currents in the lee of the breakwaters.

From the contours of the diffraction coefficient shown in Figure 17(a) it can be seen that a gradient in wave height exists along a wave crest on the sheltered side of a breakwater. At a harbour boundary along which this wave crest breaks the orbital movement of water particles is converted into an up-rush of water by the breaking process. This causes a local rise in the mean water level which is maintained by successive waves breaking. This mechanism, known as wave set-up, will increase as the wave height increases so that the diffraction pattern in Figure 17(a) will lead to a set-up which decreases along the harbour boundary in the direction of the shelter of the breakwater and which will generate a flow of water towards the sheltered side and induce a return current travelling from the tip of the breakwater towards the boundary opposite the entrance. Thus, large eddies can be formed on the sheltered sides of breakwaters; these currents can be expected to be of the order of 0.5 m/s to 1 m/s (1 knot to 2 knots) when the incident wave height is large (see **14.3.1**).

29.3 Ray methods of wave diffraction and refraction. In many cases the water depth varies within a harbour so that the diffraction patterns given in **29.2** for a flat seabed will be modified by wave reflection within the harbour.

One approximate method of combining the two effects is to use the diffraction solution for a flat bed within the harbour over a distance of 3 to 4 wavelengths from the entrance and then to send out rays that obey Snell's law (see **23.2.1**) over the remainder of the harbour area. Clearly, the wave height and direction at the starting point of each ray should be chosen to be consistent with the results of the diffraction solution.

The above types of approximation is inadequate where significant depth changes occur inside the harbour within 3 to 4 wavelengths of the entrance. In these situations it may be better to start a regular fan of rays from the centre of the harbour entrance thereby describing refraction over the entire harbour area. The diffraction effect may then be included by assuming that the energy flux of the waves in the fan at the entrance is the same as that which would be obtained after diffraction for a flat seabed. By combining the expressions developed for that case (see **29.2.2**) with those developed for refraction and shoaling (see **23.2.1**), the following expression may be derived for wave height at the arrival points of the rays inside the harbour:

$$H_a = H_{inc} \sqrt{(l_D(\theta)\, L v_{cg}\, \delta\theta / v_{cga}\, b_a)}$$

where

H_a is the wave height on arrival of the ray including the effects of refraction, shoaling and diffraction;

H_{inc} is the wave height incident on the harbour;

$l_D(\theta)$ is the intensity factor of the type shown in Figure 18 appropriate to the incident wave direction and harbour entrance width;

θ is the starting angle of the ray at the harbour entrance;

L is the wavelength for the depth of water at the harbour entrance;

v_{cg} is the group velocity for the depth of water at the harbour entrance;

$\delta\theta$ is the angular ray separation at the harbour entrance;

v_{cga} is the group velocity on arrival;

b_a is the ray separation on arrival.

Similar approximations can be made for the case of a single breakwater arm.

Solutions of the types discussed above are approximate and so will produce only rough estimates of the combined effects of refraction and diffraction.

29.4 Harbour resonance. As described in **29.1**, harbour resonance can occur when reflections of certain wavelengths reinforce and amplify the incident wave pattern.

Table 1 — Diffraction coefficients for single breakwater arm

NOTE Refer to Figure 17(b) for definition of terms.

Values of r/L	Value of θ (deg)												
	0	15	30	45	60	75	90	105	120	135	150	165	180
	Diffraction coefficient, K_d												
$\theta_0 = 15°$													
0.5	0.49	0.79	0.83	0.90	0.97	1.01	1.03	1.02	1.01	0.99	0.99	1.00	1.00
1	0.38	0.73	0.83	0.95	1.04	1.04	0.99	0.98	1.01	1.01	1.00	1.00	1.00
2	0.21	0.68	0.86	1.05	1.03	0.97	1.02	0.99	1.00	1.00	1.00	1.00	1.00
5	0.13	0.63	0.99	1.04	1.03	1.02	0.99	0.99	1.00	1.01	1.00	1.00	1.00
10	0.35	0.58	1.10	1.05	0.98	0.99	1.01	1.00	1.00	1.00	1.00	1.00	1.00
$\theta_0 = 30°$													
0.5	0.61	0.63	0.68	0.76	0.87	0.97	1.03	1.05	1.03	1.01	0.99	0.95	1.00
1	0.50	0.53	0.63	0.78	0.95	1.06	1.05	0.98	0.98	1.01	1.01	0.97	1.00
2	0.40	0.44	0.59	0.84	1.07	1.03	0.96	1.02	0.98	1.01	0.99	0.95	1.00
5	0.27	0.32	0.55	1.00	1.04	1.04	1.02	0.99	0.99	1.00	1.01	0.97	1.00
10	0.20	0.24	0.54	1.12	1.06	0.97	0.99	1.01	1.00	1.00	1.00	0.98	1.00
$\theta_0 = 45°$													
0.5	0.49	0.50	0.55	0.63	0.73	0.85	0.96	1.04	1.06	1.04	1.00	0.99	1.00
1	0.38	0.40	0.47	0.59	0.76	0.95	1.07	1.06	0.98	0.97	1.01	1.01	1.00
2	0.29	0.31	0.39	0.56	0.83	1.08	1.04	0.96	1.03	0.98	1.01	1.00	1.00
5	0.18	0.20	0.29	0.54	1.01	1.04	1.05	1.03	1.00	0.99	1.01	1.00	1.00
10	0.13	0.15	0.22	0.53	1.13	1.07	0.96	0.98	1.02	0.99	1.00	1.00	1.00
$\theta_0 = 60°$													
0.5	0.40	0.41	0.45	0.52	0.60	0.72	0.85	1.13	1.04	1.06	1.03	1.01	1.00
1	0.31	0.32	0.36	0.44	0.57	0.75	0.96	1.08	1.06	0.98	0.98	1.01	1.00
2	0.22	0.23	0.28	0.37	0.55	0.83	1.08	1.04	0.96	1.03	0.98	1.01	1.00
5	0.14	0.15	0.18	0.28	0.53	1.01	1.04	1.05	1.03	0.99	0.99	1.00	1.00
10	0.10	0.11	0.13	0.21	0.52	1.14	1.07	0.96	0.98	1.01	1.00	1.00	1.00
$\theta_0 = 75°$													
0.5	0.34	0.35	0.38	0.42	0.50	0.59	0.71	0.85	0.97	1.04	1.05	1.02	1.00
1	0.25	0.26	0.29	0.34	0.43	0.56	0.75	0.95	1.02	1.06	0.98	0.98	1.00
2	0.18	0.19	0.22	0.26	0.36	0.54	0.83	1.09	1.04	0.96	1.03	0.99	1.00
5	0.12	0.12	0.13	0.17	0.27	0.52	1.01	1.04	1.05	1.03	0.99	0.99	1.00
10	0.08	0.08	0.10	0.13	0.20	0.52	0.14	1.07	0.96	0.98	1.01	1.00	1.00
$\theta_0 = 90°$													
0.05	0.31	0.31	0.33	0.36	0.41	0.49	0.59	0.71	0.85	0.96	1.03	1.03	1.00
1	0.22	0.23	0.24	0.28	0.33	0.42	0.56	0.75	0.96	1.07	1.05	0.99	1.00
2	0.16	0.16	0.18	0.20	0.26	0.35	0.54	0.69	1.08	1.04	0.96	1.02	1.00
5	0.10	0.10	0.11	0.13	0.16	0.27	0.53	1.01	1.04	1.05	1.02	0.99	1.00
10	0.07	0.07	0.08	0.09	0.13	0.20	0.52	1.14	1.07	0.96	0.99	1.01	1.00
$\theta_0 = 105°$													
0.5	0.28	0.28	0.29	0.32	0.35	0.41	0.49	0.59	0.72	0.85	0.97	1.01	1.00
1	0.20	0.20	0.24	0.23	0.27	0.33	0.42	0.56	0.75	0.95	1.06	1.04	1.00
2	0.14	0.14	0.13	0.17	0.20	0.25	0.35	0.54	0.83	1.08	1.03	0.97	1.00
5	0.09	0.09	0.10	0.11	0.13	0.17	0.27	0.52	1.02	1.04	1.04	1.02	1.00
10	0.07	0.06	0.08	0.08	0.09	0.12	0.20	0.52	1.14	1.07	0.97	0.99	1.00
$\theta_0 = 120°$													
0.5	0.25	0.26	0.27	0.28	0.31	0.35	0.41	0.50	0.60	0.73	0.87	0.97	1.00
1	0.18	0.19	0.19	0.21	0.23	0.27	0.33	0.43	0.57	0.76	0.95	1.04	1.00
2	0.13	0.13	0.14	0.14	0.17	0.20	0.26	0.36	0.55	0.83	1.07	1.03	1.00
5	0.08	0.08	0.08	0.09	0.11	0.13	0.16	0.27	0.53	1.01	1.04	1.03	1.00
10	0.06	0.06	0.06	0.07	0.07	0.09	0.13	0.20	0.52	1.13	1.06	0.98	1.00
$\theta_0 = 135°$													
0.5	0.24	0.24	0.25	0.26	0.28	0.32	0.36	0.42	0.52	0.63	0.76	0.90	1.00
1	0.18	0.17	0.18	0.19	0.21	0.23	0.28	0.34	0.44	0.59	0.78	0.95	1.00
2	0.12	0.12	0.13	0.14	0.14	0.17	0.20	0.26	0.37	0.56	0.84	1.05	1.00
5	0.08	0.07	0.08	0.08	0.09	0.11	0.13	0.17	0.28	0.54	1.00	1.04	1.00
10	0.03	0.06	0.06	0.06	0.07	0.08	0.09	0.13	0.21	0.53	1.12	1.05	1.00

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Table 1 — Diffraction coefficients for single breakwater arm

Values of r/L	Value of θ (deg)												
	0	15	30	45	60	75	90	105	120	135	150	165	180
	Diffraction coefficient, K_d												
$\theta_0 = 150°$													
0.5	0.23	0.23	0.24	0.25	0.27	0.29	0.33	0.38	0.45	0.55	0.68	0.83	1.00
1	0.16	0.17	0.17	0.18	0.19	0.22	0.24	0.29	0.36	0.47	0.63	0.83	1.00
2	0.12	0.12	0.12	0.13	0.14	0.15	0.18	0.22	0.28	0.39	0.59	0.86	1.00
5	0.07	0.07	0.08	0.08	0.08	0.10	0.11	0.13	0.18	0.29	0.55	0.99	1.00
10	0.05	0.05	0.05	0.06	0.06	0.07	0.08	0.10	0.13	0.22	0.54	0.10	1.00
$\theta_0 = 165°$													
0.05	0.23	0.23	0.23	0.24	0.26	0.28	0.31	0.35	0.41	0.50	0.63	0.79	1.00
1	0.16	0.16	0.17	0.17	0.19	0.20	0.23	0.26	0.32	0.40	0.53	0.73	1.00
2	0.11	0.11	0.12	0.12	0.13	0.14	0.16	0.19	0.23	0.31	0.44	0.68	1.00
5	0.07	0.07	0.07	0.07	0.08	0.09	0.10	0.12	0.15	0.20	0.32	0.63	1.00
10	0.05	0.05	0.05	0.06	0.06	0.06	0.07	0.08	0.11	0.11	0.21	0.58	1.00
$\theta_0 = 180°$													
0.05	0.20	0.25	0.23	0.24	0.25	0.28	0.31	0.34	0.40	0.49	0.61	0.78	1.00
1	0.10	0.17	0.16	0.18	0.18	0.23	0.22	0.25	0.31	0.38	0.50	0.70	1.00
2	0.02	0.09	0.12	0.12	0.13	0.18	0.16	0.18	0.22	0.29	0.40	0.60	1.00
5	0.02	0.06	0.07	0.07	0.07	0.08	0.10	0.12	0.14	0.18	0.27	0.46	1.00
10	0.01	0.05	0.05	0.04	0.06	0.07	0.07	0.08	0.10	0.13	0.20	0.36	1.00

For the longest mode of harbour resonance, sometimes called the pumping or Helmholtz mode, a vertical rise and fall of the water surface occurs over the harbour area and large horizontal oscillatory currents are formed in the harbour entrance. The next longest resonant mode is one where the water rises vertically along one boundary when it is falling vertically along an opposite boundary with a region in between where oscillatory horizontal flows occur. For obvious reasons this is sometimes called the sloshing mode. A range of increasingly shorter period resonances can occur but they are increasingly difficult to describe for typical harbours because of their complex shapes.

It should be noted that resonant wavelengths are fixed by the dimensions of the harbour but the resonant wave period will vary with the state of the tide.

For typical large ship harbours, the longest resonant wavelengths are of the order of kilometres with periods of the order of minutes. Possible sources of excitation are moving pressure fronts, set-down beneath wave groups, surf beats, edge waves and tsunamis (see clause **24**). Resonant modes at storm or swell wave periods are generally of less importance in large harbours but they can occur in smaller harbours such as those used by fishing boats and pleasure craft.

29.5 Physical models. At present, the most reliable method of evaluating harbour designs is by the use of physical models.

Apart from being able to determine harbour response in the presence of the combined effects of wave diffraction, refraction and reflection the physical model has the advantage over computational models in that, being a direct attempt to reproduce reality on a smaller scale, it therefore automatically describes various secondary physical effects which can only be added to the indirect computational model once they are identified and described theoretically. Examples of such effects are flow separation (see **29.6.2**), set-down beneath wave groups where irregular waves are generated (see **24.3**), and current systems induced by wave diffraction effects (see **29.2.4**).

In general, it is advisable that all model testing should be conducted, supervised and interpreted by experienced and qualified personnel and be carried out in laboratories having the appropriate specialized facilities.

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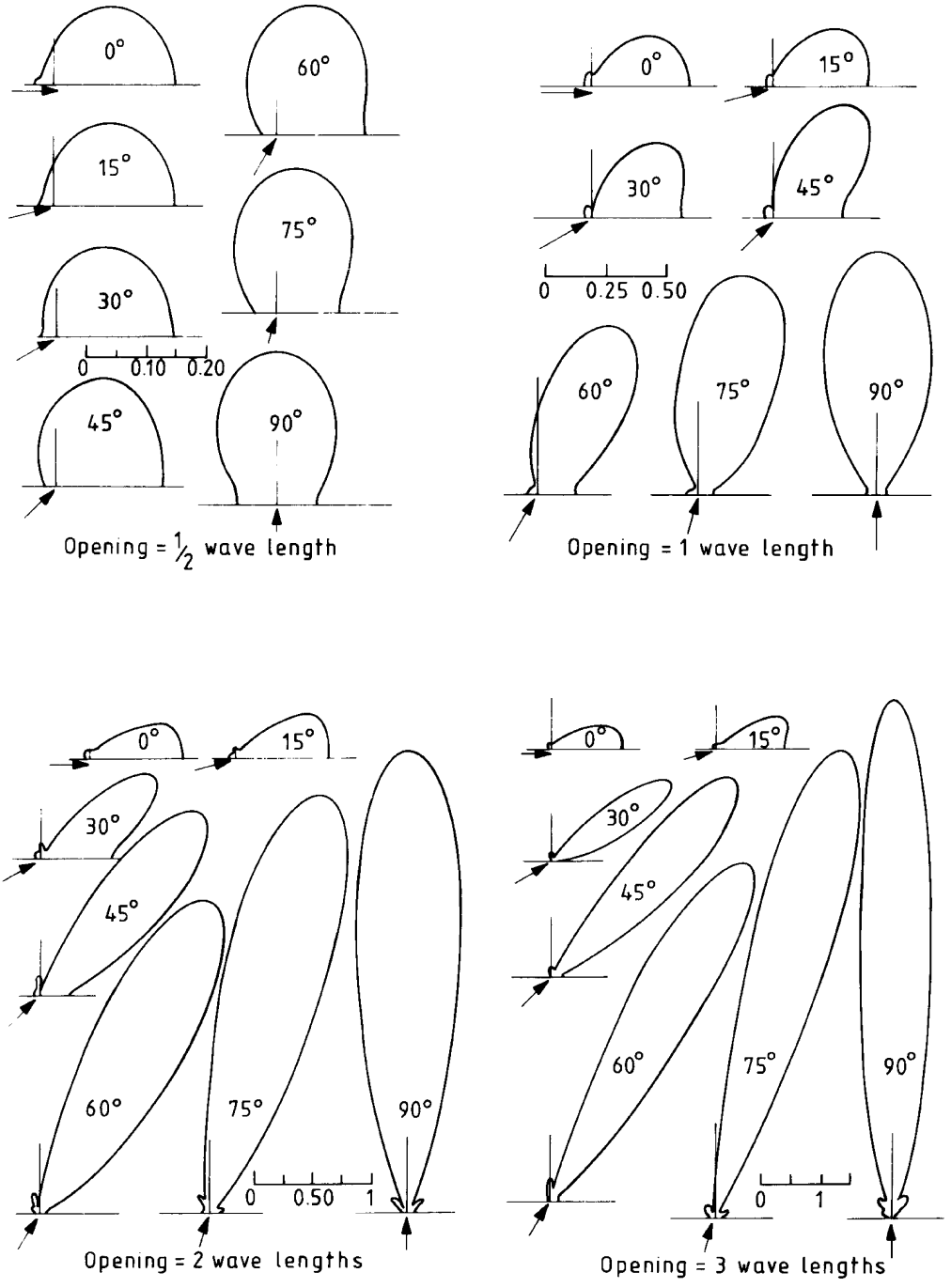

Figure 18 — Wave diffraction intensity factors for narrow breakwater openings

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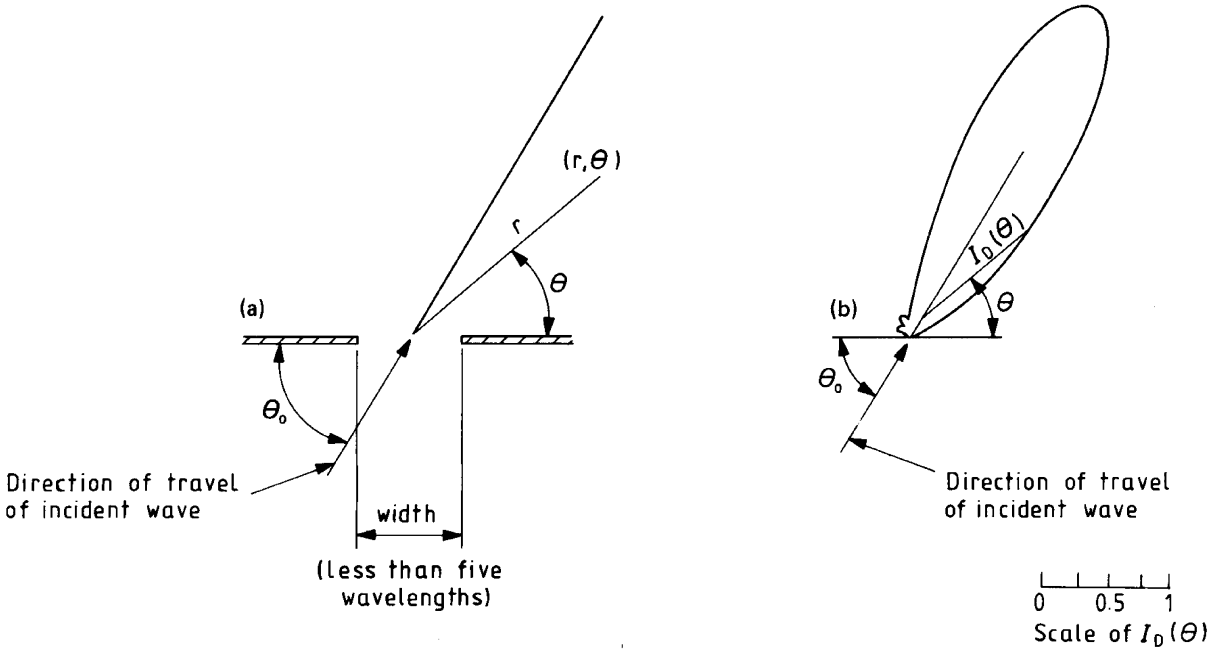

Figure 19 — Wave diffraction at narrow breakwater opening

When planning a project involving the study of various harbour layouts it is advisable to make an early decision in consultation with experts, on whether a physical model study is needed because the necessary information for such an investigation may require the installation of a wave recorder to obtain a year's wave data from the site (see clause **26**) and may also necessitate a bathymetric survey to determine the seabed topography (see clause **8**). Where necessary, a wave refraction study should be made before the physical model study, to convert deep water wave conditions to the wave condition expected at the position equivalent to the position of the model wave generator. When suitable site data are available these should always be used to prove the physical data. Typically, from the start of model construction to completion of the experimental test programme, model investigations last less than 6 months and generally cost a small percentage of the total cost of the project.

In these models the harbour area together with part of the surrounding offshore area and coastline is represented at an appropriate scale in a wave basin. The seabed is moulded, generally in the form of a skin of concrete on sandfill, to give an accurate representation of the seabed topography. The waves in these models are normally generated by an oscillating board or wave paddle and are therefore long crested. However, in many cases the wave generator can be made to produce a spread of wave energy over waves of various periods to give an irregular wave system with a one-dimensional wave frequency spectrum appropriate to the design condition. Usually these wave generators are mobile to allow evaluation of the proposals under a numer of different wave directions. Wave heights at various positions within the harbour can be measured, to the nearest 0.1 mm, with twin-wire resistive wave probes.

Since gravity in the model is the same as in the prototype it is necessary to use the Froude scaling law for the correct representation of wave motion. Therefore, if the length scale, both vertical and horizontal, is 1:N_s the following scale factors apply:

a) acceleration	1 : 1;
b) length	1 : N_s;
c) time	1 : $N_s^{½}$;
d) velocity	1 : $N_s^{½}$;
e) force, volume, mass	1 : N_s^3.

Models of harbours subject to storm or swell waves should always be undistorted, i.e. the vertical and horizontal length scales should be the same because it is impossible to generate distorted storm or swell waves. Long-wave models have been distorted, however, with the vertical scale larger than the horizontal, on the assumption that provided that the horizontal currents, which are the dominant feature of very long waves of a minute or longer period, are correctly modelled, the main effects of long-wave behaviour will be well represented. This allows a larger area to be represented in long-wave models.

Model scales for undistorted models normally lie within the range of 1 : 50 to 1 : 150 with smaller harbours being represented at the larger scale.

29.6 Computational models

29.6.1 *General.* The present generation of computational models can be expected to show up the qualitative effects of various proposals for the design of new harbours, or the modification of existing harbours, but they should not be relied upon to give accurate detailed information.

29.6.2 *Constant depth models.* Computer programs are now available which describe the combined effects of diffraction and reflection from the boundaries for harbours of constant depth and arbitrary shape. These programs assume that sources of wave energy exist along the harbour boundary and across the harbour entrance. By matching the solution inside the harbour to the wave conditions outside, a set of equations is obtained for the wave energy sources which enables the wave height to be calculated at any position inside the harbour.

Such programs can be expected to give accurate predictions of the resonant periods of oscillation of the harbour for a given constant water depth as well as the shape of the mode which the program represents, by indicating which areas will have vertical movement of the water surface and which areas will have horizontal water-particle movement.

The prediction of the amplification factors for the various resonant modes, however, is unreliable at present. In general, severe overestimates of the amplification factors result from these programs due to the fact that real harbours contain a number of dissipation mechanisms which are not usually represented in computer programs.

In both physical models and in real harbours it has been observed that the flow near sharp boundaries, with large oscillatory horizontal currents, tends to separate from the boundary to produce eddying motions. Such flow separation effects are thought to be a very important dissipation mechanism for the longest resonant wavelengths, such as those associated with the pumping or Helmholtz mode in which large oscillatory flows are produced at the harbour entrance. Another dissipation mechanism which could be significant is bottom friction (see **23.3**). Although it is possible to represent the effects of flow separation and bottom friction in computer programs by the use of linear friction terms it is not clear what values should be taken for the coefficients of friction, due to the lack of detailed knowledge of the dissipation mechanisms.

29.6.3 *Varying depth models.* Computer programs are also available which describe the combined effects of wave diffraction, refraction and reflection for those situations in which the seabed inside the harbour is not flat. In these programs a wave equation is solved using finite elements which cover the harbour area within which the water depth has to vary sufficiently slowly over distances equal to the deep water wavelength. The further assumption is generally made that the sea bed is flat outside the harbour.

Limitations in the use of these models are caused by the problems of realistically representing the effects of flow separation and bottom friction as described in **29.6.2** for constant depth harbour models.

30 Acceptable wave conditions for moored boats

30.1 General. Guidance is given in this clause on the maximum wave heights which can be tolerated for boats moored against jetties or quay walls inside marinas and fishing harbours and for barges moored alongside larger vessels. Although important, such criteria are rarely given in published literature and since the opinion of harbour masters can be expected to vary, the quoted figures should be taken only as general guidelines. In situations where careful attention is paid to moorings and boats are not moored in beam seas it may be possible to tolerate larger heights than those given below. Larger wave heights may also be tolerable if boats are moored off jetties with sufficient space between them to prevent collisions.

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The heights quoted should be interpreted as being the average maximum wave heights from which the maximum acceptable significant wave height may be obtained, if the duration of the design wave condition is known (see **27.3**). If the duration of the design wave condition is not known, then the significant height may be taken to be of the order of half the average maximum wave height.

As the response of pleasure craft and fishing boats to waves with periods of a minute or longer can be expected to be similar to the effect produced by currents, these long waves are not considered important for such boats, once they are moored. Thus, the criteria given in **30.2** to **30.4** apply to the residual height of waves inside the harbour at storm or swell wave periods.

30.2 Marinas. The maximum wave height considered as acceptable is 0.3 m. Typically, this makes the maximum acceptable significant wave height 0.15 m. One of the major limitations in marinas is that boats are often moored close to one another so that very little movement is possible before damaging collisions occur.

For marinas in exposed locations it will be necessary to build a system of overlapping breakwaters in order to achieve acceptable wave conditions. A useful guide in such situations is that the open sea should not be directly visible at water level from mooring positions inside the marina at any state of the tide.

30.3 Fishing harbours. Since fishing craft are normally larger and more strongly built than pleasure craft the maximum wave height considered as acceptable for boats up to 30 m in length is 0.8 m. Typically, this makes the maximum acceptable significant wave height 0.4 m. As in the case of pleasure craft, inner harbours or basins are frequently provided for accommodating fishing boats safely.

30.4 Lighterage. With a large open boat or barge moored alongside a larger vessel for the purpose of loading or unloading cargo, the main difficulty due to wave action is likely to be relative movement between the two vessels. Acceptable wave conditions for this type of cargo handling, which can take place inside larger harbours as well as just offshore, will depend on the nature of the cargo, the method of cargo handling and the sort of risk considered acceptable at the particular site. Generally, it is unlikely that the maximum acceptable wave height will exceed 2 m, making the maximum acceptable significant wave height about 1 m, provided that such wave conditions are also acceptable from the point of view of keeping the large vessel on its moorings (see clause **31**).

31 Acceptable wave conditions for moored ships

31.1 General. Guidance is given in this clause on the response to wave actions of larger vessels moored inside harbours or at offshore terminals.

Due to the complexity of vessel response it is extremely difficult to define acceptable conditions directly in terms of wave height. One reason for this is the non-linear relationship between horizontal oscillation of large vessels on their moorings and the storm or swell wave heights, since the natural periods of large vessels correspond more closely with those of long waves and wave groups than with storm or swell waves. A further difficulty is that mooring loads due to wind and current can be a significant proportion of the total mooring force for large vessels. Thus, to avoid mooring lines breaking, the maximum acceptable wave height has often to be reduced in the presence of wind and current.

Some background to the subject of moored ship motions is given in **31.2** and techniques available for determining acceptable sea states for moored ships are described in **31.3**. Acceptable ship movements are discussed in **31.4**.

31.2 Background information. When a floating body oscillates in water it creates a disturbance. In open water the inertia of the surrounding water accelerated by the motion of the body effectively increases the mass of that body and is known as the added mass. Oscillation also produces waves that propagate away from the body carrying energy with them which tends to damp out the oscillation.

Other damping mechanisms are skin friction, for streamlined bodies, and the creation of turbulent motions due to flow separation, for a blunt body.

The hydrodynamic coefficients of added mass and damping are different for each type of vessel movement, are dependent on the period of oscillation and, in shallow water, vary with underkeel clearance.

The forces tending to restore a moored vessel to its equilibrium position are buoyancy forces for vertical motions and forces supplied by the moorings for horizontal motions. These restoring forces give rise to natural periods of oscillation which, for vertical motions, can be within the range of swell and storm waves whereas, for horizontal motion, they vary from about 20 s for vessels of 3 000 t displacement to periods of a minute or longer for vessels in excess of 100 000 t.

The wave forces which act on a vessel to cause oscillation may be divided into two types. The first type are linear wave forces of the same period as the waves and may be obtained by integrating the fluctuating water pressure over the submerged area of the hull. Since the vessel will usually alter the wave pattern around itself the problem of diffraction of the wave system by the vessel has to be solved before the wave force can be determined. These forces are capable of exciting the natural periods of vertical oscillation of a vessel. With non-linear moorings they are also capable of exciting the natural periods of horizontal oscillation. The strongest non-linearity in moorings arises because the fenders are generally stiffer than the mooring lines. In a beam sea a vessel can move transversely on and off the fenders at a subharmonic of the wave period, i.e. the wave period divided by n, where n is an integer, and the largest motion occurs at the subharmonic which is nearest to the natural period of this motion. Although this type of vessel response can be avoided by making the fendering as soft as the mooring lines it does not necessarily follow that relatively soft fenders are better than relatively stiff ones, because the second type of wave forces described in this subclause could excite a larger resonant response of a vessel on softer moorings.

The second type of wave forces are non-linear and occur as a consequence of the irregular nature of the sea surface. Since waves travel in groups they produce secondary wave forces with the periodicity of wave groups. These secondary forces are smaller than the linear wave forces described in this subclause but they have periods similar to the natural periods of horizontal oscillation of moored ships. Since the natural damping of these oscillations is low, quite small secondary wave forces are capable of building up large resonant oscillations of a vessel on its moorings. For a vessel which is scattering the waves, a force is produced at the waterline due to scatter of the momentum carried by the waves. Since this momentum is larger in a group of high waves than in a group of small waves, the force produced will have the periodicity of wave groups. The fluctuating water pressures produced by set-down beneath wave groups (see **24.3**) will also act on the submerged part of the hull to produce a significant force in shallow water at wave group periods.

31.3 Methods for determining acceptable sea states

31.3.1 *General.* Until the advent of large ships, mooring techniques were based on practical experience. That this has proved successful is due partly to the fact that most vessels were able to moor in sheltered harbours or in relatively calm areas inshore. Past experience, therefore, can be used in determining the mooring arrangement for ships in sheltered waters. Large ships, however, require deeper water for their moorings and this frequently means that they are subjected to a greater degree of wave action since the deeper water of harbours is usually nearer to the harbour entrance. Relatively few harbours have sufficient water depth for ships with displacements in excess of 60 000 t and one method of mooring such vessels is to build an offshore mooring without the protection of breakwaters. This can take the form of a system of mooring dolphins accommodating both mooring lines and fenders.

Principally on account of the two reasons given in this subclause, the need has arisen for an adequate description of the response of a vessel moored in waves. This can be done with varying degrees of accuracy and reliability by physical models, analytical methods and computational models. At present the most reliable method of predicting a vessel's response under wave action is to build and test a physical model.

31.3.2 *Physical models.* These models use the Froude scaling laws (see **29.5**) and are similar to harbour response models. However, the model scale is rarely smaller than 1 : 100 in order to permit reasonable accuracy in the measurement of scaled mooring loads. In problems concerned with the mooring of vessels in relatively exposed harbours it is necessary to represent the harbour in the model to obtain the correct vessel response.

The load/deflection characteristics of fenders and mooring lines are usually represented in the model by a system of cantilever springs which can be made to reproduce any non-linear characteristic. Strain gauges are normally used to measure mooring loads. A variety of systems may be used to measure all six degrees of freedom of vessel movement (see Figure 20) but in the process of measurement they should not impose forces on the model vessel which could significantly affect its response.

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It is most important that an irregular sea be represented so that non-linear wave forces will be present in the model. A further requirement concerns adequate field data. At those sites where long waves are thought to be present it is important that wave recorders be installed because there is no guarantee that long wave motions with periods of a minute or more will be accurately generated in the model. In some models it may be necessary to add an appropriate long period movement to the wave generator to produce a wave climate in the model which agrees with that measured at the site. This may be explained by the interaction between set-down beneath wave groups and the coastline (see **24.3**). Such interactions, producing edge waves and surf beats, are likely to require an extensive part of the coastline to be represented in the model before they can be adequately reproduced and wave basin size will often preclude this.

When planning a project involving the mooring of ships it is advisable to make a decision, in consultation with experts, at an early stage on whether a model study is needed because a physical model study could require a year's wave data from the site to determine the long wave climate and then a further period, typically lasting 6 months or longer, to complete the model investigation.

As in the case of harbour response models, the advantage of a physical model is that many interactions are automatically described. For example, currents can be generated in these models which can have a considerable influence on the wave climate. Currents flowing against the waves tend to make the waves steepen and sometimes break, whereas currents flowing with the waves tend to reduce the wave height. This interaction can in turn influence the response of moored vessels to the waves and it is likely to be particularly important for offshore terminals. Winds can also be represented in physical models by the use of a bank of fans

The final conclusion reached after a model study frequently takes the form of an estimate of the number of days in the year when a given berth will be untenable with regard to the loading and unloading of cargo and, in severe cases, the number of days when ships cannot moor at the berth. The purpose of the model study is to minimize this length of time as far as possible by optimizing berth alignment, mooring line stiffness and, in the case of harbours, breakwater layout. In situations where the berth is subject to a combination of swell waves and long waves it is sometimes possible to choose the overall stiffness of the moorings so that the system allows movement at the swell wave period but is stiff enough to ensure that the resonant periods for surge, sway and yaw are shorter than those of the long waves. By this means a reduction of movement can be obtained without causing high mooring loads. One solution, in severe cases of wave action, which ensures a berth will be tenable throughout the year is to build an enclosed basin with a gate which can be opened to allow the ship to gain access and to leave. However, entering and leaving such a berth under wave action can be difficult due to wave currents. This effect will be amplified if a natural period of oscillation of the basin coincides with a wave period.

31.3.3 *Analytical methods.* Analytical methods of predicting a vessel's response to waves start at their simplest level with equations for vessel movement in one dimension in response to uniform waves. Solutions of this type can give information on certain aspects of the behaviour of a moored ship. In real situations, however, waves are not uniform and they usually excite all six degrees of freedom of vessel movement simultaneously (see Figure 20) so that a more general description is required. Due to the non-linearities described in **31.2**, theoretical prediction of mooring loads for general application is a complex task. It can be broken down into two main steps.

First, the response of the vessel to the primary storm or swell wave system has to be calculated. In this process the hydrodynamic coefficients of added mass and damping have to be obtained and the diffraction of the primary wave system by the vessel also has to be taken into account. Analytical methods now exist which allow reasonable estimates to be obtained of a vessel's response to the primary wave system in deep water. The approximations used, however, begin to break down for small underkeel clearances where lateral ship motions cause flow around the ends of the ship as well as under the keel, thereby making the problem truly three-dimensional.

The second step involves calculating the non-linear wave forces which will depend on both the primary wave system with diffraction effects and the vessel's response to that primary system. This second step has so far only been made for certain special cases.

Due to the complexity of the calculations it seems likely that computational methods offer the best hope of dealing with the general problem in a theoretical way.

31.3.4 *Computational models.* These models usually use a three-dimensional energy source technique where a series of sources are assumed to exist on the submerged surface of the ship. A set of equations for the unknown source strengths is then obtained by applying the boundary conditions to be satisfied on the surface of the vessel subject to a uniform incident wave. Solving for the source strengths then allows the hydrodynamic coefficients of added mass and damping as well as the wave exciting force to be obtained for that particular incident wave period and for all six degrees of freedom of the moored ship. This step can be repeated a number of times to obtain the hydrodynamic coefficients and wave force for a number of wave periods. Because this method is three-dimensional it can deal with small underkeel clearances and it can be extended to deal with the case of a ship moored against a vertical quay wall.

If the assumption is made that the mooring system is linear then the movements of a vessel moored in uniform waves may be obtained. Having calculated the response to a series of uniform waves with periods that cover a range of wave periods, the response of the vessel to an irregular sea can be represented by six spectra, one for each degree of freedom. The amount of energy in any one of the ship's spectra, say pitch, at a certain wave period is obtained by modifying the amount of wave energy present in the wave spectrum at that period, by the appropriate factor representing the pitch response of the vessel to a uniform wave of that period. This principle of superposition assumes that the vessel's response in the presence of a number of wave components can be obtained by adding together the responses to uniform waves taken one period at a time.

This procedure is, at best, only adequate for describing a vessel's response to the primary wave system when the moorings are linear. A second computation is required to obtain the non-linear wave forces and subsequent vessel response. This second step has only been carried out for special cases.

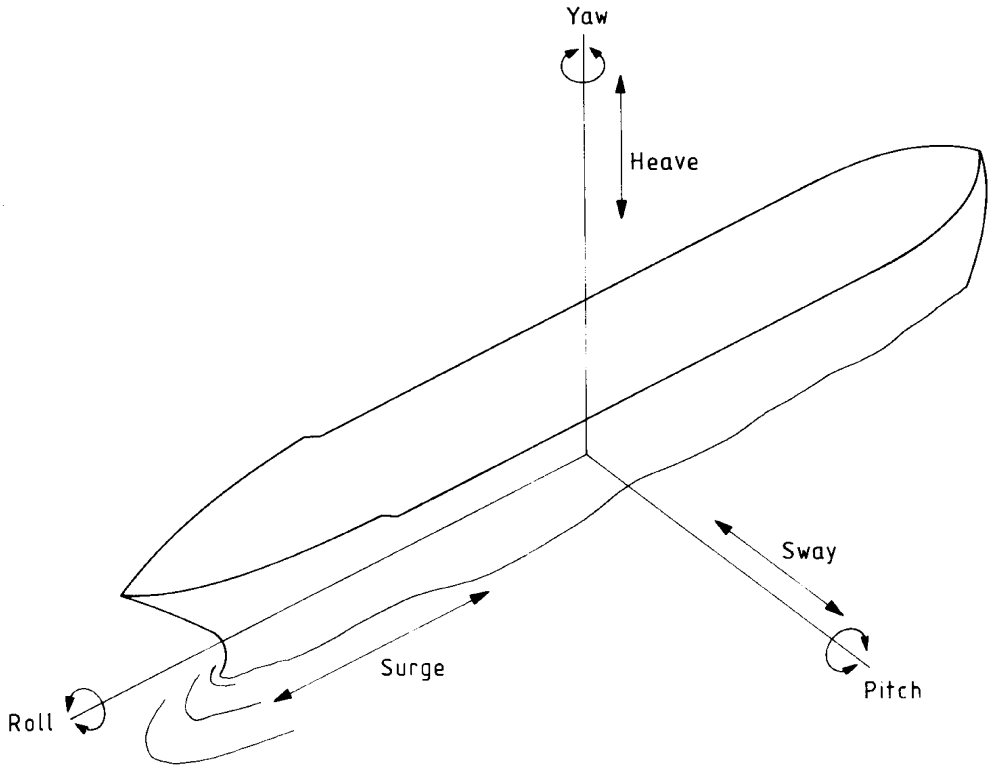

Figure 20 — The six degrees of freedom of vessel movement

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The computational method described so far can only deal with linear mooring systems and is inadequate for most situations. One method of solving the problem computationally with non-linear moorings is to obtain the vessel's position at a series of time intervals so that the appropriate non-linear mooring force tending to restore the vessel to its equilibrium position can be applied but at present the additional and important effect of non-linear wave forces has not been adequately represented in these time-development solutions.

Computational models are gradually being extended to take into account the effects of non-linear wave forces and non-linear moorings but they require considerable further development before they can be expected to produce reliable predictions of mooring loads due to wave action. Once developed, however, they should offer a method less costly than construction and operation of a large physical model.

31.4 Acceptable ship movements

31.4.1 *General.* As discussed in **31.1**, non-linearities in moored vessel response prevent general relationships being given for acceptable wave heights for ships exceeding approximately 3 000 t displacement. However, guidance on acceptable movements for such vessels is given below based on the following limiting criteria.

a) Limits imposed by safety: if exceeded, damage to the ship, other vessels and the port installations may result.

b) Limits imposed by cargo handling: if exceeded, loading and unloading of cargo can no longer be performed efficiently or may be halted.

c) Limits based on the experience of harbour authorities and ship operators.

In general, limiting ship movements obtained from a) will exceed those obtained from b) which, according to the small amount of relevant published information [26], for large ships at least, exceed those obtained from c).

31.4.2 *Limits imposed by safety.* This limit is usually specified by an upper limit to the mooring loads, e.g. the breaking load of the mooring lines or, if the lines are fixed to winches on the ship, the limiting load for the brakes on those winches. The degree of ship movement allowed in this case will clearly depend on the stiffness of the mooring system. A stiff system will not allow much movement of a vessel whereas a soft system will allow more movement before mooring loads reach their safety limit.

It is also important that the vessel should have time to leave the berth if the safety limit is likely to be exceeded. If tugs are required to assist in this manoeuvre, then conditions should not prevent them from doing their job satisfactorily. In some situations this could mean that the safety limit is governed by the operation of tugs rather than by mooring loads.

31.4.3 *Limits imposed by cargo handling.* The amount of ship movement allowed under this criterion will depend upon the type of vessel.

In the case of container ships where containers have to be lowered into, or raised from, cells within the ship's hold it is thought that total movements in excess of 1 m could significantly reduce the rate of loading and unloading, although a skilful operator of the container crane may be able to cope with horizontal movements in excess of 1 m. It should be noted that the larger the vessel the less it will respond in surge, sway and yaw to the primary wave system so that most of the horizontal movement of large container ships will be relatively slow, probably at periods longer than 20 s. The operators of container cranes may be able to cope better with these slow movements than with oscillations at swell or storm-wave periods.

In the case of tankers, the acceptable movement when taking on and discharging cargo is frequently fixed by the amount of movement allowed by a loading arm fixed to a loading platform. For oil tankers, the loading arm frequently allows movements of up to ± 4 m of surge and 4 m of sway or yaw off the fenders, although the full range may not be used if the moorings are not soft enough to allow such movements without producing excessive mooring loads. The amount of vertical movement allowed will depend upon the state of the tide and the ship's ballasting and the vertical adjustment on the loading arm. The full extent of upward adjustment should allow for highest tide, light ship and upward movement while the full extent of downward adjustment should allow for lowest tide, laden ship and downward movement. Typically, the total vertical movements allowed by loading arms in these two limiting situations will not exceed 1 m.

Similar limits apply to loading arms for liquid natural gas (LNG) and liquid petroleum gas (LPG) tankers.

In the case of roll-on roll-off (ro-ro) systems of cargo operation, there are likely to be stringent limits to the amount of movement allowed. Guide piles and tight moorings are often used to prevent horizontal movement. Also, vertical movements in excess of about 0.3 m are unlikely to be acceptable.

In the case of cargo operations which impose no obvious limitations in the link between the vessel and shore, the amount of acceptable movement should be defined by experience (see **31.4.4**).

31.4.4 *Limits based on experience.* Opinions of ship operators and harbour authorities on the degree of ship movement considered acceptable may vary considerably. The following criteria, based on the small amount of available published information [26], should be taken only as a general guide.

Roll, surge and yaw are considered to be the most dangerous movements in beam, head and quartering seas respectively. This is not unexpected as hydrodynamic damping is less for these three movements than for pitch, heave and sway. For ships above 30 000 dwt, roll and yaw movements should not exceed 0.5 m while surge should not exceed 1 m. For smaller vessels, slightly larger movements of up to 1 m for surge and yaw and movements of up to 0.7 m for roll are considered acceptable. For these smaller vessels sway movements call for the lowest limit at approximately 0.5 m whereas pitch and heave movements of up to 1 m are acceptable.

The figures given above take no account of the speed of vessel movement. An oscillatory movement of 1 m completed in say 10 s will appear far more alarming than the same movement completed in say 1 min and the acceleration in the latter case may be so low as to cause no sensation of movement for people on the vessel. The acceptable amount of long-period surge, sway and yaw movements of large vessels therefore may well be in excess of the figures quoted. This would not be true of ro-ro ships where even long period horizontal movement is unlikely to be acceptable during loading and unloading operations.

Section 5. Loads, movements and vibrations

32 General

32.1 Basic loads. In addition to dead loads and soil pressures the other forces which may act upon maritime structures are those arising from natural phenomena such as winds, snow, ice, temperature variations, tides, currents, waves and earthquakes and those imposed by operational activities such as berthing, mooring, slipping, dry-docking and cargo storage and handling.

Guidance is given in this section on the selection of relevant design parameters and methods of calculation to derive the resulting direct forces on structures, taking into account the nature and characteristics of the structures.

Unless otherwise stated, the design loads given in this section are unfactored. Guidance on appropriate partial factors for limit state design is to be found in subsequent Parts of this code.

Changes in operational practices and innovations in cargo handling and storage may tend to increase the loading requirements. In selecting design parameters, it is a matter of financial judgement what provision should be made for future changes in these fields, taking due account of the design life of the structure and possible restrictions on use. When parts of the structure have different design lives then each part should be considered separately in assessing what provision should be made.

The loading design criteria adopted should be clearly stated and recorded so that if it is proposed to change the operational use or to introduce new heavy equipment or storage systems, a check may be carried out to ensure that the new loads do not exceed those permitted under the original design criteria.

32.2 Dynamic response. Loads encountered in the maritime environment are usually dynamic, i.e. impulsive or fluctuating. The response of flexible structures to such loads may differ from that predicted by a quasi-static analysis which assumes that the displacement is equal to the loading increased by an impact factor divided by the static stiffness of the structure. In particular, where the frequency, f_c, of a forcing cyclic load approaches the natural frequency, f_N, of the structure in a relevant mode, the response of the structure to the forcing load will be magnified relative to that predicted by quasi-static analysis.

Typical frequencies of cyclic loads in the maritime environment are shown in Table 2 as a preliminary guide.

Table 2 — Typical frequencies of environmental forces

Environmental force	Typical frequency	Period
	Hz	s
Wind turbulence	0.05 to 20	20 to 0.05
Unsteady velocities in tidal flow	1.0 to 10	1.0 to 0.1
Vortex shedding in currents	0.5 to 3.0	2 to 0.3
Wave forces in regular wave trains	0.05 to 1.0	20 to 1.0
Seiches and long waves	0.001 to 0.05	1 000 to 20

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Dynamic effects will not normally be significant where f_c is less than $f_N/3$ or greater than $2f_N$, f_N being considered separately for the structure as a whole and for each important element of it. In every case a preliminary calculation should be made and f_N then compared with the expected frequencies of the loads to be applied.

Comparisons of frequency and dynamic response should be made for all conditions likely to apply throughout constructional stages, as well as for the completed structure. An approximate method for estimating dynamic amplification is given in clause **47** and guidance on the particular problem of vortex shedding is given in **38.3**.

32.3 Spectral loading. In many situations in the maritime environment the most important source of dynamic loading is from waves, either directly or as mooring loads through wave action on moored ships. Where the dynamic response is appreciable, the actual behaviour of the structure or moored ship may differ significantly from that determined by analysis or model testing applying only monochromatic wave loading. In such cases, the random nature of natural wave loading should be introduced by the use of the wave spectrum (see section 4).

To meet the needs of the offshore industry, mathematical methods have been developed for the analysis of the response of complex structures to spectral loading, using transfer function or deterministic integration techniques.

These methods may be applicable to certain inshore structures or parts thereof in fatigue or ultimate load calculations, for example jetties, pontoons or floating breakwaters, particularly in deeper water and more exposed locations. Less sophisticated methods may be applicable to other inshore structures.

In exposed situations mooring loads should be considered with respect to the wave loading spectra, but special techniques are necessary to deal with the coupled motions of the ship and non-linear behaviour of the mooring lines (see clause **31**).

32.4 Fatigue. Structural members subjected to fluctuating loads may suffer from fatigue failure. For maritime structures, problems due to fatigue are most likely to arise in steel members subjected to wave loading.

33 Soil pressures

Guidance on the calculation of soil pressures is given in section 6.

For the purposes of calculating soil pressures:

a) live loading on surfaces should be determined as described in clauses **44** and **45**;

b) extreme water levels should be derived as described in clause **37**;

c) ground pore-water pressures should be determined with reference to tidal range, soil permeability, drainage provisions and any artesian or sub-artesian ground water conditions;

d) allowance should be made for reduced passive resistance due to overdredging and/or scour.

34 Winds

For values of design wind speeds around the British Isles and for guidance on the calculation of wind loads on buildings, plant, equipment and other superstructure bodies, reference should be made to CP 3:Ch V-2. Elsewhere, the appropriate national wind code or meteorological station should be consulted.

The design wind speed for a structure or moored vessel is the mean wind speed corresponding to the shortest gust which will affect the whole structure or vessel at one time, having a return period of 50 years and taking account of the height of the structure or vessel and the wind speed/height gradient. In the case of moored vessels, the gust duration has also to be sufficient for the full line or fender strains to develop, taking account of the intertia of the vessel. This may lead to a reduced design wind speed and should be investigated where mooring loads are critical. Design wind speeds may also be limited by operational practices (see section 2).

For structures with shapes dissimilar to those for which values are given in CP 3:Ch V-2 and where wind loading is critical, values of aerodynamic drag coefficients may need to be obtained from wind tunnel tests.

In calculating the projected solid area, the possibility of ice forming on the structure should be considered and allowance made for the increased area where appropriate.

35 Snow and ice

For the coastal areas around the British Isles, accumulated snow is unlikely to affect the design of heavier maritime structures significantly. It should, however, be considered in the design of ancillary structures such as cargo sheds, port buildings and cargo handling installations, for which the appropriate imposed roof loadings recommended in CP 3:Ch V-1 should be used.

In the recent past, loading from floating sea ice has not been a problem around the British Isles and need not be considered for structures whose design life is of the order of 50 years. If, however, artificial offshore islands or other structures of very long projected life are to be considered, then long-term meteorological trends should be taken into consideration. Effects which do warrant general consideration are superstructure icing and expansion of small pockets of trapped ice. Guidance on superstructure icing is given in Appendix F of CP 3:Ch V-2:1972.

NOTE Specialist advice on particular aspects of ice and icing effects may be obtained from the Scott Polar Institute, Cambridge, England.

In countries where snowfalls and icing are likely to be more severe than in the United Kingdom or where loading from floating ice is expected to occur, reference should be made to local codes and standards.

36 Temperature variations

The loads, or load effects, arising from thermal expansion or contraction of the structure and from temperature gradients in the structure should be considered in the design, taking due account of local climate.

Maximum effective temperature ranges for various forms of suspended deck construction are given, for British coastal waters, in Table 3

Table 3 — Effective temperature range for maritime structure decks in British coastal waters

Type of deck	Maximum temperature	Minimum temperature
	°C	°C
Steel deck on steel beams	49	– 16
Concrete deck on steel beams	41	– 14
Concrete deck on concrete beams	35	– 9

37 Tides and water level variations

Maritime structures should be designed to withstand safely the effects of the extreme range of still water level from extreme low water (ELW) to extreme high water (EHW) expected during the design life of the structure. These extremes should be established in relation to the purpose of the structure and the accepted probability of occurrence (see **21.4**), but should generally have a return period of not less than 50 years for permanent works.

Extreme water levels, which may be caused by a combination of astronomical tides, surges, seiches and freshwater flow (see clause **10**), are required for the evaluation of:

a) overtopping;

b) hydrostatic pressures, including buoyancy effects;

c) soil pressures on quay walls;

d) levels of action of mooring and berthing forces, forces from other floating objects and wave forces.

In addition, the effect of waves (see clause **39**) and wave run-up (see clause **28**) should be considered in relation to overtopping and hydrostatic pressures.

When considering the effects of buoyancy on a structure, it is generally preferable to represent the buoyancy and gravitational loads as separate systems.

Reduced safety factors may be applicable in relation to soil pressures, mooring and berthing forces, forces from other floating objects and wave forces, when considered in conjunction with extreme water levels.

NOTE For guidance on methods of accessing the cross-probability of combined astronomical and surge tide levels, reference may be made to the Institute of Oceanographic Sciences, Bidston, Birkenhead, England.

38 Currents

38.1 General. For the purpose of this clause the design current speed should be the maximum value expected at the site during the design life of the structure and should be established in relation to the purpose of the structure and the accepted probability of occurrence (see **21.4**) but should generally have a return period of not less than 50 years for permanent works. Methods of determining water movement are described in clause **11**.

Loads imposed directly by tidal or fluvial currents on maritime structures may be classified as:

a) drag, or in-line, forces, parallel to the flow direction; *or*

b) cross-flow forces, transverse to the flow direction.

Current drag forces are principally steady and the oscillatory component is only significant when its frequency approaches a natural frequency of the structure. Cross-flow forces are entirely oscillatory for bodies symmetrically presented to the flow. For asymmetrical flow, the cross-flow forces should be determined from model tests or from similar situations.

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38.2 Steady drag force. For uniform prismatic structural members immersed in a uniform current the steady drag force, which acts at the centroid of the area normal to the flow, may be calculated from the expression:

$$F_D = \tfrac{1}{2}(C_D \rho V^2 A_n)$$

where

F_D is the steady drag force (in kN);

C_D is the dimensionless time-averaged drag force coefficient;

ρ is the water density (in t/m^3);

V is the incident current velocity (in m/s);

A_n is the area normal to flow (in m^2).

The values used for C_D and A_n in the above expression should be determined taking due account of the effect of marine growth on cross-sectional dimensions. Values of C_D are discussed below for various cross-sectional shapes. Guidance on marine growth in British coastal waters is given in **47.2.2**.

Where the incident current velocity is non-uniform or the member is gently tapered, the total force and the line of action may be determined by integration. Where the structure is fully submerged, where end effects may be significant, or floating or of significantly non-uniform shape, the drag force may require to be determined by measurement on models.

Where waves combine with a current to increase the drag force on a structure, the water particle velocities should be added vectorially and the resultant used to calculate the drag force from the formula given above. However, inertial forces may also need to be considered in such situations (see **39.4**).

Values of current drag force coefficients for circular section piles, tubes and cylinders are dependent on Reynolds number and surface roughness. Suggested values for use in the expression given above are given in Figure 24 for circular cylinders with different degrees of surface roughness, due to surface finish or marine growth.

Values of current drag force coefficients for non-circular sections are generally independent of Reynolds number, but depend on the angle of incidence. Values are given in Table 5 for various common non-circular pile shapes. For other shapes, values of C_D should be obtained from hydraulic tests, if not available from reliable published data.

38.3 Flow-induced oscillations

38.3.1 *Circular sections.* A bluff cylinder such as a pile situated in a current experiences fluctuating forces, both in-line and cross-flow, due to the shedding of vortices downstream of the cylinder. The frequencies of the fluctuating forces are directly related to the frequency of the vortex shedding and the amplitude of the fluctuating force increases as its frequency approaches the natural frequency of the cylinder, or of the structure as a whole, in any mode in which it is free to oscillate, by a feed-back system known as locking on. If, however, the inherent damping of the cylinder is sufficient to suppress the motion developing, then the locking on will not occur.

Piled structures are particularly vulnerable to this type of oscillation during construction and it may be necessary to provide restraint to the pile heads immediately after driving to prevent the possibility of oscillation in the cantilever mode.

The critical flow velocity V_{crit} is given by the expression:

$$V_{crit} = K f_N W_s$$

where

f_N is the natural frequency of the cylinder;

W_s is the diameter of the cylinder;

K is a constant equal to:

1.2 for the onset of in-line motion;

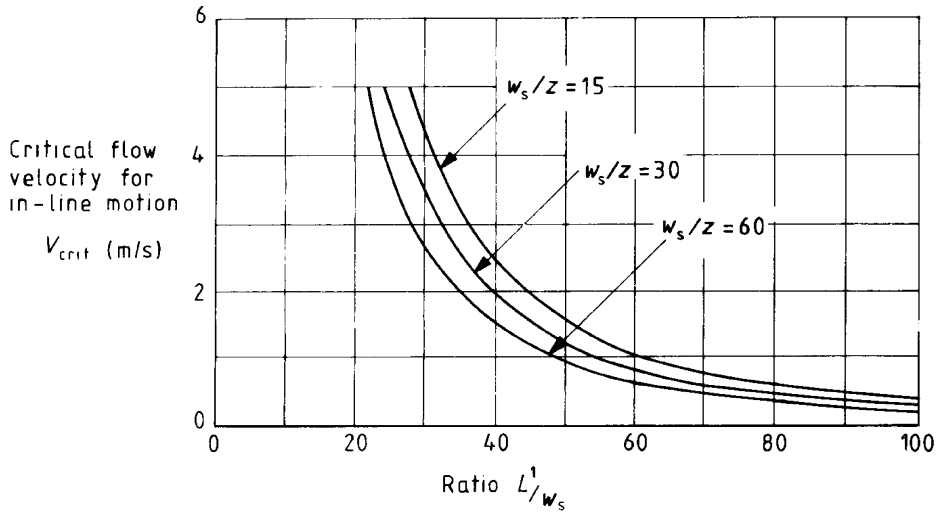
2.0 for maximum amplitude in-line motion;

3.5 for the onset of cross-flow motion;

5.5 for maximum amplitude cross-flow motion.

In the above expression the values for f_N and W_s should be derived taking due account of the effect of marine growth. However, the critical condition for flow-induced oscillation generally occurs during construction when marine growth is likely to be negligible.

Guidance on the calculation of the natural frequencies of structural members is given in **47.2.2**. However, for the common case of vertical thin-walled steel piles fixed at the bottom and pinned at the top, flooded and fully immersed in water with negligible marine growth, critical flow velocities for the onset of in-line motion are given in Figure 21. The curves are conservative in that they assume water surface is at the top of the pile. For similar piles, but with different fixity and/or different motion conditions, the critical velocities may be obtained by applying the modification factors given in Table 4 to the values obtained from Figure 21.



where

W_s is the diameter of pile;

z is the pile wall thickness;

L' is the overall pile length from deck to apparent fixity level.

Figure 21 — Critical flow velocity for circular piles for in-line oscillations

Table 4 — Modification factors for critical flow velocity

Motion	Pinned top fixed bottom	Canti-lever	Pinned top and bottom	Fixed top and bottom
Onset of in-line motion	1	0.23	0.64	1.46
Maximum in-line motion	1.67	0.38	1.07	2.43
Onset of cross-flow motion	2.92	0.67	1.87	4.25
Maximum cross-flow motion	4.58	1.05	2.94	6.68

Calculation of forces and displacements is not significant as vortex shedding is a resonant phenomenon in which the displacement gradually increases without increase in load. It can only be dealt with by prevention. Hydrodynamic spoilers can prevent excitation but such devices usually increase the drag force on piles. In permanent works, therefore, the properties of the structure and its elements should preferably be selected either:

a) to give a critical flow speed higher than the design current speed; *or*

b) to have mass and damping sufficiently high to prevent significant motion.

The first criterion will be satisfied if the current speed is less than $1.2 f_N W_s$. The second criterion will be satisfied if the mass damping coefficient is greater than 2.0 in the case of in-line motion and greater than 25 in the case of cross-flow motion, where the mass damping coefficient is calculated from the expression:

$$\frac{2\overline{m}\,\Delta}{\rho W_s^{\,2}}$$

where

Δ is the logarithmic decrement of structural damping which may be taken as 0.07 for most maritime structures;

ρ is the water density;

W_s is the diameter of the cylinder;

$\overline{m}$ is the equivalent excited effective mass per unit length, given by:

$$\overline{m} = \frac{\int_0^{L'} m_L\,(y(x))^2\,dx}{\int_0^{L'} (y(x))^2\,dx}$$

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where

m_L is the mass per unit length of the cylinder including contained water and the added hydrodynamic mass;

$y(x)$ is the bending mode shape as a function of the ordinate, x, measured from the apparent fixity level;

L' is the overall length of the cylinder measured from the apparent fixity level to deck level;

l' is the length from apparent fixity level to water level.

38.3.2 *Non-circular sections.* Non-circular cross sections are also subject to flow-induced oscillation but at a higher critical flow velocity and, once initiated, with greater amplitude. Such sections should be checked against the circular section limits quoted, using the maximum dimension normal to the direction of motion in place of the diameter in the formulae. If the actual flow velocity is close to the calculated critical flow the designer should refer to specialist texts for more detailed information [21]. Model tests may be required to determine the behaviour of particular shapes.

39 Waves

39.1 General. Direct wave loading on maritime structures is principally from waves with periods of up to 20 s. The prediction and analytical treatment of such waves is considered in section 4.

Design wave parameters are discussed in **39.3** and methods of calculation of wave forces are described in **39.4**. Where the maximum stresses due to wave loading constitute more than 40 % of the maximum total combined stresses, then the fatigue life should be checked as described in **39.2**.

39.2 Fatigue analysis. For fatigue analysis, an assessment should be made of the number of waves likely to occur during the design life of the structure within a number of ranges of height and period, typically eight similar height ranges for each of four period ranges. The maximum stress range for each height/period combination should then be determined, including dynamic magnification, where applicable, from which an assessment can be made using established stress against cycles to failure curves, i.e. *S-N* curves, of the number of waves needed to cause failure. The factor of safety against fatigue failure during the design life may then be determined using the Palmgren-Miner rule.

$$\text{Factor of safety} = \sum_{i=1}^{n_T} (n_i/N_i)$$

where

n_i is the number of waves occurring during the design life in stress range i;

N_i is the number of waves in stress range i needed to cause failure;

n_T is the total number of stress ranges considered.

39.3 Design wave parameters. For ultimate load analysis of structures having quasi-static response characteristics to wave loading (see **32.2**), the design wave parameters required for the purpose of this clause are the height and period of the average maximum incident wave having a return period of 50 years.

Where the dynamic response of the structure to wave loading is significant, consideration should additionally be given to the possible range of wave periods and associated maximum wave heights which would result in the greatest dynamic magnification (see **32.2**).

Design wave parameters should be obtained by the methods described in section 4, taking due account of the local conditions relevant to the site or structure.

If the highest waves are generated by hurricanes, typhoons or tropical cyclones, wave records may be analysed to relate wave heights and periods to cyclone paths and intensities. Design wave parameters should then be obtained by assessing the most severe probable cyclone travelling along a path to cause the worst wave conditions at the site.

39.4 Wave forces

39.4.1 *General.* Design wave forces should be derived from the design wave parameters defined in **39.3**, either by calculation, as described in this clause, or by physical model tests. However, caution is necessary in the case of vertical faced structures due to the difficulties of accurately modelling or calculating the shock pressures which sometimes occur in the prototype (see also **23.4** and clause **28**).

The magnitude of wave forces depends not only on the wave height and period and the dimensions of the structure, but also on the resulting hydrodynamic regime. This is controlled by the relationship between the width or diameter of the submerged part of the structure or member, W_s, and the wavelength, L, as follows:

a) for $W_s/L > 1$, reflection applies, see **39.4.2**;

b) for $0.2 < W_s/L < 1$, diffraction theory applies, see **39.4.3**;

c) for $W_s/L < 0.2$, Morison's equation applies, see **39.4.4**.

Linear wave theory may generally be assumed to be valid, but in shallow water, where the depth to deep water wavelength ratio, d/L_0, is less than 0.1, as the wave form starts to deviate significantly from sinusoidal, it may be found necessary to use either solitary wave theory or cnoidal wave theory for greater accuracy. Details of these theories are given elsewhere [16, 21].

39.4.2 *Reflective conditions.* Waves incident upon an infinitely long vertical surface may be reflected without breaking in which case a standing wave will be formed in front of the wall with a height, in the case of regular waves, twice that of the incident wave. This is known as clapotis. In actual cases, end effects will result in the standing wave varying in height along the wall about a mean value of twice the incident wave height. The variation may amount to 20 % for regular waves and be evident for at least two wavelengths along the wall from its end. A similar variation would occur with long crested random waves but the peak variation might be 15 % and the variation would be damped out within one wavelength from the discontinuity in the wall. Where such variations could be critical it is recommended that a site-specific investigation be made.

In certain depths, relative to the wavelength and wave height, waves may break against the wall producing impulsive loading which may be very large over small surface areas. The average wave pressure on a long structure from breaking waves and from reflected waves may be estimated as follows.

The vertical distribution of wave pressure may be assumed to take the form shown in Figure 23(a).

The design wave height (H_D) is taken as the mean of the 0.4 % highest waves ($H_{1/250}$) to be expected. Seaward of the surf zone $H_D = 1.8\ H_s$. Within the surf zone H_D is calculated for a depth "d_b" situated a distance in front of the wall equal to $5 \times H_s$ where H_s is calculated for a depth "d" equal to that to the sea bed at the wall face. H_D and H_s are obtained from Figure 11(b) to Figure 11(f) or from the corresponding equations given in **23.4**.

The elevation to which wave pressure is exerted is given by:

$$\eta^* = 0.75(1 + \cos\beta)\ H_{1/250}$$

Where β is the nominal angle between the direction of wave approach and a line normal to the breakwater. When the actual angle between the direction of approach and the normal is 15° or less, β shall be zero. When the angle exceeds 15°:

β = actual angle – 15°.

Wave pressure at the still water surface level is then given by:

$$P_{w1} = \frac{1}{2}(1 + \cos\beta)\ (\alpha_1 + \alpha_2 \cos^2\beta)\ \gamma_w\ H_D$$

Wave pressure at the base of the wall is given by:

$$P_{w2} = \alpha_3\ P_{w1}$$

Uplift pressure at the foot of the wall is given by:

$$P_u = \frac{\alpha_1 \alpha_3}{2}\ (1 + \cos\beta)\ \gamma_w\ H_D$$

where

$$\alpha_1 = 0.6 + \frac{1}{2}\left[\frac{4\pi \frac{d}{L}}{\sinh 4\pi \frac{d}{L}}\right]^2$$

α_2 is the lesser of the following:

$$\alpha_2 = \frac{1}{3}\left(\frac{1 - d_m}{d_b}\right)\left(\frac{H_D}{d_m}\right)^2 \text{; or}$$

$$\alpha_2 = \frac{2\ d_m}{H_D}$$

$$\alpha_3 = 1 - \frac{d'}{d}\left(1 - \frac{1}{\cosh 2\pi \frac{d}{L}}\right)$$

Where d, d' and d_m are as shown in Figure 22 and d_b is the depth of water at the location of H_D.

Alternatively α_1, α_2 and α_3 may be estimated from Figure 23(b), Figure 23(c) and Figure 23(d).

Note that if γ_w is in tonnes/m^3 and H_D in metres the resulting pressures are in tonnes/m^2.

The total pressure on the wall, F_w, is given by the following:

$$\text{If } h_c \geqslant \eta^* \quad F_w = \left(\frac{P_{w1} + P_{w2}}{2}\right) d' + \eta^* \frac{P_{w1}}{2}$$

$$\text{If } h_c < \eta^* \quad F_w = \left(\frac{P_{w1} + P_{w2}}{2}\right) d' + P_{w1}\left(2 - \frac{h_c}{\eta^*}\right)$$

A comparable method for calculating the wave pressure upon a wall under a wave trough, particularly for breaking waves, is not available. For the case when unbroken waves are reflected, that is where the incident wave height is less than 0.7 times the still water depth at the wall, the wave pressure distribution under the trough may then be determined according to the theory of Sainflou, as given in Figure 22.

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Local wave pressures arising when a wave breaks against a flat surface may be much larger than the average values calculated above. Analysis of prototype observations has led to the expression $P = \lambda \rho T v_c^2$ being proposed for local pressures. The value of λ depends upon the degree of aeration of the waves and on the basis of such observations 0.3 would be appropriate for a wave on a rough rocky foreshore with 0.5 applying to waves on a more regular beach. If the density is measured, in kg/m^3, the period in seconds and the phase velocity in m/s, the calculated pressure will be in N/m^2 (λ having the dimension of T^{-1}).

39.4.3 *Diffractive conditions.* Diffraction theory is likely to be of limited application to the maritime structures covered by this code. Reference should be made elsewhere [21] for guidance where necessary.

39.4.4 *Morison's equation.* For situations where the structure or member presents a relatively narrow obstruction to the passage of waves (see **39.4.1**), the total force imposed may be calculated from Morison's equation as the sum of a drag force and an inertia force, taking account of the phase difference between the two components.

As a conservative approximation, the wave force may be taken as 1.4 times the predominant component force. For $W_s/w_p > 0.2$ inertia is increasingly predominant and for $W_s/w_p < 0.2$ drag is predominant, where W_s is the width or diameter of the submerged part of the structure or member and w_p the orbit width of the water particles at the surface, is given by:

$$w_p = \frac{H}{\tanh \frac{2\pi d}{L}}$$

where

H is the wave height;

d is the still water depth at the structure;

L is the wavelength.

The Morison equation may be expressed as follows:

$$F_w = F_D + F_I$$

where

F_W is the total wave force normal to the axis of the member (in kN);

F_D is the drag force component (in kN) given by:

$$F_D = \int_0^{L_s} (\tfrac{1}{2} C_D \rho W_s |U| U) \, dL_s;$$

where

L_s is the submerged length of the member of which dL_s is an elemental length (in m)

F_I is the inertia force component (in kN) given by:

$$F_I = \int_0^{L_s} (C_I \rho A \dot{U}) \, dL_s$$

where

C_I is the inertia coefficient;

$\dot{U}$ is the instantaneous acceleration of the water particles, both measured normal to the axis of the member at the elemental length dL_s;

C_D is the drag coefficient;

ρ is the density of the water (in t/m^3);

A is the cross-sectional area of the member (in m^2);

U is the instantaneous velocity (in m/s).

Where the member extends through the wave surface the integration limit L_s will be governed by the instantaneous water level, η.

Suggested values of C_D are given for circular cylinders in Figure 24 and of C_{Ii} and C_D for some standard structural shapes in Table 5.

For closely grouped members, C_I may increase and should be determined from physical model tests. For floating structures, a modified form of Morison's equation should be used [21].

In assessing values of W_s and A, allowance should be made for the build-up of marine growth on the structure (see **47.2.2**). Expressions for U, $\dot{U}$ and η should be derived from appropriate wave theory (see **39.4.1**), any current velocity being added vectorially to the wave particle velocity to obtain U. U and $\dot{U}$ will normally be 90° out of phase for pure waves but this will be altered in the presence of currents. For vertical members, only horizontal particle velocities and accelerations need be evaluated. For inclined members, vertical components should also be taken into account.

The following expressions for instantaneous water level, particle velocities and accelerations are derived from linear theory.

$$\eta = \frac{H}{2} \cos\left\{ 2\pi \left(\frac{x}{L} - \frac{t}{T} \right) \right\}$$

$$u = \frac{\pi H}{T} \frac{\cosh\{2\pi(y+d)/L\}}{\sinh(2\pi d/L)} \cos\left\{ 2\pi \left(\frac{x}{L} - \frac{t}{T} \right) \right\}$$

$$v = \frac{\pi H}{T} \frac{\sinh\{2\pi(y+d)/L\}}{\sinh(2\pi d/L)} \sin\left\{ 2\pi \left(\frac{x}{L} - \frac{t}{T} \right) \right\}$$

$$\dot{u} = \frac{2\pi^2 H}{T^2} \frac{\cosh\{2\pi(y+d)/L\}}{\sinh(2\pi d/L)} \sin\left\{ 2\pi \left(\frac{x}{L} - \frac{t}{T} \right) \right\}$$

$$\dot{v} = \frac{2\pi^2 H}{T^2} \frac{\sinh\{2\pi(y+d)/L\}}{\sinh(2\pi d/L)} \cos\left\{ 2\pi \left(\frac{x}{L} - \frac{t}{T} \right) \right\}$$

where

η is the height of the water surface above still water level;

u is the horizontal water particle velocity;

v is the vertical water particle velocity;

$\dot{u}$ is the horizontal water particle acceleration;

$\dot{v}$ is the vertical water particle acceleration;

(all at time t at a distance x from the wave crest and, in the case of velocities and accelerations, at a height y above still water level);

d is the still water depth;

H is the wave height;

L is the wavelength;

T is the wave period.

39.4.5 *Wave slam.* For horizontal members close to the mean water level, account should be taken of wave-slamming loads caused by the sudden immersion of the member. Due to the impulsive nature of the loading, the dynamic response of the member may be particularly significant.

The vertical slam force for a cylindrical member may be determined from:

$$F_S = \tfrac{1}{2}\, C_S\, \rho V_\eta^2\, I W_s$$

where

F_S is the vertical slam force (in kN);

C_S is the slamming coefficient;

ρ is the water density (in t/m^3);

V_η is the vertical velocity of the water surface given by the rate of change of surface elevation η with time (in m/s);

I is the length of cylinder (in m);

W_s is the diameter of cylinder (in m).

Values of C_S have been determined empirically as 3.6 ± 1.0 where slamming loads are dominant, i.e. for Froude numbers greater than approximately 0.6, where the Froude number, Fr, is given by $Fr = V_\eta/\sqrt{(gW_s)}$. The variation of the water surface elevation with time according to linear theory may be obtained from the expression for η given in **39.4.4**.

40 Earthquakes

In the UK, where seismic activity is low, no allowance for earthquake effects need normally be included in the design of maritime structures.

Many countries subject to earthquakes make specific provision for seismic design within their building codes although there are considerable variations in approach and some are less complete than others. Few existing codes have been prepared specifically with maritime structures in mind.

The damaging effect of earthquakes is essentially, but not exclusively, the result of horizontal oscillatory accelerations of the soil mass being transferred to structures above ground the level through their foundations, base or pile support. The response of a structure to these accelerations will depend upon its type, mass and dimensions and the failure modes to which it may be subject. It is therefore important in seismically active areas to select a type of structure that will have as little sensitivity to seismic action as can be contrived.

The derivation of design parameters to provide for seismic loading is to a large extent a qualitative process. Specialist advice, particularly in relation to geophysical and geological aspects, should be sought where there is significant seismic activity or the danger thereof and reference should be made to local regulations and other authoritative references for guidance on the appropriate seismic loading to be used in design.

41 Berthing

41.1 General. In the course of berthing a vessel, loads will be generated between the vessel and the berthing structure from the moment at which contact is first made until the vessel is finally brought to rest. The magnitude of the loads will depend not only on the size and velocity of the vessel but also on the nature of the structure, including any fendering, and the degree of resilience they present under impact.

In the case of massive soil-backed quay walls, berthing loads will generally be resisted by passive soil pressure developed behind the quay wall with little effect on the structure itself and accordingly may require little consideration except to minimize damage to ships.

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Height of clapotis = 2 times height of free incident wave

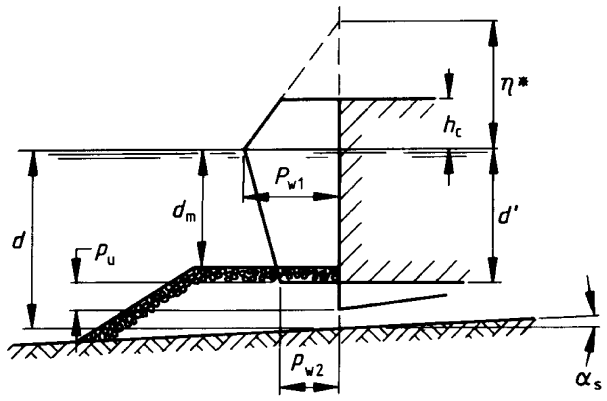
Clapotis set-up, $H_{0c} = \frac{\pi H_{inc}^2}{L} \coth \frac{2\pi d}{L}$

Maximum pressure on exposed face, $P_{max} = \rho g d + \rho g H_{inc}/\cosh(2\pi d/L)$

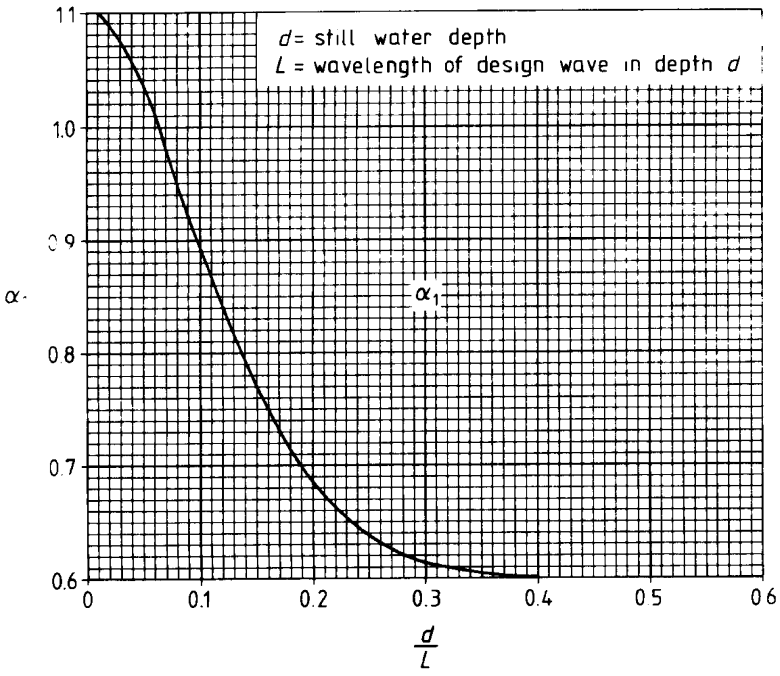
Minimum pressure on exposed face, $P_{min} = \rho g d - \rho g H_{inc}/\cosh(2\pi d/L)$

Maximum pressure on exposed face at still water level (SWL) is given approximately by $P_{w1} = P_{max} \{(H_{inc} + H_{0c})/(H_{inc} + H_{0c} + d)\}$

Figure 22 — Wave pressure distribution at reflective walls for non-breaking waves



(a) Vertical distribution of wave pressure



(b) Coefficient of wave pressure at surface dependent upon wave period

Figure 23 — Estimation of alpha values for calculation of wave pressure

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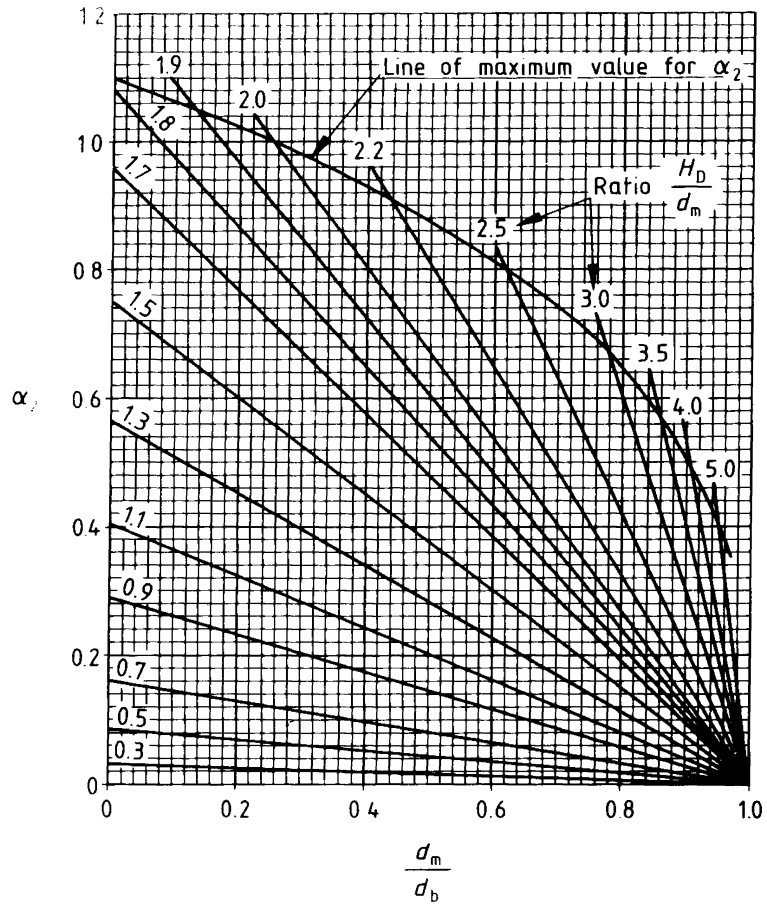
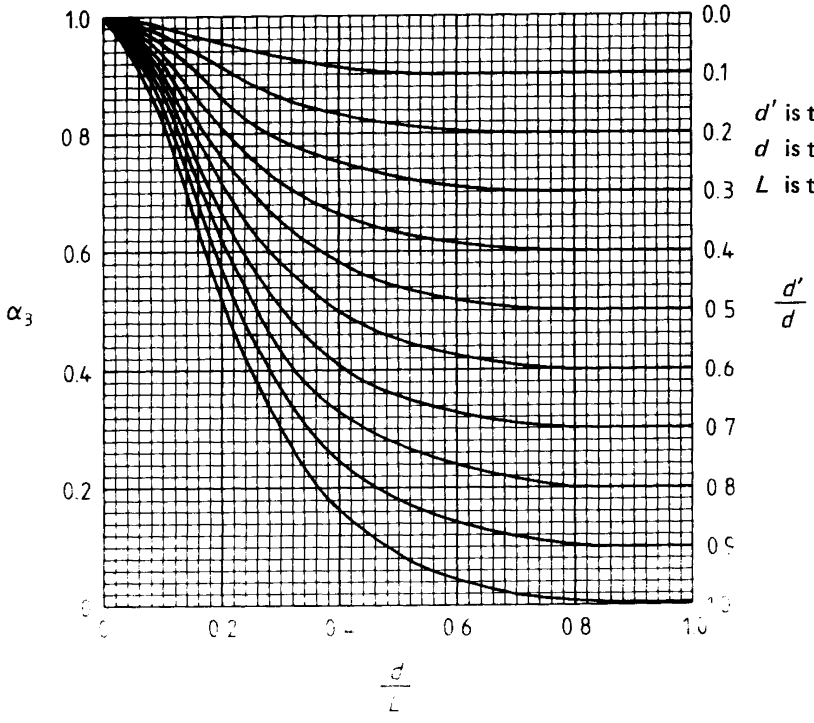


(c) Coefficient of wave pressure at surface dependent upon shoaling



d' is the depth at wall
d is the still water depth
L is the wavelength of design wave in depth d

(d) Ratio between wave pressures at surface and at depth d'

Figure 23 — Estimation of alpha values for calculation of wave pressure *(concluded)*

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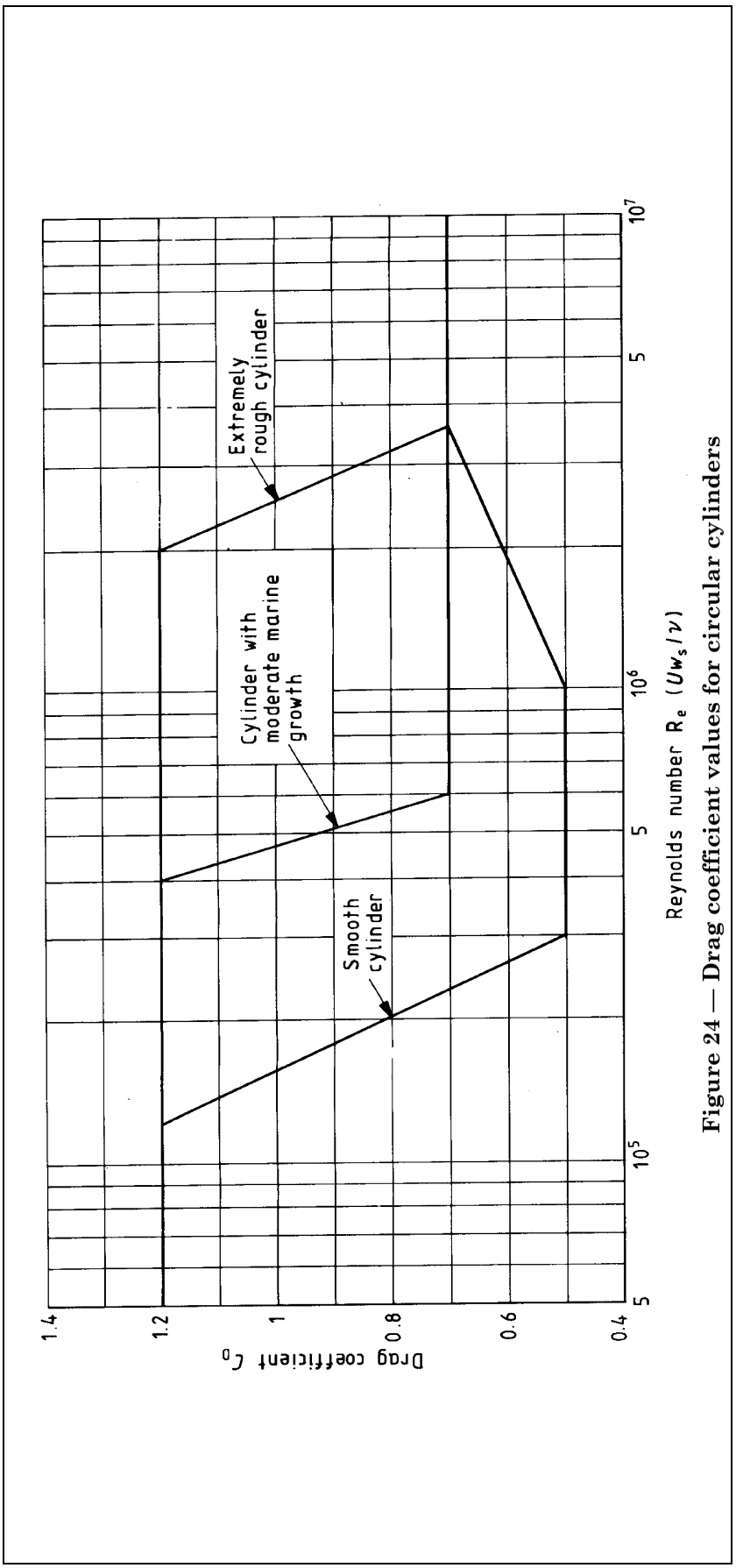

Figure 24 — Drag coefficient values for circular cylinders

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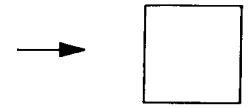
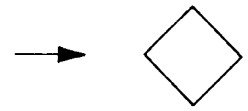
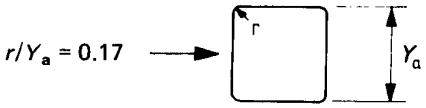
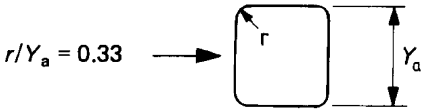
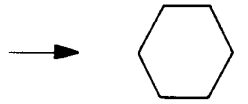
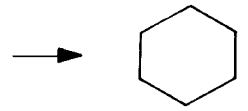
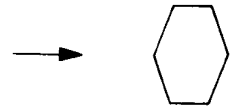
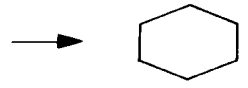
Table 5 — Drag and inertia coefficients for common structural forms

Cross section type	Attitude to flow or wave direction	Drag coefficient C_D	Inertia coefficient C_I
Circle	Any	See Figure 24	2.0
Square		2.0	2.5
		1.6	2.2
Square with rounded corners		0.6	2.5
		0.5	2.5
Hexagon		[a]	[a]
		[a]	[a]
Octagon	Any	1.4	[a]
Duodecagon	Any	1.1	[a]
Rendhex pile		1.3	[a]
		0.8	[a]

[a] The value for the appropriate square shape should be used unless more reliable values can be obtained.

The displacement of water as the ship nears the wall will have a cushioning effect and help to reduce the velocity of approach.

For other structures the berthing loads will generally be a predominant design consideration.

41.2 Operational factors. The velocity of approach of a vessel normal to the berth is a major factor in determining the kinetic energy of berthing. Since this is largely governed by operational practices, close liaison with the relevant operators and authorities is essential in the planning of new berths. Guidance on this, and other operational factors, is given in **19.2**.

It is always possible that catastrophic impacts may occur from ships drifting out of control. The necessity to provide against such impacts should be considered in relation to the consequences of such impact, including the cost of repair, particularly if alternative jetties or quays are available. The probability and the consequences of abnormal berthing loads occurring on account of breakdowns, failures and resource shortages should also be considered.

41.3 Fendering. Fenders are energy absorption devices whose principal function is to transform severe impact loading from a moving vessel into reactions which both vessel and structure can safely sustain. Fendering systems should be capable of not only sustaining maximum loads perpendicular to the fender faces but also any component parallel to the berthing face, both horizontally and vertically, that may result from ship movements (see also **47.2.3**, **47.3.4** and **67.2.2**).

41.4 Evaluation of berthing loads. Berthing loads transmitted to a structure comprise impact loads normal to the berthing face and friction loads parallel to the berthing face.

The impact load normal to the berthing face depends upon the following:

a) the berthing energy;

b) the eccentricity factor, i.e. the proportion of a centre-of-mass impact;

c) the load/deflection characteristics of the vessel, structure and fender system.

Guidance on the evaluation of design berthing energy is given in **41.5**.

The eccentricity factor is unity for a centre-of-mass impact, i.e. when the impact occurs opposite the centre of mass of a vessel. As the point of impact moves away from the centre of mass the energy imparted by the impact reduces. The energy imparted to the structure by a vessel during an unassisted angled approach will be divided between the points of first and second impact. Depending upon the disposition of the fendering system, the latter may be as much as twice the value of the former.

Guidance on the disposition and effect of fendering systems, which can be modified to produce hard or soft berthing characteristics, is to be found in subsequent Parts of this code.

The design friction load parallel to the berthing face should be taken as limited to μ times the maximum design impact load and should be considered acting in both the horizontal and vertical directions, where μ is the coefficient of friction between the two faces in contact.

41.5 Evaluation of berthing energy. The effective kinetic energy of a berthing ship may be calculated from the following expression:

$$E = \tfrac{1}{2}\, C_M\, m_D\, (V_B)^2$$

where

E is the effective kinetic energy (in kN m);

m_D is the displacement of the ship (in t);

V_B is the velocity of the ship normal to the berthing face at the time of impact (in m/s);

C_M is the coefficient of hydrodynamic mass.

If berthing energy records and ships' displacements are available from a similar jetty in a similar location, the energy imparted to the structure may be derived on a statistical basis. Care is necessary, however, as apparently similar situations may not be identical and may give rise, through variations in exposure, tidal conditions and difficulty of approach, to significantly different berthing energies.

Hydrodynamic mass is an extremely complex entity and at present cannot be evaluated satisfactorily. It is known to be influenced by form and condition, i.e. clean or otherwise, of the ship's hull, underkeel clearance, type of quay or jetty and the velocity of the vessel. Research has shown that the hydrodynamic mass coefficient can have values from 1.3 to 3.6. In the absence of more reliable information, a figure between 1.3 and 1.8 is suggested for C_M in the expression for effective kinetic energy.

Records of actual impact speeds of large vessels indicate that the maximum value of transverse velocity varies with ship size, ship arrival frequency and site conditions, extreme values in the worst cases being as much as four times those in the best cases.

The design value of transverse velocity, i.e. at right angles to the berthing face, should preferably be obtained from velocity of approach records at similar jetties elsewhere, provided that the exposure, tidal conditions and difficulty of approach are also similar. Where no records exist, the values given in Table 6 may be used, modified, as necessary, by the designer to accommodate the local factors described in this clause.

Table 6 — Suggested transverse berthing velocities

Displacement	Transverse velocity
t	m/s
Up to 2 000	0.30
2 000 to 10 000	0.18
10 000 to 125 000	0.16
Over 125 000	0.14

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Berthing manoeuvres can be either tug assisted or under self control (see **19.2**). Generally the two larger categories of ship shown in Table 6 will be helped by tugs and the others will berth without this assistance. Particular attention should be paid to local conditions when considering the berthing of the smallest range, i.e. less than 2 000 t, and whether or not the vessels have belted hulls.

The values given in Table 6 may be regarded as appropriate for normal circumstances and related to yield stress in the fender system. The choice of design velocity finally adopted for any category of ship should take account of the factors given below, which affect the control of the vessel:

a) ship size, type and frequency of arrival;

b) possible constraints on ship's movement approaching berth;

c) wave conditions likely to be encountered at berthing;

d) current conditions likely to be encountered at berthing;

e) wind conditions likely to be encountered at berthing;

f) the use of tugs;

g) whether or not speed-of-approach measuring equipment is fitted and used.

Increased frequency of use is likely to result in high approach velocities and this should be taken into account when designing berths such as ferry terminals.

Values of the design bow or stern velocity chosen by various authorities for a ferry or ro-ro vessel berthing under its own power vary considerably. They should be based on the stopping distances of the actual vessels which will use the berth in relation to the length of the berth. Figures used vary from 0.4 m/s to 1.0 m/s.

42 Mooring

42.1 General. Mooring loads comprise those loads imposed on a maritime structure by a vessel tied up alongside, both through contact between the vessel and structure or its fendering system and through tension in mooring ropes. They also include loads arising from manoeuvres of the vessel at the berth, including casting off, leaving berth, warping and heaving of breast lines during berthing, but exclude the impact and frictional berthing loads discussed in clause **41**.

Mooring loads in harbours and sheltered anchorages are principally caused by winds and currents, both of which are turbulent. Most of this turbulence is of shorter period than the resonant periods of large moored vessels so it will not excite significant dynamic response in such vessels. Small vessels, however, may respond significantly but the loads generated will generally not be critical for structural design.

Guidance on the evaluation of mooring loads caused by wind and currents is given in **42.2**.

At exposed locations, where wave loading is severe, the dynamic response of the vessel under restraint of mooring lines and fenders should be determined by model testing, mathematical analysis or other methods as described in clause **31**.

The operational aspects of mooring are discussed in **19.3** and guidance on acceptable conditions for moored boats and ships is given in clauses **30** and **31**.

42.2 Evaluation of mooring loads. For vessels up to 20 000 t loaded displacement, in the absence of a special assessment of mooring loads, bollards should be provided at intervals of 15 m to 30 m, with a load capacity as given in Table 7, which allows for more than one rope to be attached to each bollard. Where mooring ropes are returned through fairleads the corresponding load capacity of the fairlead should be as given in Table 7.

Table 7 — Nominal bollard and fairlead loadings for vessels up to 20 000 t displacement

Vessel loaded displacement	Bollard loading	Fairlead loading
t	kN	kN
up to 2 000	100	200
up to 10 000	300	500
up to 20 000	600	1 000

For vessels larger than 20 000 t loaded displacement, specific calculations should be carried out to determine the probable maximum mooring loads, taking account of the number, patterns, characteristics and pre-tensions of the mooring lines.

Design wind speeds should be derived as described in clause **34** except where it is regular practice for vessels to leave berths during high winds, in which case the design wind speed should be related to the practice followed.

Design current velocities should be derived as described in clause **38**.

Wind and current forces which may be experienced vary considerably with both the type and size of vessel, and are best established by the testing of scale models. In particular, the wind forces upon container vessels and other high sided ships are influenced greatly by detailed design, while very large tankers show marked variations in longitudinal force depending upon the design of bow. Furthermore, longitudinal current forces are very scale dependent. This is illustrated in Figure 25, where the coefficients for longitudinal forces on container ships have been scaled for the prototype, whereas the other longitudinal force coefficients are those measured on the models.

The method of calculation which follows should therefore be used only as a guide to the magnitude of wind and current forces on ships. Where such forces are critical to design, reference should be made if possible to model test results upon the vessels concerned or at least those on similar ships. The overall wind and current forces may be described either by longitudinal and transverse forces combined with a moment about a vertical axis, all acting at the centre of the vessel, or by two transverse forces, one at each perpendicular, combined with a longitudinal force. The latter method has been adopted for this standard and the magnitude and sense of the forces may be evaluated using the expressions given below.

For wind forces:

$$F_{TW} = C_{TW}\,\rho_A\,A_L V_W^{\,2} \times 10^{-4}$$

$$F_{LW} = C_{LW}\,\rho_A\,A_L V_W^{\,2} \times 10^{-4}$$

where

F_{TW} is the transverse wind force, forward or aft (in kN);

F_{LW} is the longitudinal wind force (in kN);

C_{TW} is the transverse wind force coefficient, forward or aft;

C_{LW} is the longitudinal wind force coefficient;

ρ_A is the density of air in kg/m^3 and may be taken to vary from 1.3096 kg/m^3 at 0 °C to 1.1703 kg/m^3 at 30 °C;

A_L is the longitudinal projected area of the vessel above the waterline (in m^2);

V_W is the design wind speed in m/s at a height of 10 m above water level.

For current forces:

$$F_{TC} = C_{TC}\,C_{CT}\,\rho L_{BP}\,d_m\,V_C'^{2} \times 10^{-4}$$

$$F_{LC} = C_{LC}\,C_{CL}\,\rho L_{BP}\,d_m\,V_C'^{2} \times 10^{-4}$$

where

F_{TC} is the transverse current force, forward or aft (in kN);

F_{LC} is the longitudinal current force (in kN);

C_{TC} is the transverse current force coefficient, forward or aft;

C_{LC} is the longitudinal current force coefficient;

C_{CL} is the depth correction factor for longitudinal current forces[a];

C_{CT} is the depth correction factor for transverse current forces[a];

ρ is the density of water in kg/m^3 and may be taken as 1 000 kg/m^3 for fresh water and 1 025 kg/m^3 for sea water;

L_{BP} is the length between perpendiculars of the vessel (in m);

d_m is the mean draught of the vessel (in m);

V_C' is the average current velocity over the mean depth of the vessel (in m/s).

[a] This is to be included when the depth to draught ratio is less than six.

Values for wind force coefficients are given in Figure 26, Figure 27 and Figure 28 for various angles of wind approach for various types of vessel, both in the ballasted and loaded condition. Values for current force coefficients are given in Figure 25 and correction factors for shallow water effects in Figure 29 and Figure 30. For very large crude carriers (VLCC's) both the magnitude of the longitudinal force and its direction change with decreasing depth and correction factors become difficult to apply. The total force remains small in comparison with transverse forces as the depth decreases at least until the depth to draught ratio reduces to 1.1. Figure 31 gives an envelope of force coefficients which could apply with varying angles of current approach, from the deep water case to a depth to draught ratio of 1.1. It should be noted that these values have been adapted from published interpretations of a limited number of wind tunnel and towing tank studies, to which reference should be made for further details [26A, 26B, 26C]. In addition, further information relating to liquid gas carriers may be found in reference 26D.

Typical values for the lengths, draughts and lateral areas of tankers and container vessels are given in Figure 32, Figure 33 and Figure 34. These figures may be taken as guides for the values to be inserted in the expressions given in this clause but, for working designs, it is recommended that the characteristics of the actual ships which will visit the berth be used.

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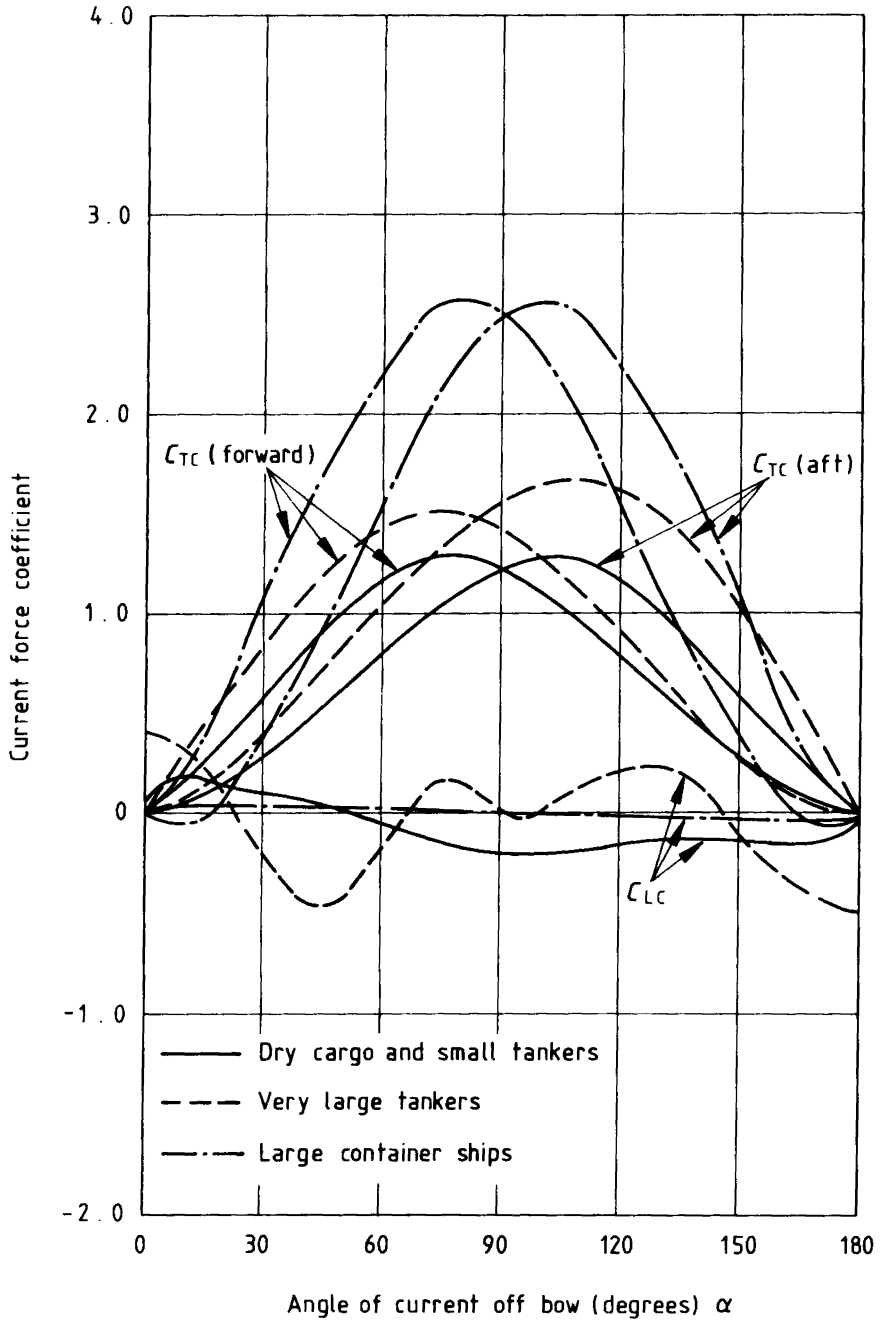

Figure 25 — Current force coefficients, all ships, deep water case

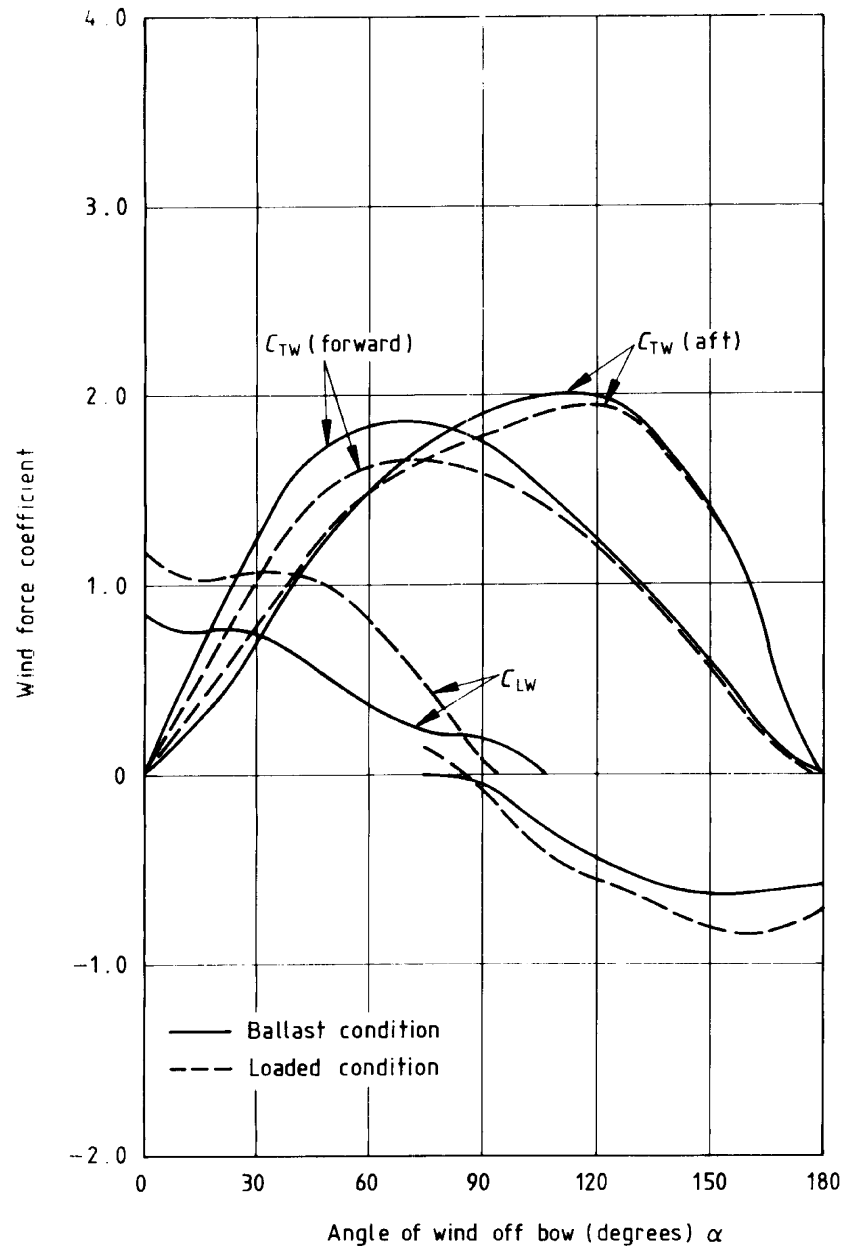

Figure 26 — Envelope of wind force coefficients for dry cargo vessels and small tankers

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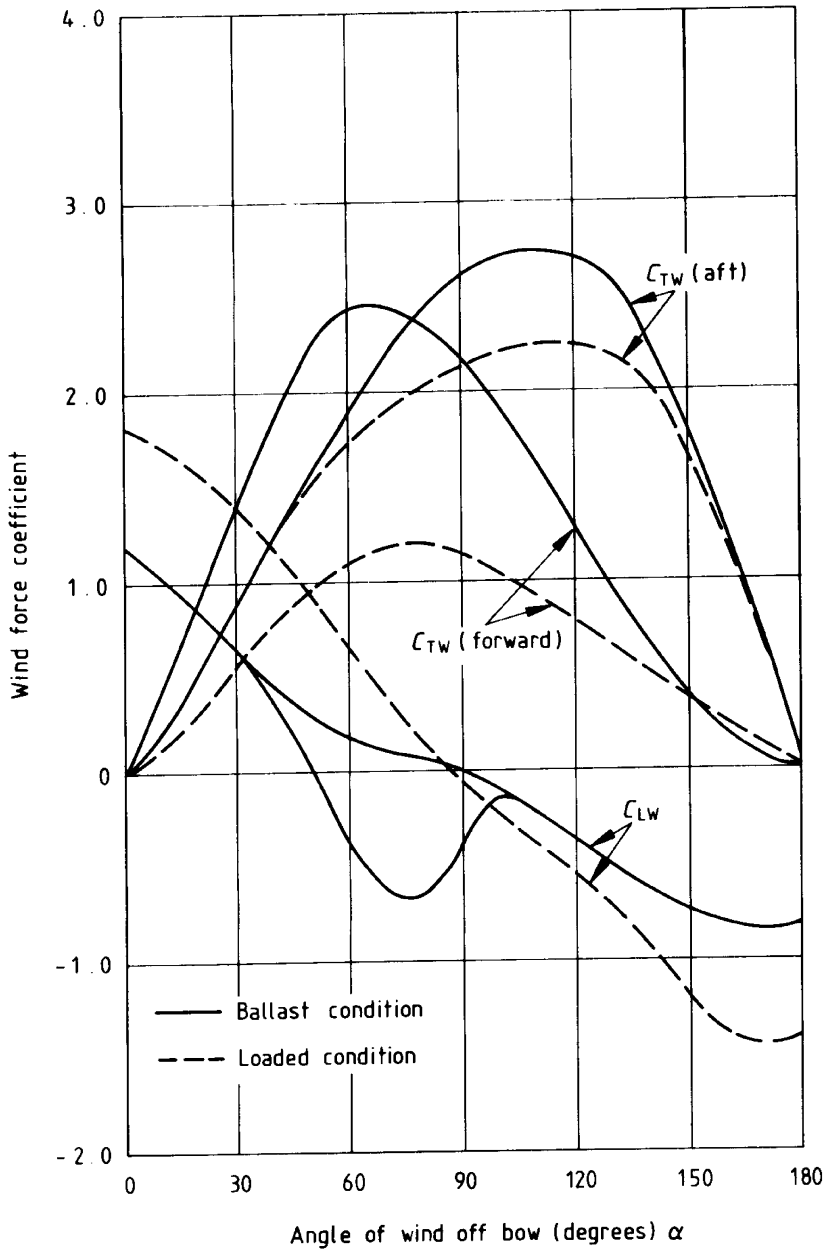

Figure 27 — Wind force coefficients for very large tankers with superstructures aft

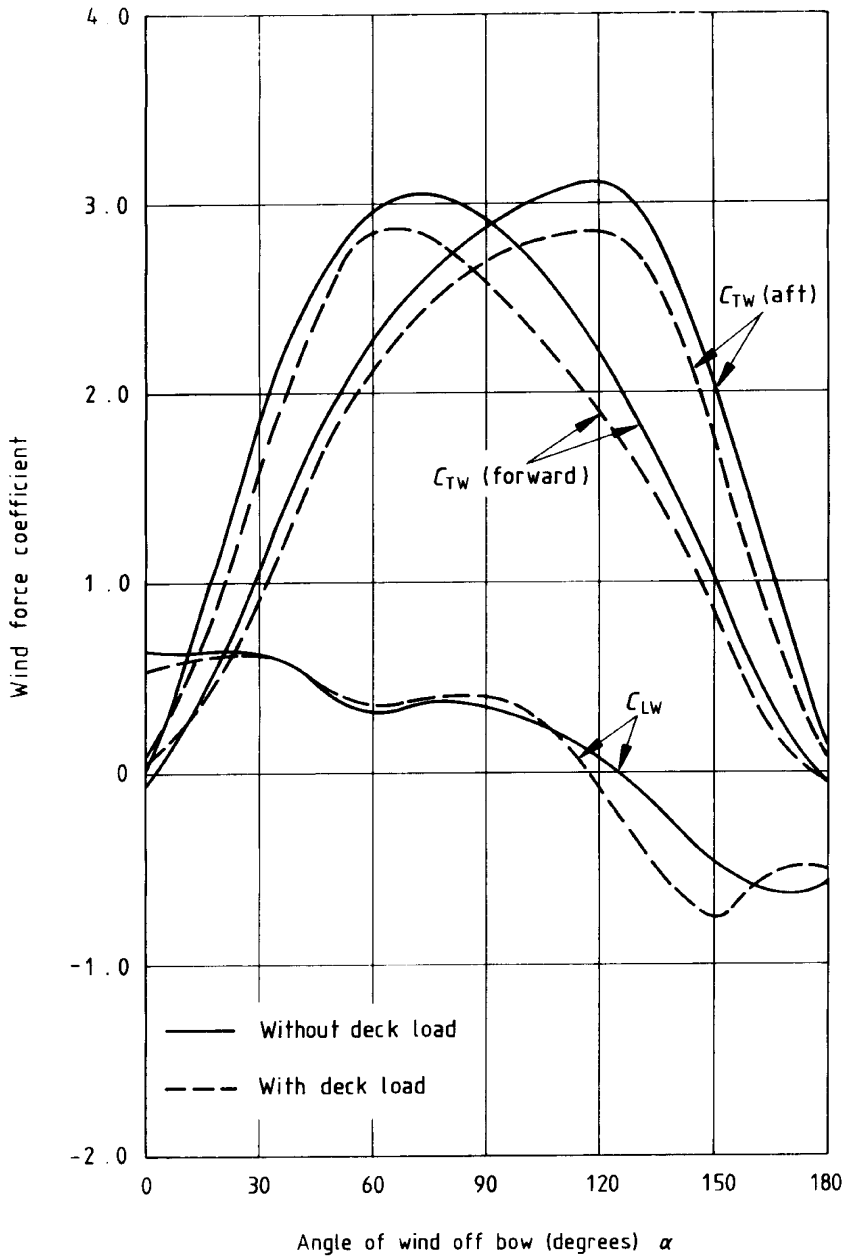

Figure 28 — Wind force coefficients for typical container ship

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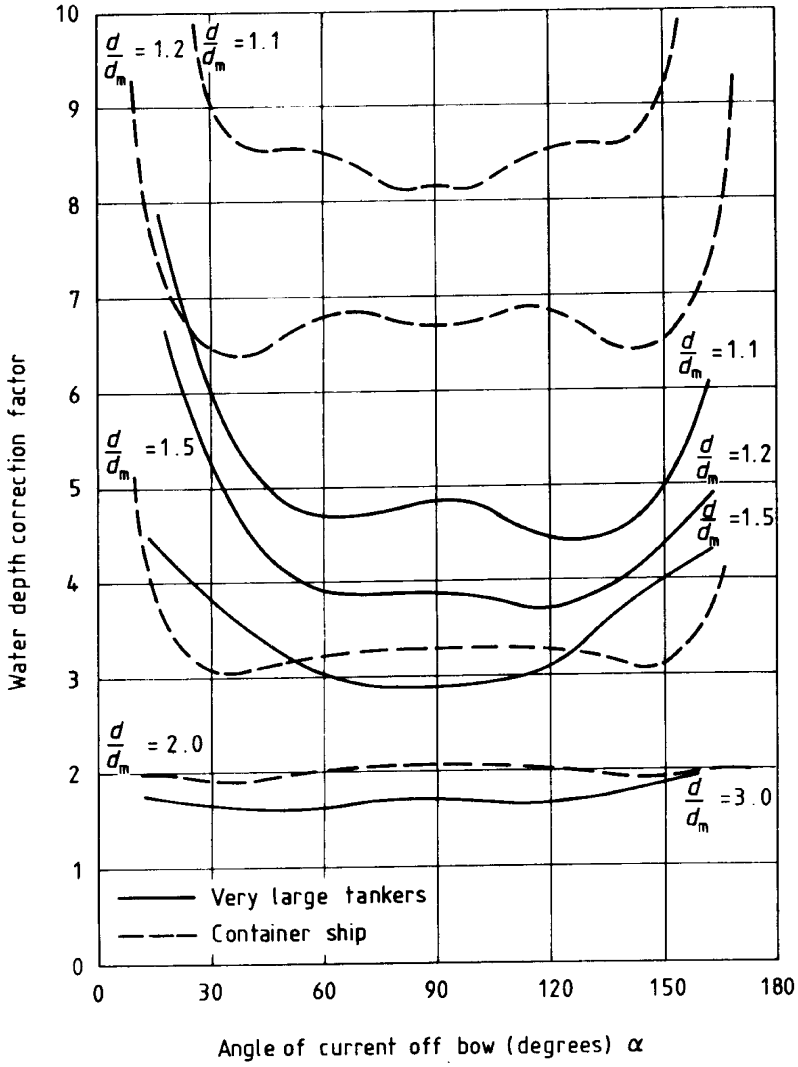

Figure 29 — Water depth correction factors for lateral current forces

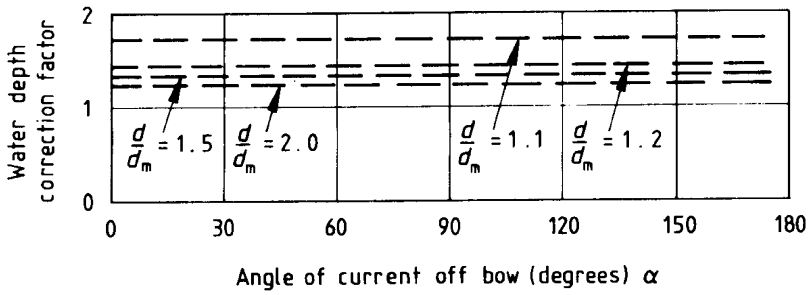

Figure 30 — Water depth correction factor for longitudinal current forces on container ships

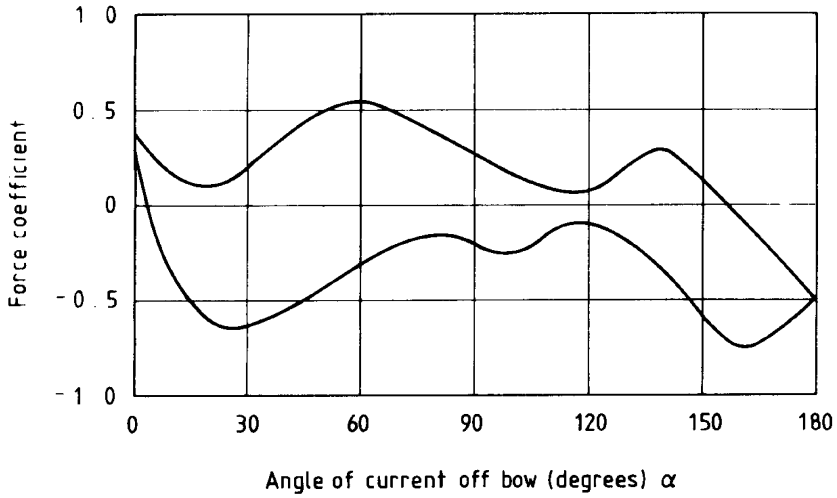

Figure 31 — Envelope of longitudinal force coefficients for VLCC's in shallow water

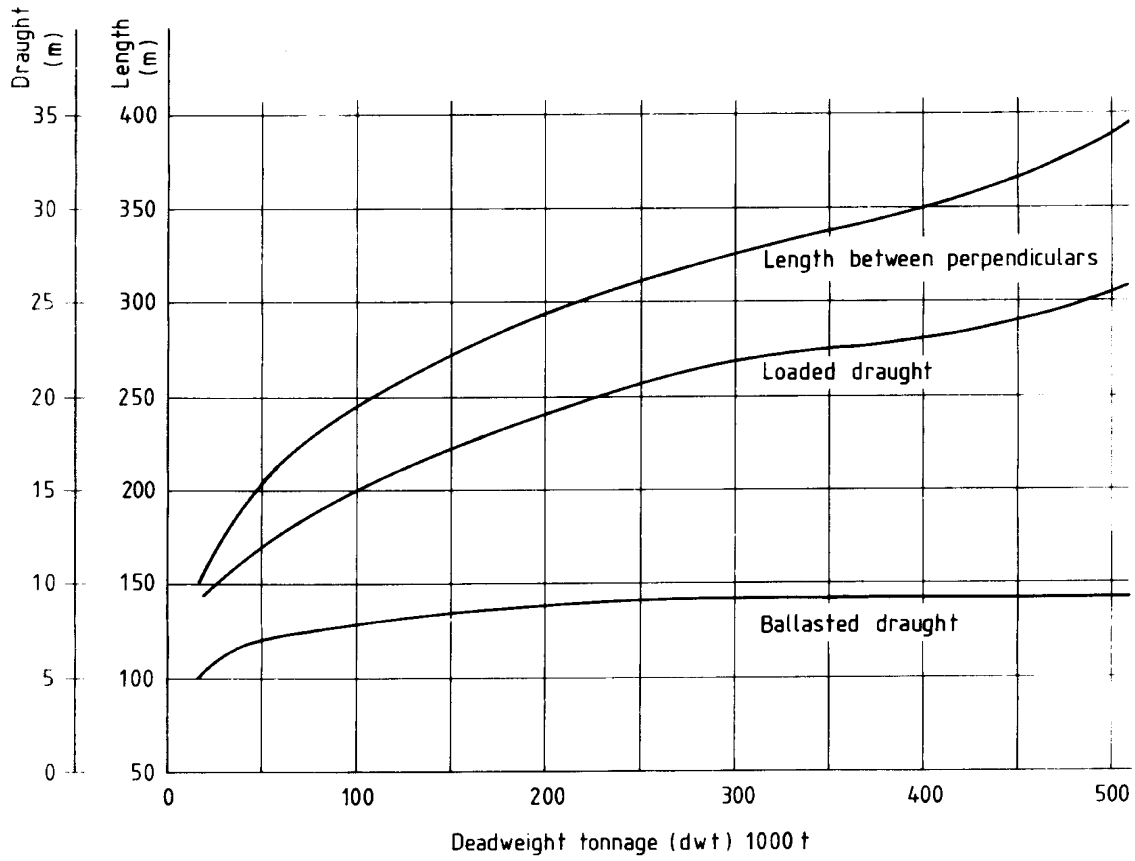

Figure 32 — Typical lengths and draughts of tankers

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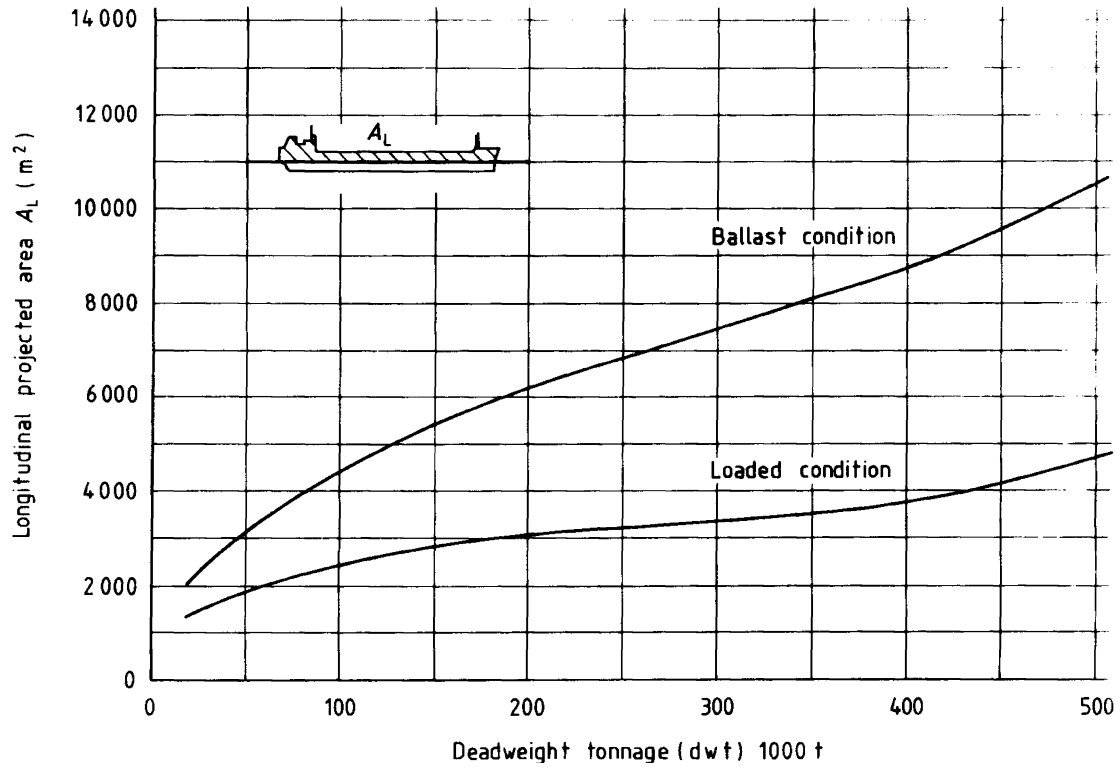

Figure 33 — Typical longitudinal projected areas of tankers

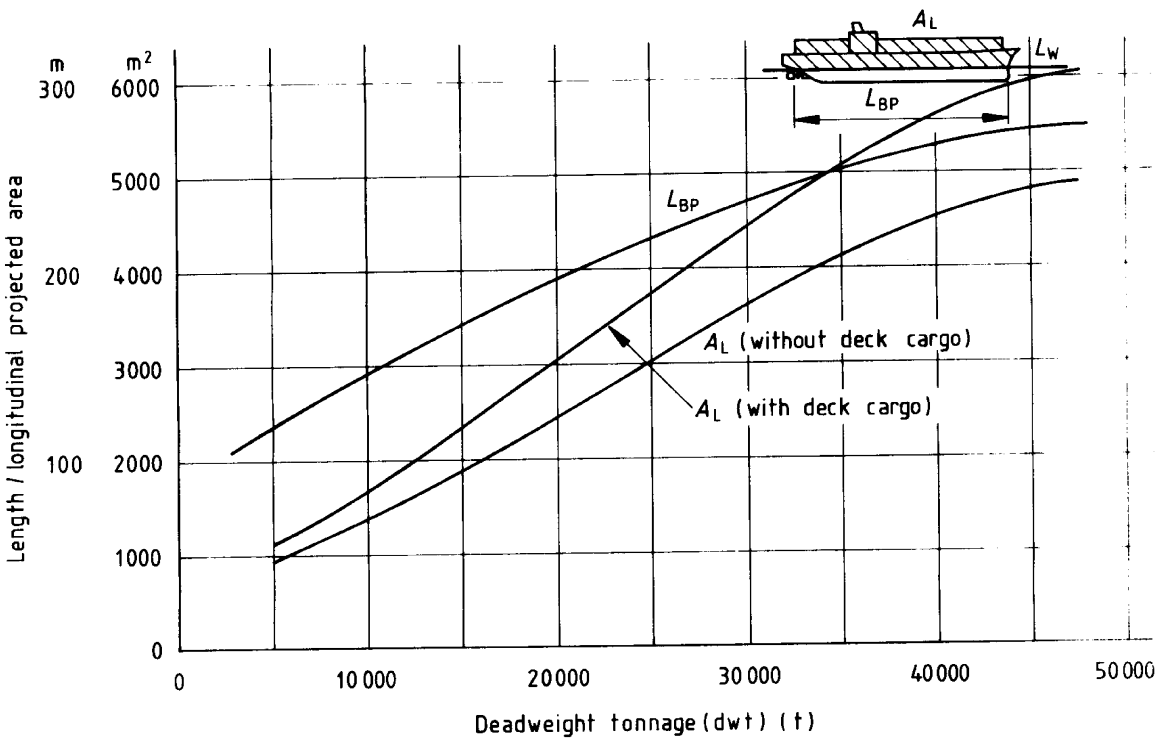

Figure 34 — Container ships: lengths and longitudinal projected areas

Figure 35 — *Figure deleted*
Figure 36 — *Figure deleted*
Figure 37 — *Figure deleted*

43 Docking and slipping

In addition to the vertical components of berthing, vessels are capable of generating significant direct vertical loads which, for certain maritime structures such as dry docks, floating docks, ship lifts and slipways, may constitute one of the major design loading considerations.

Although the total static vertical load is limited to the docking displacement of the vessel, the application and distribution of that load requires careful consideration of the operational criteria and the relative strengths and stiffnesses of both the structure and the expected vessels.

Further guidance on the selection of design loadings for these types of structure is to be found in subsequent Parts of this code.

44 Cargo storage

For specific storage installations such as silos, tanks, sheds or container stacks, the loading imposed on the substructure should be calculated, taking into account the weight of the store structure, the weight of the material stored and the effects of wind pressure and any snow loading. Where the loading may be increased or its distribution altered due to dynamic effects of setting down, filling or discharging, then these effects should also be taken into account.

For open stacks of bulk materials, the weight of material depends on the maximum heights, angles of repose and densities of the materials to be stored. For materials which are not free draining and where no protection is provided or where sprinklers are used, the saturated weight of the material should be used. Storage heights of 3 m to 15 m are commonly used. The use of edge retaining walls may lead to increased heights. Some typical values of dry bulk densities and angles of repose are given in Appendix A.

For other storage areas, the loading imposed depends on the height of stacking and effective density of the commodities as packaged.

The height of stacking may be limited by:

a) the height attainable with the stacking equipment;

b) the strength of the packaging;

c) the available height within sheds;

d) regulations or trade practice.

In the absence of more specific information, the typical values of stacking height given in Table 8 should be adopted. Typical values of effective stacked densities for some common commodities are given in Appendix A.

Table 8 — Typical stacking heights

Cargo type	Stacking height
	m
General palletized cargo	5
Timber or timber products	6 to 7
Metal products	3
Containers (according to handling system)	5 to 12
Fish	2.5
Vegetables and fruit	4

For aprons not exceeding 30 m wide, the assumed maximum stacking height may be reduced to:

General cargo 3.0 m;

Containers 7.5 m.

Storage loads may be treated as distributed loads, but consideration should be given to the local contact pressures imposed by container corner pads and parked trailer dolly wheels.

An allowance of an additional static load equal to the maximum unit load handled, but not exceeding 100 kN, should be made for setting down impacts on aprons and other areas where cranes operate.

Where hot or cold commodities are stored, consideration should be given to the effect of temperature on the ground or structure.

Storage areas for dangerous cargoes should allow for bunds or other protective measures.

45 Cargo handling and transport systems

45.1 General. Cargo handling and transport systems operating within ports may be classified as:

a) fixed and rail-mounted equipment;

b) conveyors and pipelines;

c) rail traffic;

d) road traffic;

e) rubber-tyred vehicles operating within the confines of the port, with or without lifting capacity;

f) tracked cranes.

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The loading imposed on structures should be considered in both vertical and horizontal directions. Consideration should be given in the design of superstructure works to the effects of collision impacts.

Operations of cranes will generally be halted at high wind speeds and the wind speed to be considered as acting on the crane while handling cargo may be limited accordingly. For maximum wind conditions, account should be taken of any special measures for stowage of the crane.

45.2 Fixed and rail-mounted equipment. For fixed and rail-mounted cargo handling equipment, loading should be calculated for the equipment to be installed taking into account the dead loads, live loads and wind and snow loading effects. Both vertical and horizontal forces should be considered. Live loading should include dynamic effects including travelling, slewing, braking and lifting. Collision loads between items of rail-mounted equipment or between one item of rail-mounted equipment and buffers should be calculated using a relative speed at impact of 1.0 m/s. The possibility of accidental loading due to a ship leaving berth unexpectedly should be considered.

45.3 Conveyors and pipelines. Loads from conveyors and pipelines should be calculated for each installation, taking account of rates of flow, material densities, changes of direction, temperature effects and the nature of the support framework.

45.4 Rail traffic. Rail traffic in ports differs to some extent from normal rail traffic in that:

a) speeds may be restricted;

b) crossings may be numerous and curve radii smaller;

c) shunting locomotives only may be used in some areas;

d) rail wagons are subject to setting-down impacts.

In the absence of more precise information, however, a nominal uniformly distributed loading of 50 kN/m^2 should be assumed for areas occupied by railway tracks corresponding to type RU loading. Type RU loading is defined in BS 5400-2 and where more detailed information on railway loading is required, reference should be made to that standard.

45.5 Road traffic. The nominal loading from road vehicles permitted on public highways in the UK is given in BS 5400.

For decks and pavements, consideration should also be given to the local effects of HB loading, details of which are given in BS 5400-2, and which may be taken to cover straddle carriers, side loaders and mobile cranes when travelling.

Outside the UK, heavier traffic loads may be permitted or encountered and account should be taken of local conditions.

45.6 Rubber-tyred port vehicles

45.6.1 *General.* Within the confines of a port, rubber-tyred vehicles are used which may impose considerably higher loads or local load intensities than highway traffic. Values of equivalent uniformly distributed loading are given in Table 9 for various common port transport systems. Detailed dimensions and intensities of local loading are given in **45.6.2** to **45.6.7**.

45.6.2 *Fork lift truck (FLT) loading.* This represents the loads from fork lift trucks. Nominal loads and dimensions are given in Table 10 for various ranges of fork lift trucks, expressed in terms of the payload mass capacity m_c. Where a range of dimensions is quoted, the value in the range which will give the most severe case for the structural element considered should be adopted. The dimension wheel spacing is defined here as the distance between centres of the inner wheels of an axle. Pairs of wheels should be assumed to be spaced at 0.4 m to 0.6 m centres. Wheel loads should be assumed to be uniformly distributed over either a square or circular contact area using the effective contact pressure quoted in Table 10, except where the capacity is less than 5 t, in which case solid rubber tyres may be used and the contact area should be assumed to be a rectangle, the length of the rectangle parallel to the axle being 150 mm. For heavy fork lift trucks it may be feasible to reduce the load intensities by increasing the number of wheels per axle from 4 to 6. For machines with payload capacities exceeding 25 t information should be obtained regarding the particular machines to be used.

Table 9 — Equivalent uniformly distributed loading for rubber-tyred port vehicles

Vehicle	Payload capacity	Maximum laden mass	Equivalent uniformly distributed load
	t	t	kN/m^2
Fork lift trucks	3	8	12
	5	12	15
	10	25	20
	20	50	25
	25	65	30
Side loaders	20	45	12
	40	90	15
Straddle carriers (for containers)	30	50	12
	40	70	15
Straddle carriers (other)	10	20	10
	20	36	15
	50	92	25
Tractor/roll trailer systems	20		10
	40		15
	80		20

45.6.3 *Side loaders.* Local wheel loading from side loaders used for the handling of containers and other cargoes may be considered to be covered by HB loading (see **45.5**). However, outrigger or jack loadings are also imposed by side loaders and provision should be made for these loads. Typical values of side-loader jack reactions are given in Table 11 which may be used in the absence of more precise information. The jacks are all located in a straight line at the spacing quoted in Table 11.

45.6.4 *Straddle carriers.* Local wheel loadings from straddle carriers used in the handling of containers and other cargoes may be considered to be covered by HB loading (see **45.5**).

45.6.5 *Mobile cranes.* Local wheel loadings from mobile cranes may be considered to be covered by HB loading (see **45.5**). However, provision should be made for the outrigger reactions and bearing pressures which may be imposed, relative to the maximum size of crane expected. When such cranes are only to be employed on an irregular and infrequent basis, a reduction of the load factor may be acceptable. Mobile cranes are rated according to their load-moment capacity and their maximum lift capacity at short radius. Details of the classes and reaction imposed by the outriggers are given in Table 12. The masses of the cranes have been taken for typical machines. In each case, the reactions should be taken as acting on two outriggers simultaneously at the outrigger spacing quoted in Table 12. The other outrigger loads may be calculated as sharing the sum of the maximum lift plus the machine weight, less the outrigger loads already calculated. No values have been given of imposed bearing pressures since the contact areas may be varied by the type of spreader used and by the use of packing, but pressures in excess of 1 000 kN/m^2 may develop unless restrictions are imposed.

45.6.6 *RT loading.* This represents roll trailers with hardrimmed tyres and their associated tractors. The plan dimensions are shown in Figure 38 and the loads are shown in Table 13 for various capacity trailers of mass up to 80 t. Tractor wheel loads should be assumed to be uniformly distributed over a circular or square area with an effective pressure of 700 kN/m^2. Trailer wheel loads should be assumed to be uniformly distributed over a rectangular area, the longer side, parallel to the axle, being 300 mm for trailers up to 20 t capacity and 400 mm for trailers of 40 t and 80 t capacity.

45.6.7 *Rubber-tyred gantry cranes.* Due to the wide range available, it is recommended that details be obtained of the particular equipment when it is proposed to use such cranes. Large gantry cranes for container handling may impose individual wheel loads up to 450 kN with contact pressures of 830 kN/m^2.

45.7 Tracked cranes. Where caterpillar tracked cranes are used, the imposed loading should be taken, in the absence of more precise information, from Table 14 according to the maximum size of crane expected. The maximum contact pressures may be imposed as a uniform pressure under one track or as the maximum of a triangular distribution under both tracks. The contact area considered should equal that required to support the weight of the crane and its load within the limits given. Tracked cranes are likely to cause local damage to blacktop and, to a lesser extent, to concrete surfaces unless protective mats are used.

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Table 10 — FLT loading: dimensions, loads and effective contact pressures

Payload capacity m_c	Machine mass	No. of wheels on front axle	Wheel spacing	Wheel base	Max. front axle load	Rear axle load	Effective contact pressure
t	t		m	m	kN	kN	kN/m^2
3	1.5 m_c	2	0.8 to 0.9	1.0 to 1.5	22 m_c	3.0 m_c	1 800
3 to 5	1.5 m_c	2	1.0	1.5 to 2.0	22 m_c	3.0 m_c	1 800
5 to 10	1.5 m_c	4	1.0	2.7	22 m_c	3.0 m_c	900
10 to 15	1.5 m_c	4	1.2	2.7 to 4.0	22 m_c	3.0 m_c	800
15 to 20	1.5 m_c	4	1.2 to 1.7	2.7 to 4.0	22 m_c	3.0 m_c	700
20 to 25	1.5 m_c	4	1.7 to 1.9	3.1 to 5.3	22 m_c	3.0 m_c	700

Table 11 — Side-loader jack reactions

Payload capacity	Mass unladen	Number of jacks	Jack spacing	Jack load	Contact pressure
t	t		m	kN	kN/m^2
23	30	2	2.5	250	1 300
27	40	4	2.5	160	400
40	50	4	2.5	230	500

Table 12 — Mobile crane outrigger reactions

Load-moment capacity range	Maximum lift capacity range	Typical mass of crane	Outrigger spacing	Maximum outrigger reaction
t•m	t	t	m	kN
50 to 100	6 to 10	50	6.5	250
100 to 200	8 to 20	75	8.0	450
200 to 300	15 to	100	8.0	550
300 to 500	30 to 36	130	8.0	800
500 to 900	30 to 40	200	8.0	1 100
900 to 1 300	50 to 80	240	10.0	1 500

Table 13 — RT loading: axle loads and effective wheel pressures

Roll trailer capacity	Tractor		Trailer		
	Axle line load		Number of axle lines	Maximum axle line load	Effective wheel pressure
	Front	Rear			
t	kN	kN		kN	kN/m^2
20	40	140	1	150	2 500
40	40	280	2	150	2 500
80	40	280	2	290	2 500

46 Channelized loading in pavements and decks

In assessing the effect of vehicular loading (including that from fork lift trucks and cranes) on pavements and decks allowance should be made for the effects of channelling where narrow aisles are used and for concentration of traffic close to the quay and around loading bays or shed doorways. In the absence of more direct information potential throughputs may be taken as shown in Table 15.

The conversion of axle loads to numbers of equivalent standard (8 050 kg) axles as applied in the design of highway pavements in the UK is of limited application to port pavement design because:

a) the axle loads involved may be considerably greater than the range of loads for which the conversion has been established;

b) the spacing of the wheels and the contact pressures imposed may differ significantly from those associated with highway traffic;

c) other effects, such as jack loads, setting-down impacts and concentrated loads from dolly wheels or container corner pads, may also have to be taken into account.

Where rubber-tyred gantry cranes are operated, the wheel loads should be ascertained for the specific equipment.

Table 14 — Loading due to tracked cranes

Maximum lift capacity	Unladen mass	Track spacing centre to centre	Track contact length	Track width	Unladen contact pressure	Maximum contact pressure
t	t	m	m	m	kN/m^2	kN/m^2
6	12	2.1	2.6	0.50	35	120
20	30	3.0	3.8	0.75	45	160
30	45	3.0	4.0	0.75	52	200
40	50	3.0	4.2	0.75	60	250
50	57	3.0	4.5	0.90	78	300

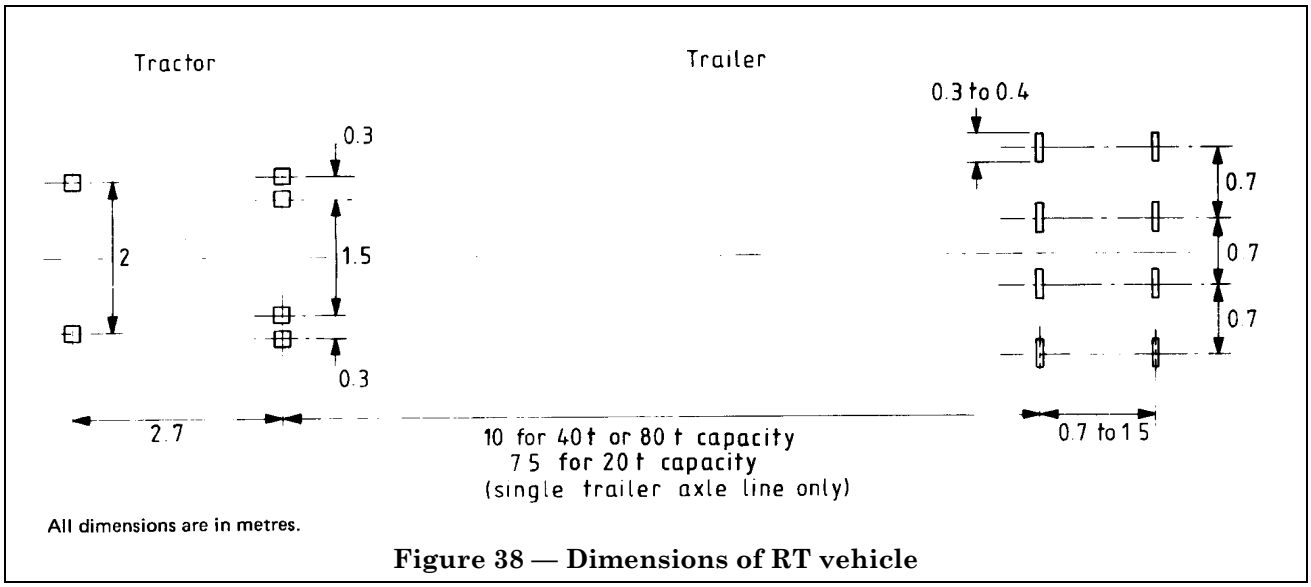

Figure 38 — Dimensions of RT vehicle

Table 15 — Typical throughputs for new cargo handling berths

Type of berth	Shifts worked per day	Throughput
		t × 10^3/year
Container berth	3	600 to 1 000
Ro-ro berth	3	200 to 600
Timber	2	200 to 300
Timber products	2	200 to 300
Steel products	2	200
General cargo	2	100

For straddle-carrier container operations, in the absence of specific information, laden carriers may be considered as equivalent to either:

1) 6 wheels, each imposing 130 kN, arranged in two parallel lines; *or*

2) 8 wheels, each imposing 100 kN, arranged in two parallel lines.

For special-purpose straddle carriers, information on wheel loads should be obtained for the particular machines.

Details of characteristic loads and contact pressures for fork lift trucks, side loader and mobile crane jack loads and roll trailer systems are given in **45.6**, from which the load or stress ranges to be considered may be derived. As a conservative assumption, the throughput may be assumed to be handled in units equal to the heaviest unit load, since the damaging effect of one pass of a vehicle transmitting a 10 t axle load, for example, is generally greater than that of two passes of the same vehicle transmitting a 5 t axle load. Alternatively, a more precise spectrum may be used if sufficient information on traffic patterns is available.

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47 Movements and vibrations

47.1 General. With regard to movement, many maritime structures are significantly different from the majority of land-based structures in that the more significant loads are frequently those which cause horizontal displacements. In addition, the loading is very largely of a dynamic nature and may give rise to larger displacements than the same loading applied statically.

There are many areas in which movement and vibration problems may arise but in practice the number of occurrences of problems has been small. However, with the modern tendency to build in deeper water to cater for larger vessels, problems with movement are likely to occur more frequently.

Maritime structures may be classified into two general groups: rigid and flexible. Rigid structures are those which cater for horizontal loading by carrying it mainly in direct compression and/or tension. This group includes earthfill structures such as quay walls and breakwaters, and structures incorporating raked piles. Flexible structures are those which carry horizontal loading by bending of the whole structure or individual members of the structure.

Rigid structures, by reason of their greater stiffness, have high natural frequencies and are not likely to experience large amplitude deflections due to the dynamic amplification of loading, but impulsive loads from berthing are likely to be more severe. Flexible structures have larger deflections under impulsive loading and, as a consequence, reduce its effect.

47.2 Assessment of movements

47.2.1 *General.* For the purpose of calculating movements the loadings on maritime structures may be classified into four groups:

a) cyclic;

b) impulsive;

c) random;

d) static and long term cyclic.

Each group of loadings will cause a different type of response in the structure and will require a different approach to the calculation of movement.

Apart from loads in the fourth group, loading on maritime structures is dynamic in character and therefore its effect is dependent on the magnitude of the load, its variation with time and on the response of the structure to that particular load. It is often not possible to calculate precisely the displacements resulting from these dynamic loads by the use of the traditional engineering method of representing the real dynamic loads by equivalent static forces. The analysis of the dynamic response of complex structures to any fine degree of accuracy is a specialist subject and for further information reference should be made to textbooks or similar sources [21].

Conservative assessments of the displacement of both flexible and raked pile rigid structures may be made using the approximate methods given below. For flexible structures in open sea or in water more than 30 m deep, the results may be too conservative and a more exact analysis may be necessary.

47.2.2 *Cyclic loads.* Cyclic loads are those which repeat in all essential features after regular intervals of time. They will cause cyclic displacement of the structure at the frequency of the loading and the maximum displacement is dependent on the relationship between the frequency of the applied loading and the natural frequency of the structure. The main cyclic loadings are:

a) wave loading from regular trains of waves;

b) vortex shedding from circular sections in steady currents;

c) vibrations from vehicular traffic;

d) vibrating loads from heavy, out-of-balance, rotating machinery fixed to the structure.

A reasonable approximation to the true response of the structure may be obtained by idealizing the structure as a single-degree-of-freedom system. This requires that the stiffness be represented by a single spring and the inertia by a single mass constrained to move in one direction only. Force is proportional to displacement for the spring and, given an initial impulse, the mass will oscillate at the natural frequency f_N such that:

$$f_N = \frac{1}{2\pi}\sqrt{\left(\frac{K}{m_e}\right)}$$

where

K is the stiffness of the spring;

m_e is the equivalent mass of the structure.

Under any applied cyclic loading of maximum value P and frequency of application, f_c, the maximum displacement, d_c, will be:

$$d_c = \frac{P}{K}\sqrt{\left\{\frac{1}{(1-(f_c/f_N)^2)^2 + (2qf_c/f_N)^2}\right\}}$$

where

q is the proportion of critical damping, equal to between 0.01 and 0.05 for maritime structures. In the absence of better information, q may be taken as equal to 0.01.

P is the maximum applied cyclic load.

The square root term in the above expression, which is a multiplier for the static displacement (P/K), is known as the dynamic amplifier. If this exceeds 1.2, more exact analytical methods should be used.

When using the above expressions to obtain approximate dynamic response, the stiffness of elements of the structure may be calculated from normal structural principles. Dynamic values of Young's modulus should be used.

In calculating the stiffness of a piled system, the effective length of pile from deck level to apparent fixity level should be used. The apparent fixity level will lie at a depth below seabed of between $4W_s$ for stiff clays and $8W_s$ for soft silts where W_s is pile diameter, but allowance should be made for possible scour.

The single equivalent mass representing the inertia of the system is an idealization of the actual mass distribution. This may be estimated by assuming a simple pinned support in the direction of motion, at the node at which the equivalent mass is to be placed, e.g. horizontally at deck level on a piled jetty, and performing a static analysis using the distributed masses as loads to give the reaction at the assumed support. This reaction may be taken as the equivalent mass, m_e.

The distributed mass of the structure should be taken to include:

1) the actual mass of the structure, including the mass of any attached marine growth, with no allowance for buoyancy;

2) the mass of water enclosed within the structure;

3) the mass of water externally entrained by the structure, including that entrained by the attached marine growth.

In the absence of more precise information, the extent and mass of marine growth in British waters may be taken from Figure 39. The added mass of entrained water may be estimated from Table 16.

Vortex shedding from circular sections produces a particular type of cyclic loading in which the displacement gradually increases without increase in load. It can only be dealt with by prevention. Further details are given in **38.3**.

Traffic-induced vibrations are unlikely to cause significant displacement and data recorded on similar structures should indicate whether or not there is a problem. Heavy rotating machinery may have some effect on structural elements although significant amounts of energy are unlikely to be present at the natural frequencies of the structural elements, being more likely to occur at 25 Hz and 50 Hz. Manufacturers of equipment should be asked to provide data on expected frequency and energy levels.

47.2.3 *Impulsive loads.* Impulsive loads cause the displacement to rise to a maximum and thereafter to decay cyclically about the original position at rest. The main impulsive loads are:

a) berthing forces;

b) release or failure of tensioned hawsers;

c) wave-slam forces on horizontal structural members due to the passage of the wave profile through the member;

d) crane snatch-loads when lifting cargo from moving vessels;

e) vehicular impact and braking loads from cranes and road and rail traffic.

The most significant of these loads is likely to be berthing impact. An estimate of the deflection under impulsive loading may be obtained by idealizing the structure as a single-degree-of-freedom system (see **47.2.2**). Using the displacement of the idealized structure as the unknown, the initial energy of the berthing vessel should be equated to the energy stored in the structure at maximum displacement. From the resulting equation the displacement may be calculated. The added mass of water moving with the berthing vessel should be included in the calculation (see clause **41**). The energy absorption of fenders incorporated in the structure should also be included in the calculation.

It will seldom be necessary to carry out more elaborate calculations than described in this subclause to arrive at a reasonable estimate of the structural movement because the approximations built into the estimation of berthing impact energy are greater than those brought about by the simple idealization of the structure.

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Individual horizontal members of the structure which occur in the wave-affected zone, i.e. between extreme wave crest and trough, will be subjected to wave-slam impact forces which may cause excessive deflection and hence excessive stress. No completely satisfactory theoretical method is available for the calculation of these forces; a suggested approximate method is given in **39.4.5**. The force so derived is applied as a static force to the member and the resulting deflection is taken to be the peak dynamic deflection.

47.2.4 *Random loads.* Random loads vary with time in a non-regular manner. The main random loads are:

a) normal wave loading;

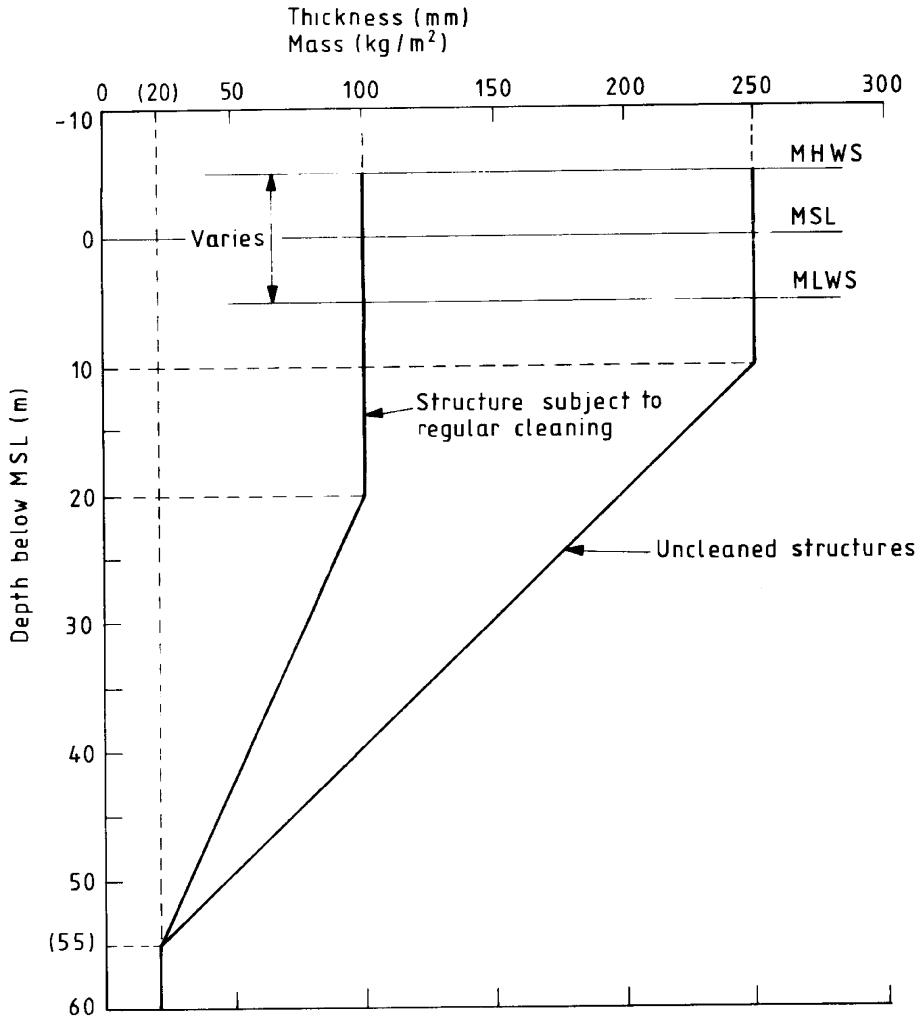
b) loading from wave-induced motion of a moored vessel;

c) seismic loading;

d) turbulent-wind loading.

The displacement of maritime structures can be conservatively estimated by approximating the random loading to a combination of cyclic and impulsive loads. For structures in depths of water greater than 30 m or in open sea conditions, the more complex and lengthy spectral analyses may be justified by savings in material and construction costs resulting from the more rigorous analysis (see **32.3**).

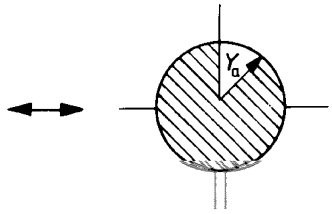
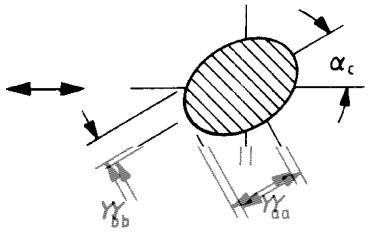
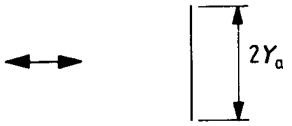
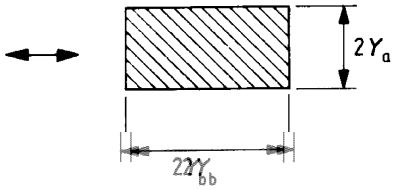
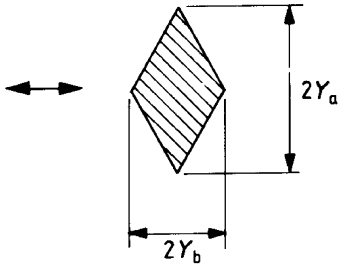
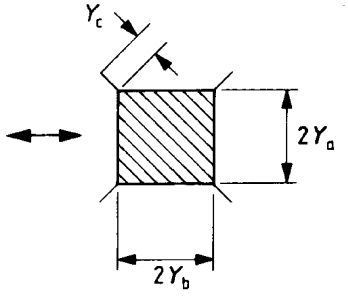
Normal wave loading is random since a normal wave train consists of numbers of waves of varying height and frequency in a complex combination. A conservative estimate of the displacement of a maritime structure under wave loading may be made by subjecting it to a series of loads resulting from a regular wave train of maximum design height with differing frequencies. Calculations may then be carried out using the methods described in **47.2.2**.



NOTE Based on effective growth density of 1 kg/m^3. Marine growth rates and densities may vary with individual location and environment.

Figure 39 — Thickness and mass of marine growth in British coastal waters

Table 16 — Added mass of entrained water

Cross-sectional shape of prismatic member		Added mass of entrained water per unit length[a]
Circular section		$\rho\pi\ Y_a^2$
Elliptical section		$\rho\pi\ (Y_b^2 \cos^2 \alpha_c + Y_a^2 \sin^2 \alpha_c)$
Flat plate		$\rho\pi\ Y_a^2$
Rectangular section		$K_1\ \rho\pi\ Y_a^2$ Y_a/Y_b / K_1 0.1 / 2.23 0.2 / 1.98 0.5 / 1.70 1.0 / 1.51 2.0 / 1.36 5.0 / 1.21 10.0 / 1.14
Lozenge section		$K_2\ \rho\pi\ Y_a^2$ Y_a/Y_b / K_2 0.2 / 0.61 0.5 / 0.67 1.0 / 0.76 2.0 / 0.85
Square section with corner projections		$K_3\ \rho\pi\ Y_a^2$ Y_c/Y_b / K_3 0.05 / 1.61 0.10 / 1.72 0.25 / 2.19

[a] ρ is the water density.

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Within a harbour the loadings from wave-induced vessel motion will be less than the berthing loads unless the harbour is subject to long wave activity caused by harbour resonance. For open sea berths in deep water, more detailed analysis will be necessary because of the greater flexibility of the structures, the more severe environment and the size of vessels. The analysis of the movement for such berths will require the assistance of specialist dynamicists and it may well be necessary to carry out model tests to establish the loading from wave-induced vessel motion (see clause **31**). It has been found in some cases that large vessels with low natural frequencies in roll can produce loadings when moored in open-sea conditions in excess of those due to berthing. This may occur if the natural frequency in roll of the vessel is below 0.25 Hz.

Seismic activity may cause very large displacements since the distribution of energy in an earthquake generally shows a marked concentration at the low frequency end of the spectrum, within which fall most of the natural frequencies of maritime structures.

47.2.5 *Static and long term cyclic loads.* Certain cyclic loads have such long periods that they act on the structure as static loads. The main loads of a static or quasi-static nature are:

a) dead load of structure;

b) earth pressure;

c) superimposed live load;

d) current loading;

e) tidal change loads;

f) time averaged wind loading.

Normal methods of static analysis may be used to calculate the resulting movements.

47.2.6 *Expansion and contraction.* Maritime structures should allow for expansion and contraction movements of the structure which occur both as short term daily movements and as long term annual movements. Maximum temperature ranges for calculation of movements in structures in British waters are given in Table 3 (see clause **36**).

47.3 Acceptability criteria

47.3.1 *Structural adequacy.* Displacements in maritime structures have to be such that the corresponding stresses are within the allowable limits. Since most live loadings on maritime structures are of a dynamic nature, the displacements are greater than those produced by an equivalent static force. In particular, a fluctuating force at or near the natural frequency of the structure or individual member of the structure will produce greatly increased displacements. In such cases it is recommended that more exact methods of calculation be adopted in preference to the simple method described in **47.2.2**.

47.3.2 *Compatibility of movements.* In the case of maritime structures composed of several distinct elements, differential movement between adjacent elements may occur, e.g. a jetty head and its approach structure may be separated by an expansion joint. In such cases suitable means for dealing with the relative movements should be incorporated in both the structure and any in-built services. In particular, special care should be taken to protect oil, gas and water pipes and crane rails from the effects of these relative movements.

47.3.3 *Stability of equipment on structure.* Large horizontal displacements of maritime structures will be associated with high horizontal velocities and accelerations. Mechanical equipment and plant on the structure will experience the same horizontal velocities and accelerations and, where this equipment is not rigidly fixed to the structure, e.g. a rail-mounted crane, it has to be checked against overturning. Rigid buildings on the structure may also be adversely affected by movements and vibrations.

Consideration should also be given to the psychological effect of sway on pedestrians, in that small movements seem to be much larger than they really area, giving rise to disquiet. In the case of structures intended largely for pedestrian or passenger traffic, designs should be sought which minimize this problem.

47.3.4 *Fendering limits.* Fenders can absorb a fixed amount of energy before failure. If fenders are designed to take only part of the berthing energy they should be provided with devices to prevent overload, which come into effect when the design limit of the fenders has been reached. The use of fenders to absorb part of the energy without such devices will result in their regular failure. Such devices may consist of simple stops or more elaborate absorbing devices.

Most fendering systems are weak along axes parallel to the berthing face of the structure. Considerable motion of berthing and moored vessels occurs along these lines. Fendering systems should therefore be designed to prevent failure due to this type of loading. Fender panels should be anchored in such a way as to limit longitudinal and vertical motion of the fenders parallel to the berthing face or they should be provided with low-friction facings which will limit the transverse forces on the fenders to acceptable values.

Section 6. Geotechnical considerations

48 General

A study of the surface and subsurface conditions at and near the site of proposed works is an essential preliminary to the design of maritime structures. Much can be learnt from an early assessment of the basic geology of the area, with particular attention to existing morphological processes. This can assist in determining the extent of geotechnical investigation and laboratory testing required and also aid in the interpretation and evaluation of the information obtained.

The study should include assessment of the characteristics of soil or rock formations which may be retained by structures or provide their foundations or which may be incorporated in or affected by earthworks in the form of dredging and reclamation. s should also include the collection of data on locally available materials for use in constructing the works, including the long term durability of these materials in the particular maritime environment.

These studies all form part of a site investigation for the works and the geological, geophysical and geotechnical aspects of the investigation are described in clause **49**. The collection of data for the site is followed by the selection of design parameters (see clause **50**) from which the behaviour of soils and rocks may be predicted. The next stage in the geotechnical design process is the calculation of earth pressures and earth resistance which may then be used, with other applied loadings, to check the adequacy of the proposed works.

Appropriate methods of calculation, drawing attention to the interdependence and interaction of different types of structure and their surrounding soil masses, are described in general terms in clauses **51** to **54**. Detailed applications to particular types of maritime structure are to be found in subsequent Parts of this code.

Descriptions of particular methods of construction are not appropriate to this Part of the code and the effects of constructional methods and procedures are generally referred to only where these could affect stability or design loadings. However, because of its essentially geotechnical nature, the use of thixotropic liquids in excavations has been included and this is described in clause **55**.

49 Site investigations

49.1 General. Procedures for the investigation of sub-surface conditions by means of trial pits, trenches, shafts, boreholes and geophysical surveying are described in BS 5930, in which guidance is also given for investigations over water. Guidance particularly relevant to the maritime situation is given below.

49.2 Existing data sources. The first stage of the site investigation should be a desk study of available published and unpublished information. Sources of information relevant to maritime structures include the following:

a) Admiralty charts and handbooks (the Pilot series);

b) Ordnance Survey maps and old maps;

c) meteorological data;

d) national and local government regulations;

e) geological maps and memoirs;

f) aerial and satellite photographs;

g) published papers on existing works in the locality.

Admiralty charts and Ordnance Survey maps covering a long period of years should be studied for evidence of changes in seabed levels and in the configuration of the foreshore which might indicate areas of erosion or accretion.

49.3 Site reconnaissance. A thorough visual examination of the site should be made. Exposures of soil or rock on the foreshore or coastal cliffs should be examined to obtain preliminary information on the geology of the site and to obtain any evidence of erosion, accretion or instability. The appearance of existing structures and earthworks should be examined for signs of subsidence or ground heave.

The landforms of the foreshore should be studied in relation to seabed contours and information on littoral currents, in order to delineate areas of active erosion and accretion and to provide a basis for predicting changes which might result from construction of the new works.

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In particular it should be remembered that many maritime sites will be areas of recent geomorphological development and that the nearer the site is to the shore the more active such processes will be. The time-scale of these changes may often be comparable with the life span of the project and they may well be the most significant feature of the ground conditions affecting many maritime works.

It is important to recognize characteristic features of coastal sediments. Those laid down in estuarine conditions are commonly flocculated silts and clays although the flocs can also entrap coarser sediments brought into the estuary by coastal currents. Thus estuarine deposits are often characterized by great depths of very fine soils. Deltaic deposits have been laid down seasonally and therefore usually contain alternating bands of coarse and fine sediments corresponding to the variation in transporting power of the river in winter and in summer. Beach deposits will in general have resulted from coastal rather than river transport and will usually be of medium to coarse size. They may arise from small embayments and would therefore be very local whereas they may occasionally be from lengthy strands although these are likely to be exotic.

Organically formed deposits are frequently of significance. They are mainly calcarous of which coral and calcite-exuding algae which cement detrital material are two examples. All such materials are subject to leaching and may therefore be cavernous. Particular attention should be given to such a possibility, especially where major structures such as breakwaters are proposed. Sedimentary rocks formed by induration are also subject to cavernous formation so that a site investigation programme arranged to locate such features would be particularly desirable in these types of soils.

49.4 Exploratory drilling, sampling and in situ testing. Sub-surface exploration for maritime works in soils and weak weathered rocks is generally achieved by cable percussion drilling and, in the stronger rock formations, by rotary core drilling. In the case of works sited over water, the drilling operations should be undertaken from a moored vessel, a jack-up barge, or a temporary platform assembled on the seabed.

Trial pits and shafts are limited to ground investigations for shoreside structures. They have useful applications in assessing ground conditions for shallow anchorages for earth-retaining structures and for the foundations of lightly loaded installations, particularly where these are sited in made ground when direct visual examination of the fill material is desirable.

Where dock basins and lock chambers are to be excavated on the landward side of a site, the ground conditions can be examined in large diameter shafts sunk by hand excavation or by mechanical drilling equipment. These are desirable where the requirements for dealing with ground water and conditions for excavation in rocks are more reliably assessed by direct visual inspection of the material en masse.

The diameter of boreholes drilled by cable percussion methods should be sufficient to recover undisturbed soil samples 100 mm in diameter. Complex soil stability problems could justify drilling larger diameter boreholes to obtain undisturbed samples 200 mm or 250 mm in diameter for special laboratory testing [27].

The diameter of rotary drilling in rocks should be such as to achieve, as far as practicable, complete recovery of rock cores, including all very weak and friable material. This is particularly important where an assessment has to be made of the required penetration of piles into weak rocks to obtain resistance to lateral and uplift loads and to evaluate the pull-out resistance of drilled anchorages. Complete recovery of weathered rock cores is helpful in assessing dredging characteristics, particularly where the use of explosives may be needed to facilitate removal by dredger.

Standard penetration tests may be made in boreholes drilled by rotary or cable percussion methods in granular soils to obtain a measure of their relative density and hence to derive parameters for the angle of shearing resistance and deformation modulus. Static cone penetration tests provide more reliable indications of the bearing capacity and settlement of structures on granular soils but these tests are usually limited to the exploration of the landward side of a site. The problem of obtaining sufficient reaction to the thrust on the cone as it penetrates the soil makes this type of test difficult to execute from a moored vessel, particularly in the case of exploration in dense soils at a shallow depth below the seabed, although seabed or down-hole supported equipment is available to overcome this.

Standard penetration tests can be made in weak rocks at the end of each run of the core barrel when rotary drilling and the diameter of the drilled hole should be of the size required to accommodate the test equipment. The observed penetration resistance values can aid prediction of the penetration of piles and the results can be correlated with the state of weathering of a particular type of rock, from which an assessment can be made of its bearing capacity and deformation characteristics.

In situations where site investigations are land-based and where close estimates are required for the deformation of heavily loaded structures, for example the lateral and vertical movements of sills supporting caisson-type gates for locks and graving docks, plate bearing tests should be made. The inclination of the load applied to the plate should be considered in relation to the direction of loading imposed by the structure and the inclination of critical soil or rock strata. Plate bearing tests should be made only after detailed exploration of the ground by boreholes and trial pits so that conditions representative of material having the highest and lowest compressibility can be selected for test.

49.5 Layout of boreholes and trial excavations. The layout of exploratory boreholes, trial pits and shafts should cover the full longitudinal and lateral extent of the works. In the excavation for wharves and quay walls, the boreholes should be spaced at intervals along the waterside frontage of the structure. In a transverse direction the boreholes should be sited to include exploration of the ground conditions for any anchorages on the landward side and to investigate possible risks for instability due to a rotational slip (see **54.2**) encompassing the shoreside works and any dredging of the seabed for berths on the water side of the site (see Figure 40).

Boreholes for jetties should be sited along the line of the berthing heads and mooring structures and along the line of the approach structures.

Where dredging is required for berths, navigable channels and borrow areas for hydraulic fill, the boreholes should be laid out on a grid pattern covering the areas to be dredged.

49.6 Depth of boreholes. Boreholes should be drilled to a depth safely exceeding the likely penetration of sheet piles and bearing piles. In order to obtain soil parameters to form the basis of slope stability analyses the depth of the boreholes should cover the likely maximum depth and lateral extent of slip surfaces (see Figure 40).

Boreholes should be drilled to a depth of twice the height of the ground supported by earth-retaining structures in order to investigate the foundation conditions and stability against a rotational shear slide (see Figure 41).

In order to obtain information on the stability and deformation of the foundations of gravity-type quay walls such as sheet pile cellular structures, or monoliths, the boreholes should be drilled to a depth below the base of the structure of 1.5 times the width of the structure or pile group supporting the structure (see Figure 41).

Boreholes drilled in rock formations should be taken to a depth sufficient to explore the thickness and characteristics of any weathered rock which could affect the behaviour of slopes and foundations. Where heavily loaded structures depend for their stability on the strength of fresh unweathered rock the boreholes should be drilled to a depth of at least 3 m into such material to prove its quality and continuity. Exploratory boreholes for anchorages in rock should be taken to the full depth proposed for the drilled anchor holes.

Dredging below the seabed may involve removal of soil from below the designed dredge level to comply with the requirements of a particular excavation technique and the possibility of future deeper dredging should be considered. Therefore boreholes in dredging areas should be extended below design dredge level until a stratum of known geological characteristics is encountered or to a depth of 5 m below design dredge level, whichever is the less.

49.7 Ground water investigations. The piezometric head of pore water in soils is a critical factor in the analysis of the stability of excavated slopes and earth-retaining structures. Fluctuations in tidal or seasonal levels in the waterway may cause corresponding fluctuations in piezometric head in the ground water. The effects on ground water levels of meteorological surges in sea levels should also be considered. It is therefore important to establish the relationships by means of simultaneous observations of waterway levels and ground water levels in piezometers, installed on the landward side at various distances back from the waterside face of a structure or slope. The observations should be made to cover periods of spring and neap tides and if possible they should cover periods of seasonal peak conditions in waterway levels. Observations of the salinity of the ground water at various positions back from the waterside face may indicate the relative influences of saline water and non-saline ground water on piezometric levels.

The possible existence of ground water under artesian or sub-artesian pressure within pervious soil layers confined by impervious strata should be investigated by observations in piezometers installed within the pervious layers.

Where ground water lowering schemes are proposed to enable the construction of dock basins or lock chambers to be undertaken in pumped-down excavations the ground water investigations should include measurements of the permeability of the soil in situ. Permeability tests can be made in the site investigation boreholes as described in BS 5930. Experience of North Sea platform site investigation should be useful for offshore permeability tests.

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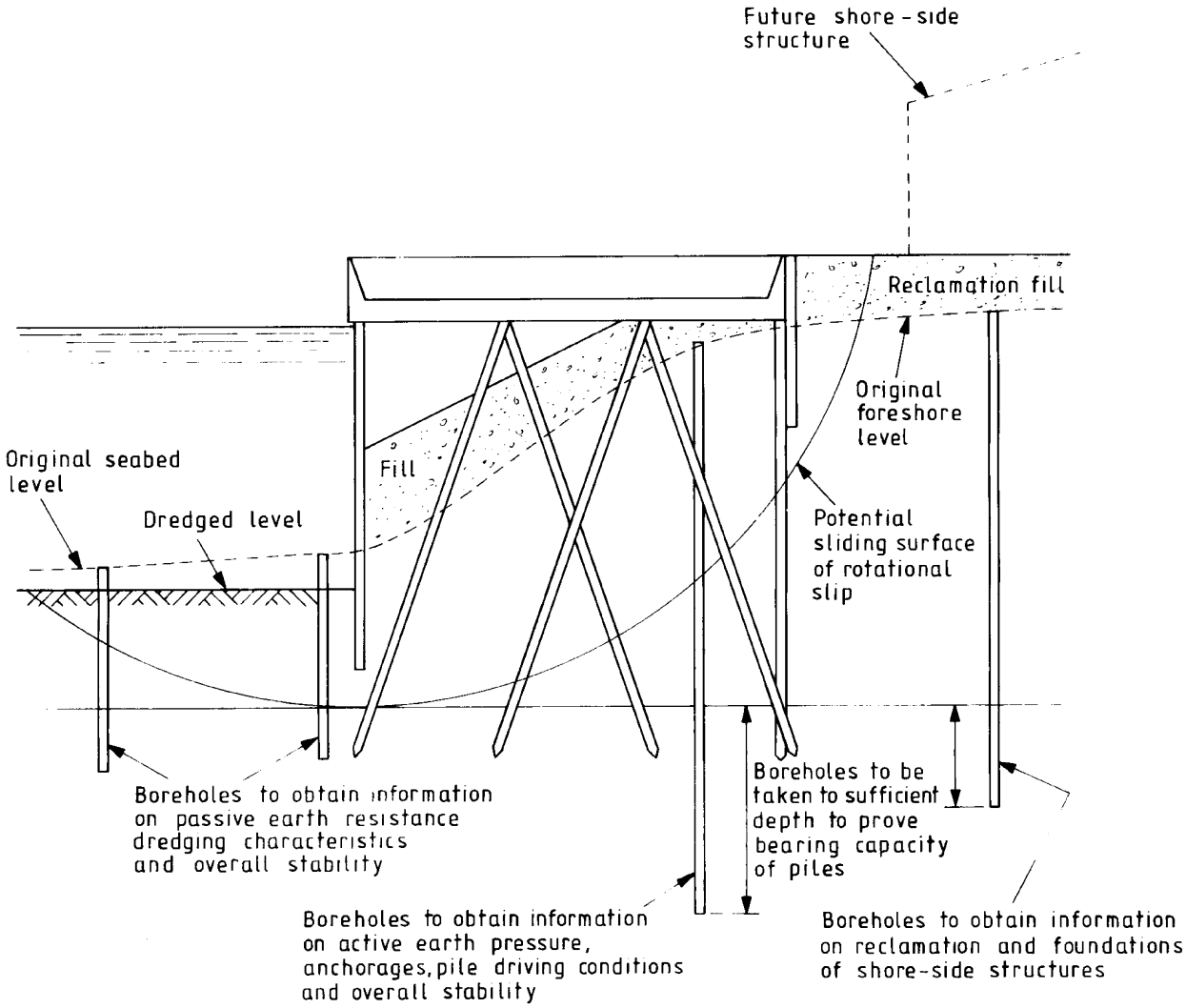

Figure 40 — Location and depth of boreholes for piled wharf

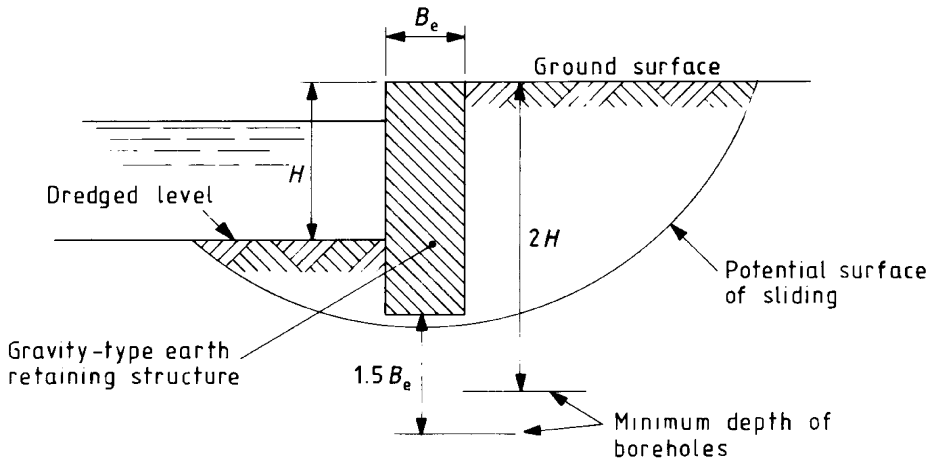

Figure 41 — Depth of boreholes in relation to retained height of soil and width of quay wall

49.8 Determination of earth pressure coefficient at rest. Where earth-retaining structures can be formed directly against the soil to be retained, e.g. by bentonite slurry techniques, without the construction process permitting any significant lateral deformation of the soil, consideration should be given to in situ measurements of the coefficient of earth pressure at rest. The conditions of earth pressure at rest are applicable to rigid unyielding types of earth-retaining structure as strutted reinforced concrete walls or mass gravity structures anchored to rock (see **51.3.1**). Methods of making these measurements are described elsewhere [28].

49.9 Detection of underground movements at depth. Where maritime works are to be constructed on sites suspected of having suffered previous instability it may be desirable to monitor any movements in the ground at various depths in advance of construction followed by similar observations during, and for as long as possible after, construction of the project. This form of monitoring also forms part of investigations of instability which may arise as a result of earthworks associated with shoreside structures. For further guidance see **5.1.3.5** of BS 6031:1981.

49.10 Geophysical surveys. The information obtainable from seismic refraction or reflection surveys is limited to delineation of the profile of the interface between successive strata having markedly differing seismic velocities. Information on the engineering characteristics of the strata is not obtained from the seismic observations and information on ground water conditions is also lacking. Geophysical surveying by seismic refraction or reflection techniques is therefore of limited application to the detailed design of maritime structures. However, such surveys can be used to establish the thickness of soils or weak weathered rocks overlying strong unweathered rock formations, thus enabling predictions to be made of the maximum penetration of piles in jetty structures. Delineation of the interface between weak overburden material and strong rock formations is also helpful to the estimation of quantities of rock requiring loosening by explosives in projects involving dredging berths and navigable channels.

Seismic surveying, and in particular the continuous seismic reflection profiling technique, has useful applications when considering the relative suitability of alternative routes for dredged channels in locations where rock dredging is expected.

In all cases it is essential to correlate the surveys by an adequate number of boreholes located to identify and confirm the levels of the various strata indicated on the seismic profiles.

Side-scan sonar surveys can be used to locate surface irregularities in the seabed such as sand ridges, ledges of rock, coral heads, and submerged wrecks. When supported by a suitable programme of seabed sampling or diving inspection, side-scan surveys can frequently be used to delineate the extent of the different types of seabed materials.

49.11 Field trials. Pile driving and loading tests may be desirable to confirm the feasibility of driving piles to the penetration required to develop the necessary resistance to compressive, uplift and lateral loading. Tests made in advance of the main construction programme can often prevent heavy costs being incurred later, due to delays and changes in design brought about by difficulties in piling operations.

The resistance of piles to various forms of loading is influenced to a great extent by the method used to install them and accordingly the preliminary trials should, as far as possible, use the same type of equipment as will be employed for the construction of the permanent works.

This can result in a costly operation for piling works over water if floating equipment has to be mobilized for pile loading tests. Consideration should therefore be given to conducting the preliminary trials on shore in situations where ground conditions can be found similar to the adjacent works over water.

The resistance of piles to lateral loads or the deflection of a pile under a given lateral load is dependent more on the stiffness of the pile than the soil conditions below ground level. Field trials should therefore be made with a pile of the same dimensions and stiffness as will be used for the permanent works.

Similarly, installation and pulling trials on ground anchors (see clause **53**) can be made to confirm the design capacity of anchorage systems for dock floors and tied-back retaining walls. The trials may include testing anchors to failure under pull-out load, or they may be limited to load testing to 1.5 to 2.0 times the working capacity of the anchors.

Where dock basins and lock chambers are to be constructed in dewatered excavations it may be advantageous to make trial excavations to assess the suitability of a particular system of ground water control and to confirm estimates of pumping rates made on the basis of field permeability tests (**49.7**).

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Trial dredging operations are not always feasible or desirable since the dredgeability of particular soils or rocks is dependent on the type of dredger used and the manner in which it is operated. However, trial dredging may be desirable as a means of making full scale experiments to determine the safe side slopes of dredged areas (see clause **14**).

49.12 Studies related to constructional materials. A site investigation may include a study of the durability and suitability in the particular maritime environment of various materials proposed for incorporation in the works. Requirements for typical constructional materials are given in section 7.

Studies of water quality, marine growth and environmental impact are discussed in detail in clause **13**. Chemical and bacteriological analyses should be made on the seawater and ground water, and on samples representative of the various types of soil in contact with buried structures and foundations. These analyses should include determination of the concentration of any substances potentially aggressive to these structures.

Samples of seawater taken for analysis should take account of possible variations in salinity and in the concentration of pollutants due to the effects of tides or the periodic efflux of water discharged from land sources. Variations in the temperature of the water may have a significant effect on the aggressive action on submerged structures at locations adjacent to discharge of heated water from electricity generating stations or factories.

Studies of the potential corrosion of steel structures (see clause **68**) should include measurements of the electrical resistivity of the soil and the presence or absence of sulphate-reducing bacteria (see **68.8**). The occurrence of agents accelerating corrosion, such as untreated sewage or trade waste effluents, should be investigated.

Where shoreside works are to be constructed in made ground the composition of any industrial waste materials present in the fill should be investigated by chemical analyses.

The site investigation should include observations of existing maritime structures in the locality with particular reference to attack by marine organisms (see **13.5.1** and **60.2**) on submerged structures such as piling, fenders and groynes; the extent of barnacle growth; and the durability (see **57.2**) of locally available stone in break-waters and slope protection. If works and/or materials are proposed for which there is no local precedent then expert advice, e.g. from a marine biologist, should be sought.

Sources of quarried stone for breakwaters can be investigated in the first instance by geological mapping supplemented by rotary core drilling to prove the quality and thickness of suitable rock strata. An assessment may be required of the size of stones which can be blasted from the face of rock quarries, particularly when large stone is required for armouring purposes (see clause **57**). It is not always possible to assess the available quantities of large stone by examination of rock exposures or cores from undeveloped quarry sites. Trial excavations using explosives to blast rock from a face may be needed to obtain information on the fragmentation characteristics of the rock.

Assessment of the suitability of seabed soils for hydraulic fill should take account of changes in the particle size distribution caused by dredging, transportation by pipeline or hopper barge and discharge on to the reclamation area. Undesirable fine soil particles in the form of silt and clay can be washed out of the excavated soil in the course of these operations, resulting in a general improvement in the characteristics of the dredged material for hydraulic fill purposes.

The environmental effects of dredging and reclamation should be considered [29]. Adverse effects of dredging include creation of turbidity (see **13.4**), removal or disturbance of flora and fauna, acceleration of beach erosion, extension of areas of salinity in adjacent land areas and increase in oxygen demand of the water due to suspended organic matter caused by disturbance of the seabed. Adverse effects of reclamation may include deposition of polluted material, blanketing of flora and fauna, creation of turbidity in waterways adjacent to disposal sites, pollution of beaches and ground water, objectionable odours in reclamation areas and adjacent to pipelines and despoliation of the landscape.

Beneficial effects of dredging may include removal of polluted materials, supply of construction aggregates, supply of material for beach nourishment and reoxygenation of sediments.

50 Properties of the ground

50.1 Average properties for preliminary design. It may be desirable to make preliminary designs before full information is available concerning the geotechnical properties of the ground and any imported materials. In such cases the data in Table 17 may be used but it should be noted that for flexible structures, such as sheet piled walls, deformation of the structure can lead to significant variations in earth pressures (see clause **51**).

50.2 Selection of parameters for working design

50.2.1 *General considerations.* Detailed examination should be made of the borehole records, soil samples and preliminary test data with the object of locating and defining the soil layers which are critical to the stability of the structure and, where necessary, to adopt special methods of sampling and testing in these layers to obtain parameters of the desired quality and relevance to the particular geotechnical problem. In the course of this detailed examination, consideration should be given to the geology and geomorphology of the site, including its stress history and the occurrence of layering or intercalations of different materials; the effects of disturbance caused by construction operations; the effects of dynamic loading during the service life of the structure; and the strain-dependent and time-dependent characteristics of the soil.

In the following subclauses consideration is given to the following groups of materials classified in accordance with their particle size distribution and stress history:

a) sands and gravels;

b) silts and fine silty sands;

c) normally-consolidated and lightly over-consolidated clays;

d) heavily over-consolidated clays;

e) rocks;

f) fill materials.

Guidance on the properties of the ground in relation to various structures may be found in BS 6031 and CP 2004. For coastal sites these codes should be read in conjunction with **50.2.2** to **50.2.7** which are particularly relevant to maritime structures.

50.2.2 *Sands and gravels.* Values of relative density and angle of shearing resistance, ϕ, can be obtained from empirical relationships between these parameters and the results of standard penetration tests or static cone penetration tests [30]. The latter tests are, however, usually restricted to investigations of land-based structures.

The shearing strength of dense sands and gravels are strain dependent showing a marked reduction from a high peak value at small strains to a relatively low value at large strains (see Figure 42). This should be considered in relation to the large deformations which are possible with flexible structures, such as sheet pile retaining walls, or berthing structures designed to sustain large deformations as a means of absorbing the kinetic energy of moving vessels.

The maximum angle of friction, δ_{max}, between the soil and an earth-retaining structure can be taken as $2\phi_r/3$. Mobilization of this maximum value requires movement between the structure and the soil. The amount mobilized, δ_m, for passive resistance may be taken from Table 17.

Table 17 — Mobilized angle of fiction

Type of structure	Relative soil density	δ_m/δ_{max}
Masonry walls with horizontal movement	Loose Dense	0 0.5
Light anchor walls	Loose Dense	0 0
Sheet pile walls freely embedded	Loose Dense	1.0 1.0
Sheet pile walls driven into rock	Loose Dense	0 0.5

Sands and gravels are highly permeable, therefore ground water will show rapid response to fluctuation in water levels of an adjacent waterway. On a falling tide this can result in outflow and erosion due to rapid drawdown of ground water from unprotected slopes. Sands and gravels are also susceptible to erosion due to flowing water or wave action. Erosion of the seabed adjacent to a quay wall can cause a reduction in passive resistance to pressure from the ground retained by the wall. Similarly, erosion around bearing piles can result in weakening of their resistance to lateral loads and reduction in vertical skin friction resistance.

The deformation of the soil beneath heavily loaded foundations is likely to be a factor more critical to their design than considerations of failure in shear under the superimposed loading. The deformation characteristics of sands and gravels can be obtained by correlation with the results of standard penetration or static cone penetration tests [30].

The skin frictional and end bearing resistance of piles can be calculated from the angle of shearing resistance of sands and gravels as obtained by field tests or by direct correlation with field tests, as described in **53.2.3**.

Both the deformation characteristics and resistance to shear of sands and gravels are affected by dynamic loading and in particular to large numbers of cyclic repetitions of load. These effects should be considered in relation to stresses induced by wave action, tidal fluctuations and, where appropriate, earthquakes.

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Sands and gravels may be found to be cemented to varying degrees by a saline or calcareous matrix which can impart an appreciable cohesive strength to the soil. However, the possibility of complete or partial loss of the cementicious value of the matrix should be considered. This loss may occur in tropical or subtropical saline soils when excavation, made under water or exposed to seepages of water, allows fresh or brackish water to reach soil deposits which previously existed under fully saturated saline conditions. The saline or calcareous matrix is then dissolved with complete or partial loss of cohesion.

50.2.3 *Silts and fine silty sands.* Where these soils are predominantly cohesive, shearing strengths can be obtained in terms of total stresses by means of field vane, cone penetrometer or pressuremeter tests. Undisturbed samples of silts and fine silty sands can be obtained by means of piston samplers. Consolidated drained triaxial compression tests can be made on these samples for the calculation of earth pressures and stability conditions, in terms of effective stresses. Cone penetrometer, and possibly some pressuremeter, tests should be regarded as giving only a qualitative measure of shear strength.

Silts and fine silty sands are susceptible to instability as a result of high pore pressures generated within permeable layers of fine sand confined by more impervious silt layers. Pore pressure variations can be caused by tidal waters, or water from landward sources under a high piezometric head, gaining access to the permeable layers. High pore pressures can also be generated by loads applied to the ground surface or by vibrations transmitted to the soils from embedded structures. High frequency vibrations transmitted to silts and fine silty sands can lead to liquefaction and complete loss of support to foundations or passive resistance to earth pressures. The possibility of large scale flow slides caused by pile driving vibrations or earthquakes should be considered in respect of shallow underwater slopes dredged in fine sands (see **54.2.8**).

Fine sands and silts are susceptible to erosion by flowing water and the consequent effects are as described above for sands and gravels (**50.2.2**). Piping and quicksands can also occur.

The deformation characteristics of fine sands and silts where loading is applied through foundations and the axial and lateral load resistance of piles embedded in these soils are obtained in the same manner as described in **50.2.2** for sands and gravels.

Some fine sands and silts, notably loess soils, may exist in a weakly cemented and desiccated state. When water is introduced to these soils, either by diversion of surface water or by inundation in reclamation works, the matrix is dissolved and the structure of the soil collapses with considerable subsidence of the ground surface.

50.2.4 *Normally- and lightly over-consolidated clays.* Normally-consolidated or lightly over-consolidated clays are those for which the undrained shear strength does not normally exceed 40 kN/m^2 and which have not been subjected to heavy overburden pressures during their geological history.

Undrained shear strengths of these clays can be obtained by field vane or pressuremeter tests or by undrained triaxial compression tests made in the laboratory on undisturbed samples from boreholes. These parameters should be used for considerations of the short term stability of earth-retaining structures and slopes under total stress conditions.

Normally-consolidated clays are sensitive to the effects of disturbance. The degree of disturbance depends on the geological history of the clays [31]. The effects should be considered in relation to construction activities associated with maritime structures. For example, driving bearing piles to support the deck of a wharf can result in loss of the undrained shear strength of the mass of clay surrounding the piles, with consequent reduction in the passive resistance on the face of any nearby sheet piles which may be forming an earth-retaining structure on the landward side of the wharf.

Vibrations from pile driving or earthquakes can cause collapse of slopes in normally-consolidated clays sensitive to disturbance.

The development of lateral pressures on earth-retaining structures and the stability conditions for these structures and for slopes should be calculated in terms of effective stresses. The appropriate shear strength parameters for normally-consolidated clays are obtained by means of consolidated drained triaxial compression tests made in the laboratory on undisturbed samples. Field tests are not appropriate to the determination of effective stress parameters since they measure strengths corresponding to the stress conditions at the level at which the test is made, whereas subsequent dredging for slopes and berths will reduce the total stress in the soil followed by volumetric expansion, absorption of water and softening of the clay. The rate at which the softening takes place will depend upon the permeability of the soil. It can be accelerated by the presence of drainage channels in the form of layers or laminations of silt and sand within the mass of the clay.

The stress-strain relationship for normally-consolidated clays is shown in Figure 43. The selected shear strength parameters should take account of the predicted or permissible deformations in structures such as flexible sheet pile walls and berthing dolphins and the predicted deformation of slopes (see clauses **51** and **54**). The residual shear strengths at large strains are applicable to shear or ancient slip surfaces where large displacements have already occurred.

The effective stress parameters of normally-consolidated clays may vary according to the direction of application of the deviator stress to the test specimen. This is due to the effect of discontinuities in the specimen or to differences in the ratio of vertical to horizontal stress in the soil in situ [31]. Also, in the triaxial compression tests the effects of pore pressure changes in the test specimen should be studied in relation to the predicted variation in piezometric levels behind the under-water and above-water slopes (see clause **54**). Repeated cycling of the deviator stress may be required in order to study the effects of tidal variations in piezometric levels or the effects of wave action on structures embedded in the clays.

50.2.5 *Over-consolidated clays.* Undrained shear strength parameters can be obtained from triaxial compression tests made on undisturbed samples of over-consolidated clays. These parameters can be used to determine the stability of foundations of structures and also the short term stability of underwater slopes in terms of total stress (see clause **54**).

The long term stability of slopes should be analysed in terms of effective stresses and the appropriate parameters should be obtained from consolidated drained triaxial compression tests on undisturbed samples of the clay. Over-consolidated clays show the same strain-dependent effects as normally-consolidated clays (see Figure 43) and the influence of these on the calculation of lateral pressures and slope stability is discussed in clauses **51** and **54**.

Over-consolidated clays are likely to have a fissured structure, therefore the orientation of the principal fissure system within the test specimen should be considered when assessing triaxial compression test results.

The fissured structure of over-consolidated clays also affects their mass permeability. This is significant when considering the shearing resistance of a mass of clay after excavation by dredging. Relief of overburden pressure will cause fissures to open allowing the ingress of water and softening of the mass. The effects on passive soil resistance are discussed in clause **51**. Some glacial clays may have a laminated structure with similar effects on mass permeability to those of a fissure system.

50.2.6 *Rocks.* In maritime structures the stability of rock masses may need to be considered in relation to excavations for the retaining walls of quays, lock chambers and docks. These are considerations of short term stability until the retaining structure is built against the rock face. Long term stability may need to be considered in the case of under-water and above-water slopes. Over the height attacked by waves, cavities and joints should be plugged and structural overhangs and re-entrant angles should be avoided.

The stability of slopes in rock formations is governed by the strength of the material within discontinuities such as fissures, laminations, fault planes, bedding planes and layers of weak weathered rock. Behaviour of the rock is likely to be different above, within, and below the band attacked by waves. Some modes of failure are shown in Figure 74, Figure 75 and Figure 76.

Considerations of lateral pressure due to rocks are normally concerned with pressures caused by fill in the form of crushed rock and the selection of appropriate parameters is discussed in **50.2.7**. However, the possibility of high lateral pressures should be considered where concrete retaining walls are cast against the excavated face of a rock with potential swelling characteristics.

50.2.7 *Fill materials.* The design parameters for materials placed as fill behind earth-retaining structures or used in reclamation areas should be considered in relation to their characteristics as fully remoulded soils. It is unlikely that either normally-consolidated or over-consolidated clays would be used as fill exerting pressure on a retaining wall. In particular, over-consolidated clays can exert very high pressures due to swelling of the clay lumps after excavation and exposure to the effects of air and water and they should not be used as backfill materials in this situation.

Cohesionless soils, i.e. sands and gravels, or crushed rocks are normally used as fill behind retaining walls and in reclamation areas where stable conditions capable of carrying surface loadings are required.

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Table 18 — Physical characteristics of soils and rocks

General description of soil	State of compaction or consolidation	Natural bulk density		Angle of shearing resistance in terms of effective stresses	
		Drained	Submerged	Active	Passive
		t/m^3	t/m^3	deg.	deg.
Gravels	Loose	1.6	1.0	35	35
	Medium dense			38	37
	Dense	1.8	1.1	41	39
	Very dense			44	41
Sands	Loose	1.65	1.0	30	30
	Medium dense			33	32
	Dense	1.85	1.15	36	33
	Very dense			39	34
Silts		1.6		24	
				25	
		1.8		26	
				27	
Clayey silts		1.7		21	
Silty clays	Normally-consolidated	1.5		15 to 18	
	Over-consolidated	2.0		15 to 18	
Glacial till				26 to 30	
Peat	Unloaded	1.1	0.1	0	
	After moderate loading	1.3	0.3	15	
Granite		2.50[a]			
Sandstone		2.20[a]			
Basalts and dolerites		1.75 to 2.75	1.10 to 1.60		
Shale		2.15 to 2.30	1.20 to 1.35		
Stiff to hard marl		1.90 to 2.30	1.00 to 1.35		
Limestone		2.70[a]			
Chalk		0.95 to 2.00	0.30 to 1.00		

[a] Measured in the solid, i.e. not crushed or broken.

The soil parameters to be selected for cohesionless soils depend on their manner of deposition. Where they are discharged from a pipeline or dumped to fall through water they will be in a loose state of deposition and parameters for density and shearing resistance obtained from the values given for loose conditions in Table 18 should be used.

Where drainage can take place from cohesionless soils placed above water-level by hydraulic fill methods or where these soils can be tipped and compacted above water-level, they will be in a medium dense to dense state depending on the degree of compaction given to the fill and the parameters should be selected accordingly. Consideration may be given to compacting loose materials above or below water-level using special techniques of deep compaction [32].

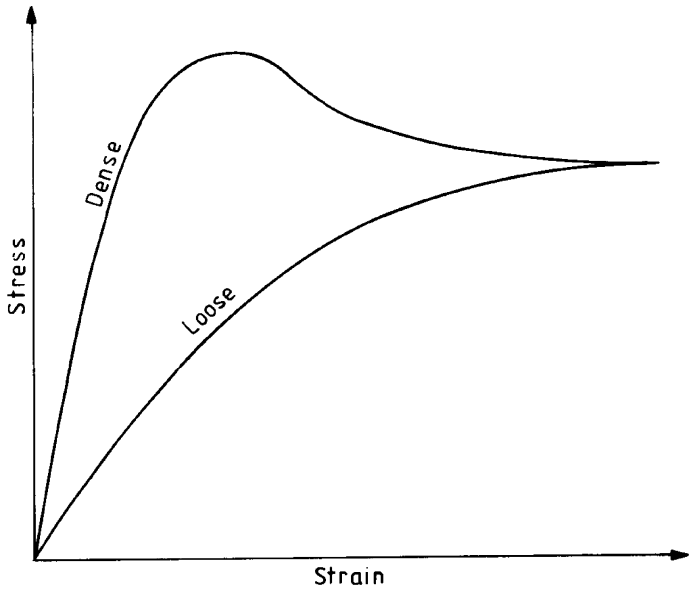

Figure 42 — Plane strain shear diagram for sand

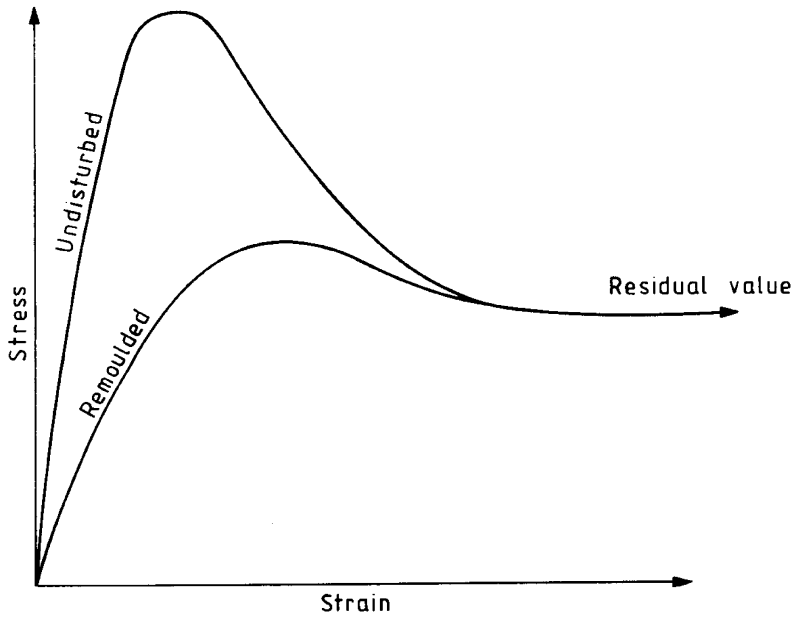

Figure 43 — Plane strain shear diagram for normally-consolidated clay

Where cohesionless soils are pumped immediately behind a retaining wall the fill above normal standing water-level may need to be treated as a fluid having a density equal to that of a fluid containing suspended solids. This condition will apply until such time as drainage takes place to dissipate pore water pressures in the pumped material.

Crushed rocks are treated as granular soils when selecting shear strength and deformation parameters. When tipped through water they are in a loose state of deposition and may undergo long term consolidation as a result of crushing and degradation of points of contact between rock fragments. High densities with a correspondingly high angle of shearing resistance can be obtained from crushed rocks compacted above water level.

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The effects of weathering on rocks used as fill has to be considered. Softening at the points of contact of rock fragments will result in a reduction of angle of shearing resistance. Complete degradation will result in the formation of a mass of fill of soil-like consistency, in which case the selected parameters will depend on whether the rock degrades to a cohesionless or cohesive soil or some intermediate type.

51 Sheet piled structures

51.1 General. Sheet piles capable of being driven into the ground while interlocked are frequently used to form earth-retaining structures in maritime works. Although timber, glass-reinforced plastics, corrugated asbestos cement and reinforced concrete are all used in sheet piling the most common material is steel. Guidance on the strength and durability of the various types of sheet piles is given in section 7.

Sheet pile structures can be divided into the following five groups:

a) anchored single-wall;

b) cantilevered single-wall;

c) double-wall;

d) cellular;

e) anchored double-wall or cellular.

The distribution of earth pressure and resistance on these structures (see **51.3**) depends on the type and flexibility of structure as well as the nature of the soil. This interdependence applies similarly, but more simply, to the remaining principal loadings, namely surcharge (see **51.4**) and hydrostatic pressure (see **51.5**). When considering the general stability of these structures the particular interaction of specific structures and their surrounding soil masses is again evident (see **51.6**).

51.2 Types of sheet pile structure

51.2.1 *Anchored single-wall.* Resistance to active pressure from the retained soil is provided by the bending strength of the piling which spans vertically between anchors at one or more levels and the distributed lateral resisting pressure on the lower embedded part of the piles where passive soil resistance is available (see Figure 44). Anchorages may be limited to a single level at or above MHWS because of the difficulty in installing anchors and walings below MLWS or at some inter-tidal level.

Single-wall sheet piling can be installed in a wide range of ground conditions but, because reliance is placed on passive soil resistance, this form of retaining wall can be used only in situations where adequate embedment of the piles is possible.

51.2.2 *Cantilevered single-wall.* This form of construction is limited to low retaining walls, generally with a retained height of soil less than 6 m, since resistance to active pressure of the retained soil is provided only by the passive resistance of the embedded portion of the piles (see Figure 45). The soil yields in mobilizing this resistance with consequent large deflection of the tops of the piles.

51.2.3 *Double-wall.* Double-wall sheet pile structures are used where the bending strength of a single wall is insufficient to resist the lateral earth pressures and where the depth of embedment is insufficient to provide the required stability by passive earth resistance alone (see Figure 46). Combined action of the two walls is obtained by connecting them by transverse sheet pile bulkheads or by tie rods and walings to contain the granular fill placed between them.

Lateral earth pressures are resisted partly by the action of gravity on the soil contained by the piling and partly by the passive soil resistance on the lower parts of the walls embedded in the natural ground [see Figure 46(a)]. To ensure adequate mobilization of gravitational forces in situations where passive soil resistance provides only a small proportion of the total resistance to sliding or overturning, the distance between the two rows of parallel sheet piles should not be less than 0.8 times the retained height of soil. Only a free-draining granular soil or crushed rock should be used to fill the space between the sheet piles and provision should be made for drainage as described in **51.5**. Any weak soil in the cells should be removed before placing the fill.

Double-wall structures have limitations in situations where little or no embedment of the piles is possible; for example on a bare rock site where the wall acts solely as a gravity structure [see Figure 46(b)]. In maritime structures it is unlikely, without underwater working, that ties and walings can be installed at a sufficiently low level to prevent significant bending deflection and spreading of the two rows of piles under the internal lateral pressure of the contained soil. Close-spaced transverse bulkheads acting as ties would require uneconomically large tonnages of sheet piling.

51.2.4 *Cellular.* The cellular sheet pile wall has economic advantages over the double parallel type for high retaining walls. This is because the steel acts in hoop tension, instead of in bending, to resist lateral earth pressures and the costs of supplying and installing walings, tie bolts and bulkheads are eliminated or greatly reduced (see Figure 47). When advantage is to be taken of using hoop tension, straight web piles with high interlock strength have to be used.

The cellular sheet pile wall can act as a free-standing gravity structure where little or no embedment of the piles is possible. Where embedment can be achieved the wall acts partly as a gravity structure and partly as a cantilever wall.

The structure can consist of circular cells connected by short circular arcs [see Figure 47(b)]. Where the retained height is large the tension in the interlocks or in the flat webs of the sheet piles may be too great to permit the use of a circular cell. In such cases flat transverse bulkheads are used connected by circular arcs [see Figure 47(c)]. Strong points, e.g. clover-leaf cells, should be provided to preclude progressive collapse.

Even so the flexibility of flat piles tends to limit their use for structures exceeding 25 m deep. However, stable walls of the same diaphragm cell concept have been built to 45 m depth using slurry tench concrete panels, in which case it would probably be necessary to provide tension connectors and/or to invert tension arches into compression.

In addition to their main function as quay walls, cellular sheet pile structures have useful applications for breakwaters or training walls in situations where large rocks or other suitable materials are not available to form embanked structures.

In all applications it is essential that the structures should be constructed on ground of good bearing capacity in order to avoid shear failure in the supporting soil. The cells should be filled with free-draining granular soil or crushed rock and any weak soil within the cells should be removed before placing the filling. Provision for drainage of the fill within the cells should be made as described in **51.5**.

51.2.5 *Anchored double-wall or cellular.* The resistance to overturning or sliding of double-wall and cellular sheet pile structures provided by base friction and gravity, with or without passive resistance of the soil below dredge level, can be augmented by anchoring the structure (see Figure 48). The anchorage forces may be high and substantial walings or capping structures should be provided to distribute the horizontal or inclined anchor reactions on to the profiled or curved faces of the sheet piles.

51.3 Distribution of lateral earth pressure and earth resistance

51.3.1 *Cohesionless soils.* Sheet pile structures are flexible structures such that appreciable deflections accompanied by strains in the soil occur when they perform as earth-retaining structures. The magnitude of these strains affects the shearing resistance of cohesionless soils as described in **50.2.2**, and ϕ_r should be taken as the value corresponding to active pressure conditions for the calculation of the pressure on the landward face of the structure. Cohesionless soils do not normally show time-dependent effects.

The deflections of a flexible earth-retaining structure are frequently such that the active earth pressure and passive earth resistance can be taken as increasing in linear proportion to the depth below ground surface or dredge level. This linear or Coulomb distribution is applicable in all cases to loose cohesionless soils, but not in all cases to dense cohesionless soils.

For linear distribution in uniform soil the active force on the landward side of the wall is given by:

$$P_A = \tfrac{1}{2} K_A \gamma H_r^2$$

where

P_A is the total active force per unit length of wall;

K_A is the coefficient of active earth pressure;

γ is the effective density of the soil;

H_r is the height of the retained soil.

The effective density should be taken as the drained bulk density γ_d when the retained soil is wholly above the water table and as the submerged bulk density γ_s when the retained soil is wholly below the water table. For intermediate cases the active force should be calculated by summation of the pressure diagram taking account of the change in density at the water table.

Values of K_A should be read from Table 19 for appropriate values of ϕ and the angle of wall friction δ_{max}. Where necessary, the inclination of a sloping ground surface behind the wall should be taken into account in obtaining the value of K_A.

Values of γ_d and γ_s should be obtained from laboratory tests or read directly from Table 18.

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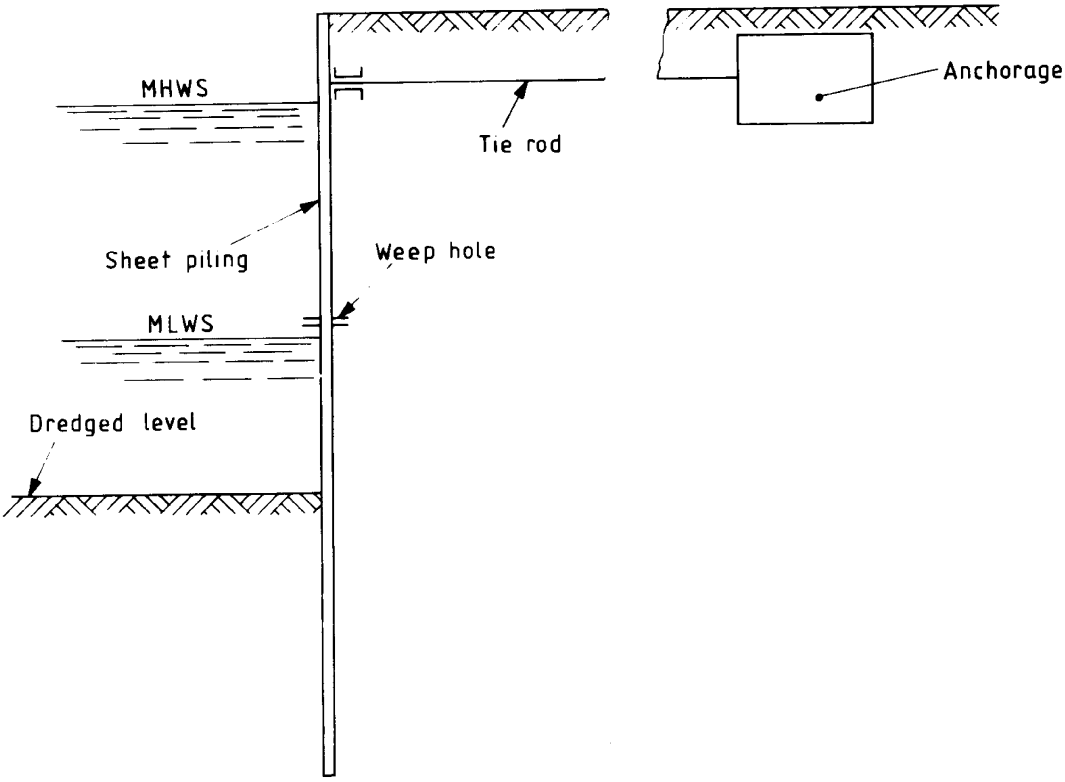

Figure 44 — Anchored single-wall sheet pile structure

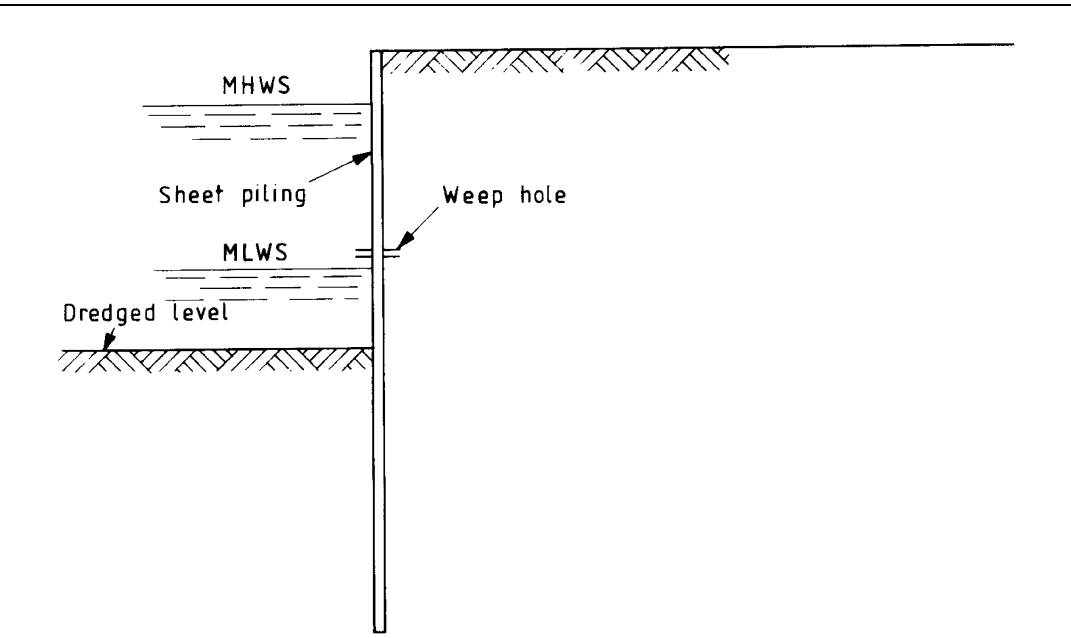

Figure 45 — Cantilevered single-wall sheet pile structure

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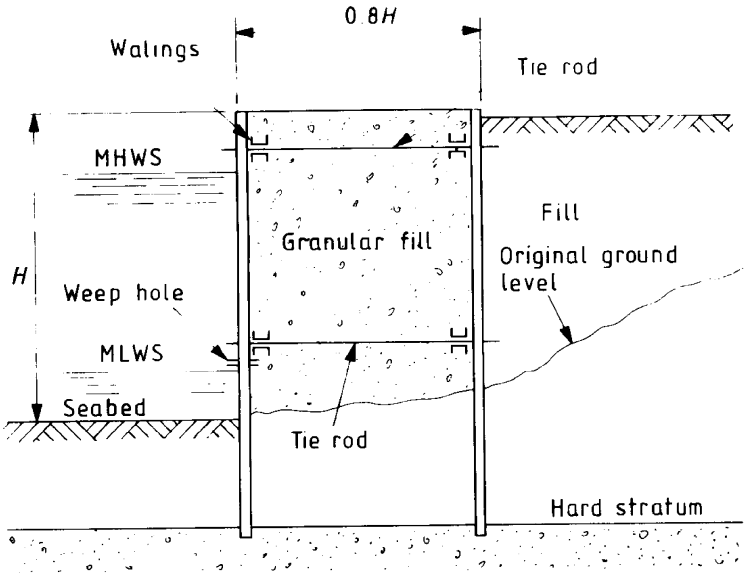
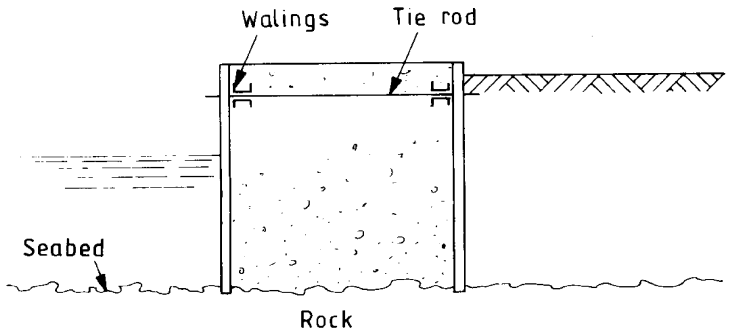


(a) Sheet piles driven into soil below seabed



(b) Sheet piles terminated on rock at seabed

Figure 46 — Double-wall sheet pile structures

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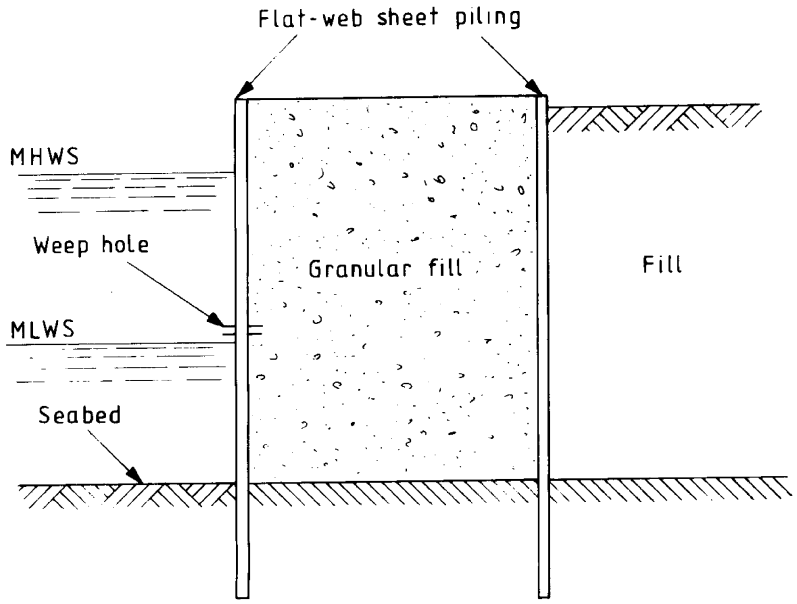
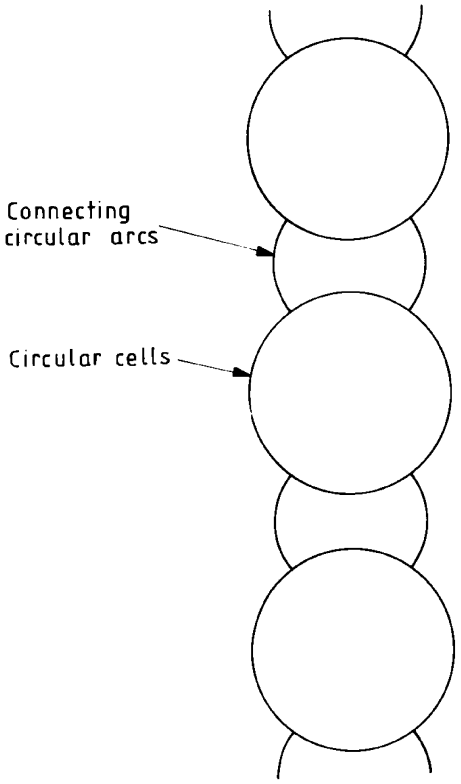
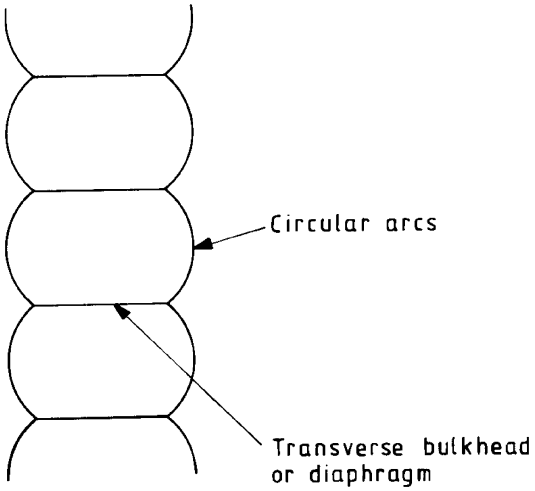


(a) Cross section of cell where sheet piles are driven into soil below seabed



(b) Plan of circular cells



(c) Plan of diaphragm cells

Figure 47 — Cellular sheet pile structures

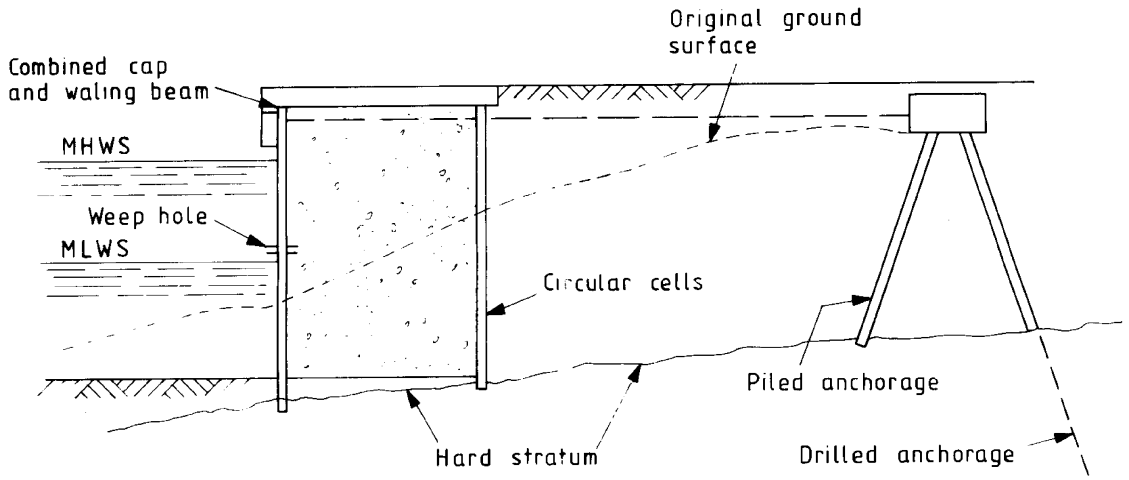


Figure 48 — Anchored cellular sheet pile structure

In the case of a cantilevered single-wall structure where movement occurs by outward rotation about an apparent point of fixity near the base, it can be considered that equal horizontal strain occurs at every point above the base. Hence the pressure distribution in a dense cohesionless soil can be taken as increasing linearly with depth after very small movements have occurred and the above expression may be used in determining the total force (see Figure 49).

In the case of an anchored single-wall structure where movement is permitted to take place by outward displacement of the toe about a pivot point at anchorage level, then non-uniform horizontal strains occur within the wedge of dense cohesionless soil behind the wall. Initially, the lower part of the wedge attains the fully active state before the upper part of the wedge because the latter is restrained by the tension in the anchor so that forward movement is insufficient to develop active pressure conditions.

If the resultant lateral force causes yielding of the anchor then the active pressure state will be developed over the full depth of the wedge, giving a linear distribution similar to that shown in Figure 49. Where no yielding of the anchor occurs the arched pressure distribution will remain non-linear as shown in Figure 50.

The total lateral force imposed by the wedge of soil remains approximately constant during the process of initial arching followed by yielding and redistribution of pressure but the initially high position of the resultant lateral force should be considered in relation to the calculation of the anchorage force and the stability of the wall against overturning.

Arching and the development of high lateral pressures in the upper part of the soil wedge should also be considered in cases where cohesionless soil fill is compacted above standing water level and/or where surcharge is imposed on the ground surface behind the wall (see **51.4**). It is considered that arching effects are nullified when the yield of the anchorage system is 0.1 % of the height of a dredged-type wall. This order of movement would normally take place in an anchored sheet pile structure.

Similar arching conditions should also be considered in relation to the lateral pressure distribution on gravity-type retaining walls (see Figure 46 and Figure 47). Results of limited full scale tests of a dense uniform medium grained dry sand show that a total translational wall movement of 0.5 % of the wall height was required to reduce the level of the pressure resultant from 0.45 times the height of the wall to the linear distribution level of 0.33 times the wall height. At that stage of movement a slip plane is developed in the surface of the compacted sand backfill.

Arching conditions should be taken into account in relation to the sequence of construction of an anchored sheet piled retaining wall. When backfilling is completed before the soil in front of the wall is dredged away, the movement of the wall due to pressure from the small retained height of soil may be insufficient to develop active pressure conditions. As dredging takes place the wall will yield and the pressure distribution will change from arched conditions at the upper level to the final assumed linear active condition. The initial and final stages are shown in Figure 51.

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If, however, the soil is dredged away before placing any fill behind the wall, a linear active pressure distribution will develop over the depth within the existing ground after completion of dredging. When backfill is placed and the part above ground water level is compacted, the additional yielding of the wall and anchorages may not be sufficient to develop active earth pressures from the filling at the higher levels. In this case provision should be made for lateral pressures from the upper part of the filling at a state intermediate between the active and at-rest condition, depending on the expected forward movement of the wall as the filling is placed (see Figure 52).

Where sheet piles are toed into rock, movement of the sheet piles will be prevented and the distribution of active pressures will be modified. At the toe the pressures will correspond to at-rest conditions.

For these conditions the effective force may be obtained by substituting the coefficient of at-rest pressure, K_0, for the active pressure coefficient, K_A, in the expression for P_A given in this subclause, where K_0 may be taken from the following:

a) Loose sand: $K_0 = 0.40$;

b) Dense sand: $K_0 = 0.80$.

Mobilization of the full passive resistance of a cohesionless soil requires a greater forward movement than is required for the development of active earth pressure. Model tests show that an outward movement of a wall at dredged level of 5 % and 0.5 % of the wall height is required for mobilization of half the passive resistance in loose and dense sand respectively. The movement required to mobilize the ultimate passive resistance is 10 % to 20 % of the depth of embedment for loose sands and 2 % to 4 % of this depth for dense sands. In practice the movements of a sheet piled structure are unlikely to be such as to develop the ultimate passive resistance and a conservative approach to the calculation of available resistance should be allowed.

For the normal order of movement permitted below dredged level the distribution of passive resistance can be assumed to be linear. The total ultimate resistance is given by:

$$P_p = \tfrac{1}{2} K_p \gamma_s d_e^2$$

where

P_p is the total passive resistance per unit length of wall;

K_p is the coefficient of passive earth resistance;

γ_s is the submerged bulk density of the soil;

d_e is the effective depth of pile penetration in front of the wall after allowing for overdredging and/or scour.

Typical values of K_p for appropriate values of ϕ_r and δ_m should be obtained from Table 20. Values for γ_s should be obtained from laboratory tests or from Table 18.

The passive resistance indicated by this expression will not be mobilized without movements which would be unacceptable in a permanent structure. It is common practice therefore to assume an effective resistance of approximately half that given by the expression.

When sheet piles are driven sufficiently deeply to achieve fixed earth support conditions the effective depths may be diminished by the effect of reverse curvature in the piles. In the case of cantilevered walls, passive resistance can be generated on the landward side of the wall at the toe (see Figure 49).

When sheet piles are toed into rock and movement at the toe is thereby prevented, the passive resistance in the soil immediately above rock level will correspond to at-rest conditions and the coefficient K_0 should be substituted for K_p in the expression for P_p.

51.3.2 *Normally-consolidated and lightly over-consolidated clays.* The forward movement of 0.5 % of the wall height required to mobilize the fully active shearing resistance of a clay soil is likely to take place in a sheet pile structure. Therefore lateral earth pressures should be calculated in terms of effective stresses on the assumption of zero effective cohesion and ϕ' being taken as the value representative of active conditions. The pressure distribution may be taken as linear as shown in Figure 49, and the total active force may be calculated from the expression for P_A given in **51.3.1**.

For calculating passive earth resistance the short term undrained conditions are likely to be more critical than the long term drained state. Therefore the passive resistance should be calculated on the basis of the peak undrained cohesion, c_u, and in terms of total stresses. The distribution of nett passive resistance will be uniform as shown in Figure 53.

In the long term, the positive pore water pressures induced by the shear stresses will dissipate leading to consolidation and strengthening of the soil, accompanied by wall movement. However, a check calculation of passive earth resistance should be made for long term conditions in which the effective cohesion c' should be taken as zero and the effective angle of shearing resistance, ϕ', should be the drained value for passive conditions.

Alternatively, Bell's theory may be used to obtain the total active and passive forces as follows (see Figure 53):

$$P_A = \tfrac{1}{2}\,\gamma\,(H_R + d_p)^2 - 2\,c_u\,(H_R + d_p)$$

$$P_P = \tfrac{1}{2}\,\gamma\,d_p^{\,2} + 2\,c_u\,d_p$$

where

c_u is the undrained shear strength of the clay, based on the weakest 5 % of samples;

H_R is the retained height of the structure;

d_p is the pile penetration.

A tension crack should not be taken into account except when considering hydrostatic pressure at the ground surface (see **51.5**). A cantilevered wall will fail before H_R reaches the theoretical maximum value of $4c_u/\gamma$ while an anchored wall will also fail with a small increase in H_R. Causes of failure are absence of, or inadequate, passive pressure. If the clay contains water-bearing horizontal joints or sand lenses then uplift pressures can develop and the submerged density γ_s should be used.

51.3.3 *Overconsolidated clays.* Dredging the soil away from in front of a sheet piled wall embedded in an over-consolidated clay will cause the wall to deflect forwards. The clay behind the wall will swell with the development of high negative pore pressures in the soil. Initially this will give highly stable conditions in terms of effective stresses but, with time, water will be drawn into the zone of swelling causing the clay to soften. The rate of softening will depend on the mass permeability of the soil (see **50.2.5**).

Similar swelling and softening of an over-consolidated clay will occur in front of the sheet pile wall as a result of a reduction of overburden pressure consequent on excavating down to final dredged level. Therefore active pressures and passive earth resistance should be calculated for long term conditions in terms of effective stresses. The effective cohesion c' should be taken as zero and the value of ϕ' should be representative of fully drained conditions.

Linear pressure distribution may be assumed and the active and passive forces may be calculated from the expressions for P_A and P_p given in **51.3.1**

Because of the high swelling pressures caused when an over-consolidated clay is allowed to soften, such material should not be used as filling behind a retaining structure.

51.3.4 *Layered soils.* Lateral soil pressures at any level are calculated by factoring the overburden pressure at that level by the soil pressure coefficient relevant to the soil stratum. Thus changes in the lateral soil pressure diagram will occur at interfaces between different strata. The pressures will also be affected by the presence of water in one or more of the strata.

51.4 Effects of surcharge

51.4.1 *Single-wall structures.* Uniform surcharge applied to the soil surface behind an earth-retaining structure can be assumed to act as an effective raising in height of the soil behind the wall. Consideration should also be given to the lateral pressure induced by surcharge applied in the form of line loading and point loading.

Where structures behind a retaining wall are supported by piles driven below the potential surfaces of sliding, pressures from the imposed loading on the piles will not be transmitted to the retaining wall, but consideration should be given to earth pressures caused by soil displacement and to the effects of pore pressures in the soil retained by and beneath the wall induced by soil displacement, as the piles are driven [33].

51.4.2 *Double-wall and cellular structures.* Double-wall and cellular sheet pile structures can be provided with a reinforced concrete deck capable of carrying heavy imposed loading. Where the load on the deck structures is carried by the sheet piles, internal lateral pressures from the fill material due to the surcharge loading on the deck need not be considered as acting on the retaining structure.

Where the sheet piles are terminated in a cohesive soil there will be long term settlement of the structure under the deck loading. Relative vertical movement between the structure and the retained soil will have the effect of reducing the wall friction on the active pressure side. The increase in wall friction on the passive side should not be taken into consideration for design purposes.

In cases where it is necessary to drive bearing piles within the cells to support heavy deck loading, the effects of soil displacement and pore pressure development in the natural soil within and beneath the cells should be considered. Where necessary this soil should be removed before driving the bearing piles.

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Table 19 — Values of K_A, the coefficient for active earth pressure

Angle of wall friction, δ	Angle of shearing resistance, ϕ_r (degrees)										
	15	18	21	24	27	30	33	36	39	42	45
	Coefficient for active earth pressure, K_A										
deg.											
0	0.589	0.528	0.472	0.422	0.376	0.333	0.295	0.260	0.228		
10	0.533	0.480	0.431	0.387	0.346	0.308	0.274	0.243	0.214		
12		0.474	0.426	0.382	0.342	0.305	0.271	0.240	0.212		
14			0.421	0.378	0.339	0.303	0.269	0.239	0.210		
16				0.375	0.336	0.300	0.267	0.237	0.209		
18					0.334	0.299	0.266	0.235	0.209		
20						0.297	0.265	0.235	0.208		
22							0.264	0.235	0.208		
24								0.235	0.208		
26									0.208		
28											
30											
	Silty clays and clayey silts			Sandy silts and silts		Sands and gravels					

NOTE The above values assume a horizontal ground surface to the retained soil.

Table 20 — Values of K_p, the coefficient for passive earth resistance

Angle of wall friction, δ	Angle of shearing resistance, ϕ_r (degrees)										
	15	18	21	24	27	30	33	36	39	42	45
	Coefficient for passive earth pressure, K_p										
deg.											
0	1.70	1.89	2.11	2.37	2.66	3.00	3.40	3.86	4.40		
$\phi/3$											
$\phi/2$	1.98	2.32	2.73	3.26	3.92	4.78	5.91	7.44	9.53		
$2\phi/3$											
	Silty clays and clayey silts			Sandy silts and silts		Sands and gravels					

NOTE The above values assume a horizontal surface to the soil providing passive resistance.

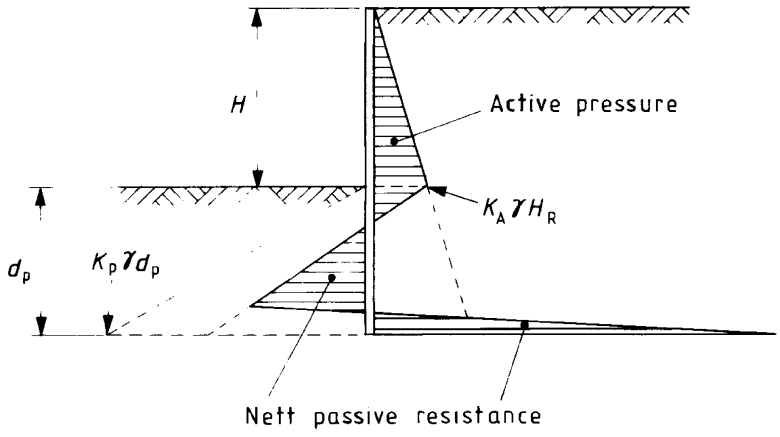

Figure 49 — Distribution of earth pressure and earth resistance on cantilevered single-wall sheet pile structure

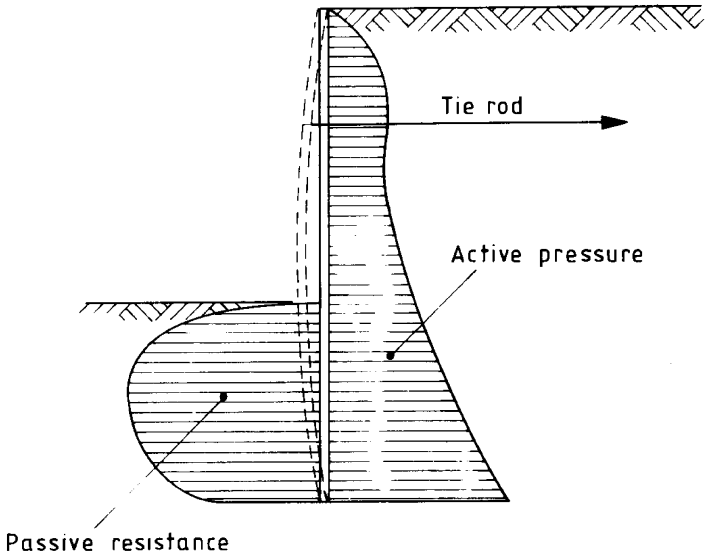

Figure 50 — Distribution of earth pressure and earth resistance on anchored single-wall sheet pile structure

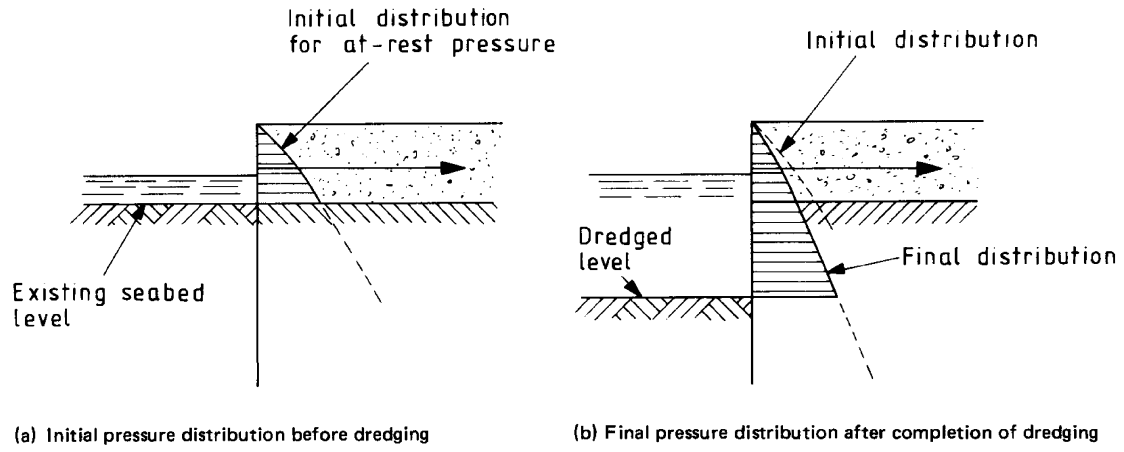

Figure 51 — Active pressure distribution on anchored single-wall structure where filling is placed before dredging

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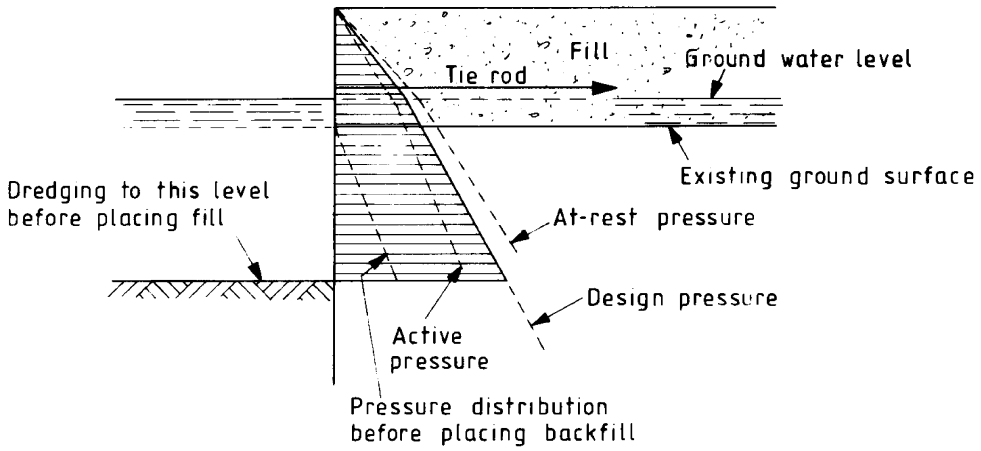

Figure 52 — Active pressure distribution on anchored single-wall structure where filling is placed after dredging

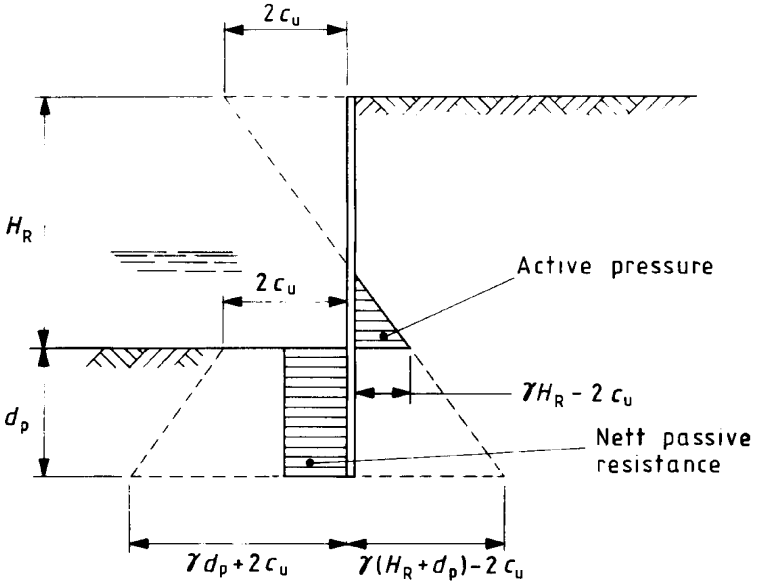

Figure 53 — Distribution of active pressure and passive resistance for total stress conditions in normally and lightly over-consolidated clay

Granular soil used to fill the cells should not be placed until the bearing piles have been driven. Bored and cast-in-place piles may be installed after completion of filling but care may be needed to avoid the development of excessive pore pressures in any layered or laminated clays below the base of the structure [34].

Superimposed loading on the deck will increase the resistance of the structure to sliding and overturning but the need to assume zero live loading in stability calculations should be considered.

51.5 Hydrostatic pressure distribution

51.5.1 *Single-wall structures*

51.5.1.1 *General.* Tidal variations will cause a differential hydrostatic pressure between the waterside face and the landward face of an earth-retaining structure. Fluctuations in the level of the ground water (GWL) should normally be less than the tidal variation, depending on the type and efficiency of the drainage measures provided in the wall, the permeability of the soil retained by and beneath the wall and the flow of surface or subsoil water from landward sources.

For a structure which retains a permeable soil, but where the sheet piles are driven into an impermeable soil acting as a cut-off to flow below dredged level, the cases described in **51.5.1.2** to **51.5.1.6** should be considered.

51.5.1.2 *Minor non-tidal water level variations (weephole drainage provided).* In non-tidal waterways where variations are due to minor seasonal fluctuations the differential water pressure from the landward to the waterside face may be taken as 0.5 m as shown in Figure 54(a).

51.5.1.3 *High flood flows in non-tidal rivers (weephole drainage provided).* Where a rapid fall in level of the water-way occurs at times of recession of floods, the differential water pressure should be taken as equal to the maximum predicted fall in water level over a 24 h period [see Figure 54(b)]. The maximum water level in the waterway from which the predicted fall takes place should be considered in relation to the available statistics of flood flows and to the most unfavourable effects of the differential water pressure on the risks of failure of the structure due to the combined effects of earth and water pressure.

51.5.1.4 *Large tidal variations (no drainage provided).* The differential water pressure may be taken as that due to a ground water level at mean tide level and a low tide level in the waterway ranging from MLWS, for normal cases, to an assumed water level down to extreme low water ELW (see clause **37**) when it is necessary to assess the risks of failure of the structure due to the combined effects of earth pressure and water pressure conditions for extreme low tides [see Figure 54(c)].

51.5.1.5 *Large tidal variations (flap valve drainage provided).* The differential water pressure may be considered as corresponding to a ground water level at 0.3 m above the invert of the flap valve, i.e. the head required to operate the valve, and a level in the waterway at MLWS, for normal cases, or at some intermediate level down to ELW (see clause **37**) when the risks due to the effects of combined earth pressure and water pressures due to extreme low tides need to be considered [see Figure 54(d)].

51.5.1.6 *Embanked soil behind retaining structure (drainage provided).* Where the surface of the soil behind the retaining structures is sloped back and the flow from landward sources is horizontal, the effects of a sloping ground water profile as shown in Figure 55 should be considered in relation to the effective weight of the wedge of soil behind the structure or within the zone of any anchorages (see clause **53**).

51.5.1.7 *Special considerations.* In all the cases described in **51.5.1.2** to **51.5.1.6**, the assumed ground water level is taken to be that due to flow in a homogeneous permeable soil. Special consideration should be given to hydrostatic pressures resulting from a strong subsoil water flow from a landward source. The effects of excess water pressures in permeable layers within a layered or laminated soil or rock formation should also be considered.

Where the retaining structure is backed by clay, drying and shrinkage of the clay above ground water level may cause a tension crack to form. Consideration should be given to water pressure resulting from surface water finding its way into such a crack (see Figure 56). The depth of the tension crack may be calculated from:

$$d_t = 2\, c_u/\gamma_s$$

where

d_t is the depth of the tension crack;

c_u is the undrained shear strength;

γ_s is the bulk density of the saturated clay.

In cases where permeable soil extends below the toe of the sheet piles, thus allowing seepage to take place below the piles, the simplified hydrostatic pressure distribution for flow from the landward to the waterside is shown in Figure 57. The effects of seepage forces causing an increase in active earth pressure and a decrease in passive earth resistance should be considered.

51.5.2 *Double-wall and cellular structures.* It is essential to use free-draining material as filling within the sheet piles of double-wall or cellular structures. Weephole or flap valve drainage should be provided in the walls. The water level within the structure may then be taken as corresponding to the assumed ground water level for **51.5.1.2**, **51.5.1.3**, **51.5.1.4**, **51.5.1.5** or **51.5.1.6**.

If hydraulic filling operations are used to place sand in the cells, the rate of pumping the fill may exceed the drainage capacity of the weepholes or flap valves, in which case, as a temporary condition, the water level within the cells should be taken as the level of the tops of the cells or at any lower level at which water is permitted to overflow freely.

Where double-wall or cellular structures are employed as breakwaters or as cofferdams subjected to wave action on the waterside face, the tops of the cells should be covered to prevent a rise in internal water level due to overtopping by waves.

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51.6 Stability of sheet pile structures

51.6.1 *Anchored single-wall structures.* The distribution of active earth pressure and passive earth resistance on a rigid retaining wall is shown in Figure 58(a). As stated in **51.3.1**, yielding of a flexible wall will modify the pressure distribution as shown in Figure 58(b) and Figure 58(c). The pressure distribution shown in Figure 58(a) represents a condition of free-earth support where the rigid anchored wall yields by forward translation. The effects of flexibility in causing arching of the soil are shown in Figure 58(b). Where the sheet piles are driven deeply [see Figure 58(c)] fixity occurs in the lower portions of the piles giving a reversal of curvature in the piling and a reduction in the embedded depth over which the passive resistance is operative.

For the purposes of calculating the total active forces, P_A, the active pressure distribution in terms of effective stresses should be assumed to be linear as shown in Figure 58(a). However, for the purpose of determining the stability of the wall against overturning and for calculating anchorage forces and bending moments on the sheet piling, the flexibility of the structure should be taken into account. The effects of flexibility on increasing the anchorage forces and raising the level of the resultant passive resistance should be considered as shown in Figure 58(b) and Figure 58(c).

Water pressures should be considered separately and added to the earth pressure for the purpose of obtaining the total lateral force on the structure.

Methods of calculating anchorage forces and bending moments and shears in the structure, taking account of flexibility, are to be found in subsequent Parts of this code.

The overall stability against sliding of the structure and the mass of earth retained by and beneath the structure should be considered. Sliding may be rotational, planar, or on a composite rupture surface. Methods of calculating the stability against sliding of earth masses in any of these failure modes are described in BS 6031.

For both cohesionless and cohesive soils, the long term considerations of stability are likely to be critical and the analyses should be made in terms of effective stresses under equilibrium conditions of pore water pressure. In cohesive soils the effects of soil shrinkage causing tension cracks within and up-slope of the anchorage zone should be considered. The effects of scour and dredging below the nominal design depth should be considered in relation to the depth of embedment required to mobilize passive resistance. It may be necessary to specify a maximum depth for overdredging. In cohesive soils, removal of overburden by dredging will cause swelling and softening of an over-consolidated clay which should be taken into account in the selection of soil parameters.

When considering the stability of sliding of the structure and earth masses, the mooring forces and surcharge loads on the structure should be included where these act as disturbing forces. The effects of earthquakes on stability may need to be considered in areas subject to seismic activity.

The weight of soil encompassed by the anchorage zone of the structure should be included in the disturbing forces in relation to the length and inclination of the anchors (see Figure 59).

51.6.2 *Cantilevered single-wall structures.* The distribution of active earth pressure and passive earth resistance on a cantilevered sheet pile wall is shown in Figure 60(a). The sheet piles have to be driven to an embedment sufficient to obtain fixed earth support conditions at the toe. For the purpose of calculation, the wall should be assumed to be rigid. The pressure distribution can be simplified as shown in Figure 60(b) when the force represented by the area CDE can be replaced by a single horizontal force F_3 acting at the point A, and the line AB is assumed to be horizontal. The value of the force F_3 is such that the algebraic sum of the forces F_1, F_2 and F_3 is zero.

Methods of calculating bending moments and shears in the sheet piles are to be found in subsequent Parts of this code.

The overall stability against sliding of the wall and the surrounding earth masses should be considered as described in **51.6.1**.

51.6.3 *Double-wall structures.* The effects of ties or transverse bulkhead walls connecting the two parallel rows of sheet piles is to form a gravity structure in which the resistance to forward sliding or overturning is provided by the passive resistance of the soil on the waterside face of the structure, by the frictional force beneath the base of the material filling the cells, and by the gravitational force of the contained material (see Figure 61).

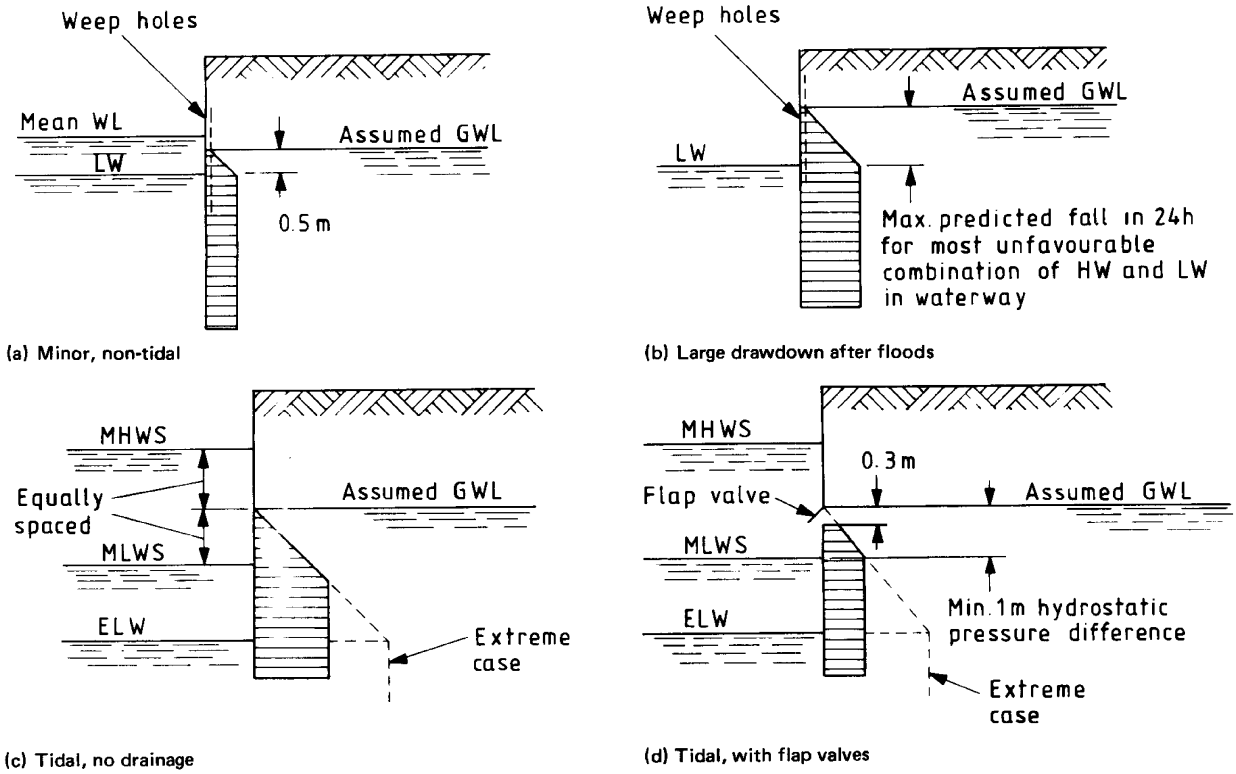

Figure 54 — Hydrostatic pressure distribution on waterfront structures where soil is retained to full height of structure

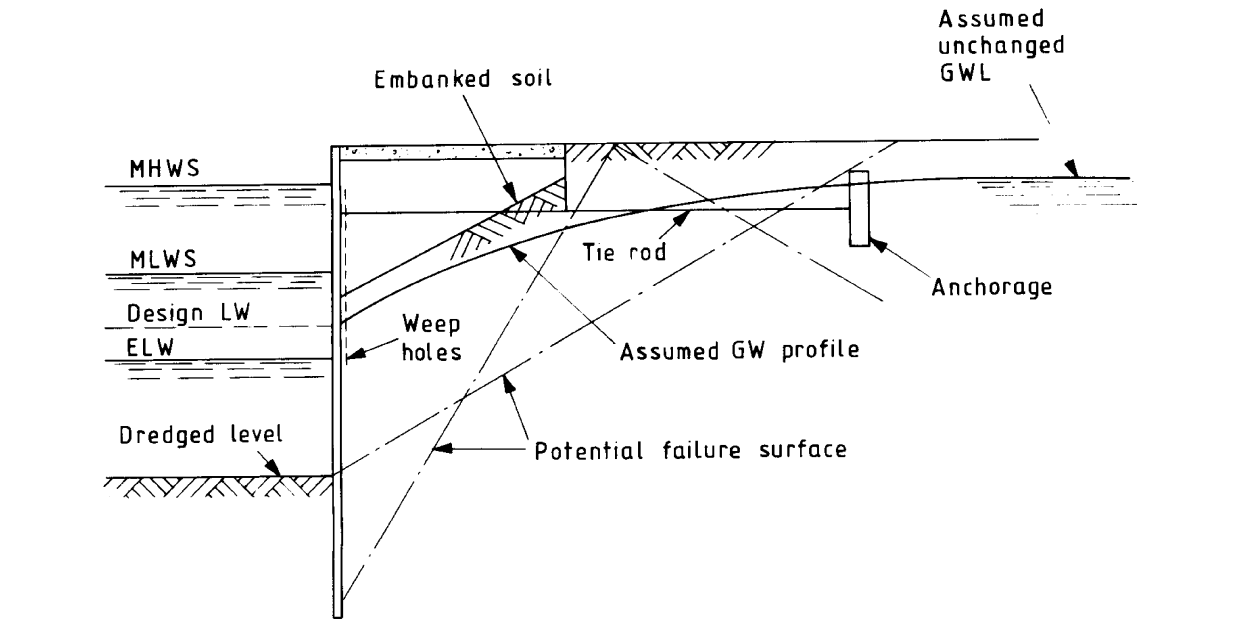

Figure 55 — Hydrostatic pressure distribution on waterfront structure where the soil is embanked behind the structure

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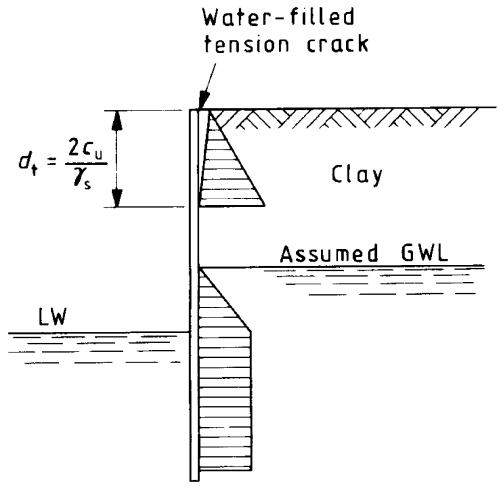

Figure 56 — Hydrostatic pressure behind waterfront structure backed by clay

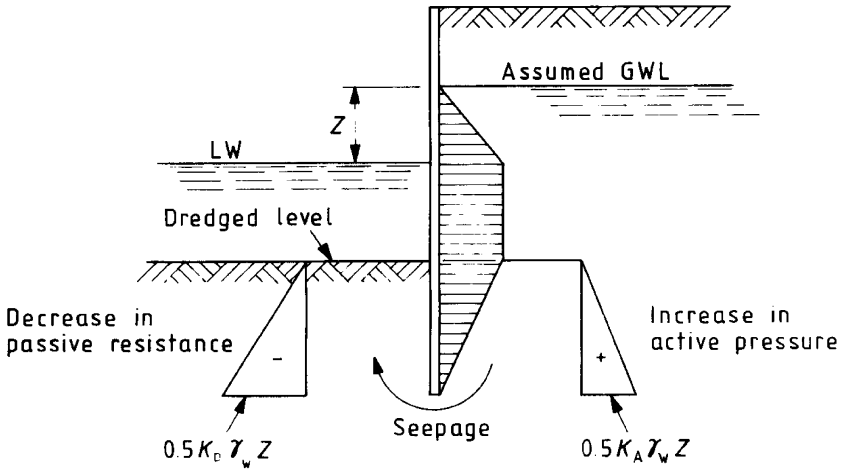

Figure 57 — Effects on hydrostatic and soil pressure distribution where seepage takes place beneath retaining structure

Because the material filling the cells is confined and the sheet piles are restrained from outward movement towards the waterside and landward side of the wall, the sheet piles should be designed to withstand an internal lateral pressure corresponding to at-rest conditions and the coefficient K_0 for earth pressure at rest should be used instead of the active pressure coefficient K_A in the expression for P_A given in **51.3.1**. Drainage should be provided in both walls so that the water pressure conditions will correspond to those described in **51.5.2**.

Where the double-wall sheet pile structure is founded on soil such that the lateral earth pressures are resisted partly by passive resistance of the soil below dredged level, the structure will deflect towards the waterside. For these conditions the lateral earth pressure on the landward side of the wall may be taken as fully active. However, where the two rows of sheet piles are driven on to a hard rock surface at, or a short distance below, dredged level, the wall will act wholly as a gravity structure and the lateral earth pressure on the landward side of the wall will be in a condition intermediate between the active and at-rest states depending on the amount of horizontal deflection of the structure.

The bearing capacity of the soil beneath the double-wall structure should be adequate to support the weight of the structure and infill material, any superimposed loading on the walls and the vertical force transmitted to the landward side sheet piles due to wall friction and the impact forces of waves and berthing ships. Guidance on methods of determining the factor of safety against failure in shear of the soil beneath the structure is given in CP 2004.

Where the double-wall structure is founded on a soft clay, it will be necessary to remove the soft clay from within the cells before placing the filling between the sheet piles. The possibility of heave of the soil below excavation level should be investigated [35].

Methods for calculating the bending moments and shears in sheet piles and the forces in the connecting tie rods or sheet pile bulkheads are to be found in subsequent Parts of this code.

The overall stability against sliding of the wall and the adjacent earth masses should be considered as described in **51.6.1**.

51.6.4 *Cellular structures.* The flat web sheet piles used for the circular-arc types of cellular structure (see **51.2.4**) have little stiffness in a vertical direction, consequently lateral earth pressure on the landward face of the cells is resisted by internal shear forces within the material filling the cells. The structure will deform in mobilizing these internal shear forces; therefore the earth pressure on the landward side may be taken as corresponding to the active pressure state. The material filling the cells is confined by the hoop tension forces acting on the interlocks between the sheet piles, consequently the internal lateral pressure on the sheet piles should be taken as corresponding to at-rest conditions.

Drainage should be provided in the cell walls so that hydrostatic pressure within the cells and on the landward side may be assumed to act as described in **51.5.2**.

Where the sheet piles are driven into soil below dredged level [see Figure 62(a)], the resistance to sliding or over-turning of the structure will be provided partly by passive resistance of the soil below dredged level, partly by the pull-out resistance of the sheet piles on the landward side, partly by skin friction on the base of the filling material and partly by the gravitational forces of the structure and infill material.

Where the sheet piles are driven on to hard rock at, or a short distance below, dredged level [see Figure 62(b)], the resistance to overturning and sliding will be provided by base friction and gravitational forces only.

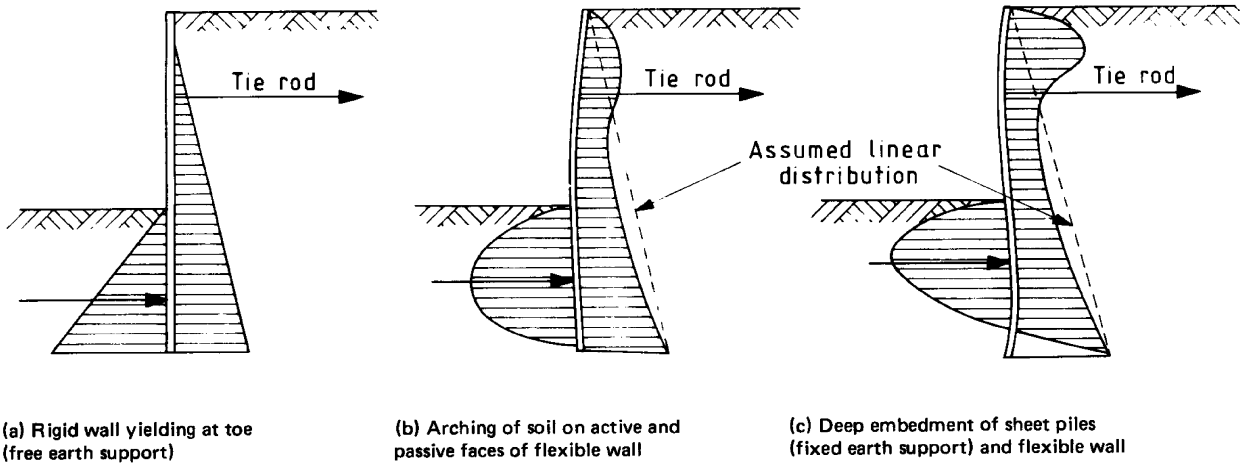

Figure 58 — Effects of flexibility of anchored single-wall sheet pile structure on earth pressure and resistance

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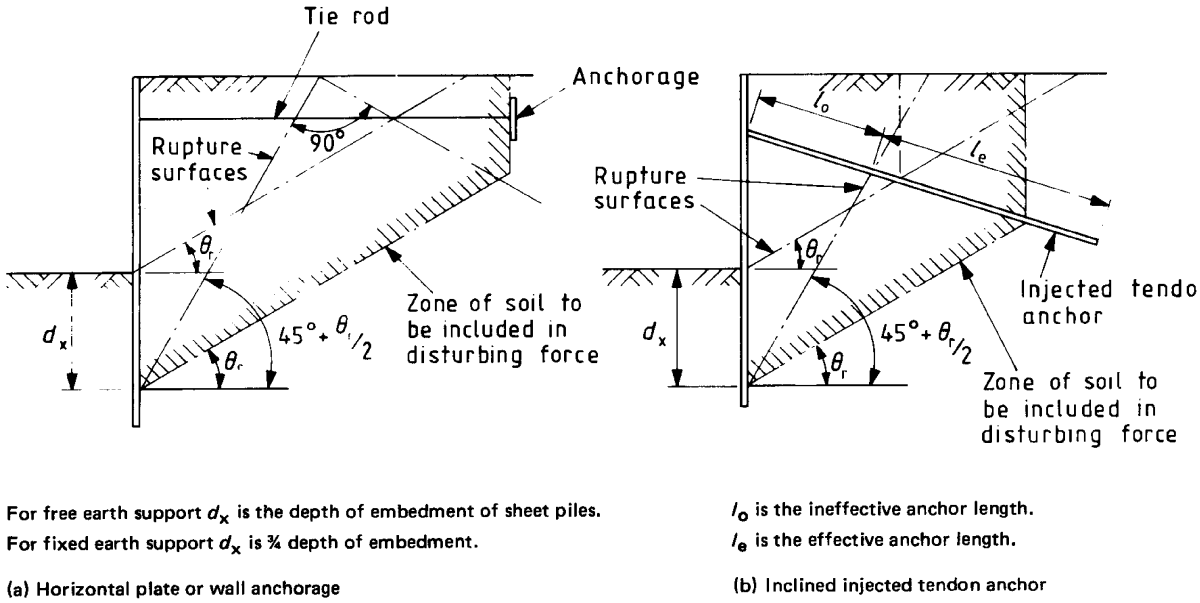


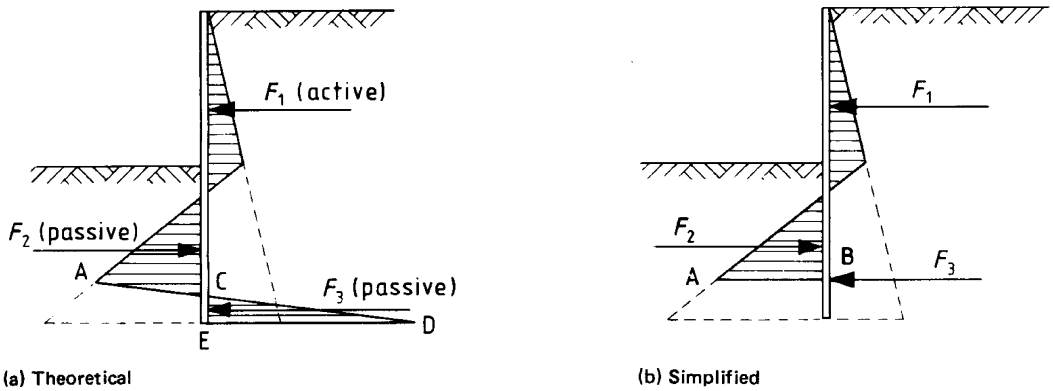
For free earth support d_x is the depth of embedment of sheet piles.
For fixed earth support d_x is ¾ depth of embedment.

(a) Horizontal plate or wall anchorage

l_o is the ineffective anchor length.
l_e is the effective anchor length.

(b) Inclined injected tendon anchor

Figure 59 — Location of anchorages in relation to rupture surfaces in retained fill



(a) Theoretical

(b) Simplified

Figure 60 — Simplified distribution of active earth pressure and passive earth resistance on cantilevered single-wall sheet pile structure

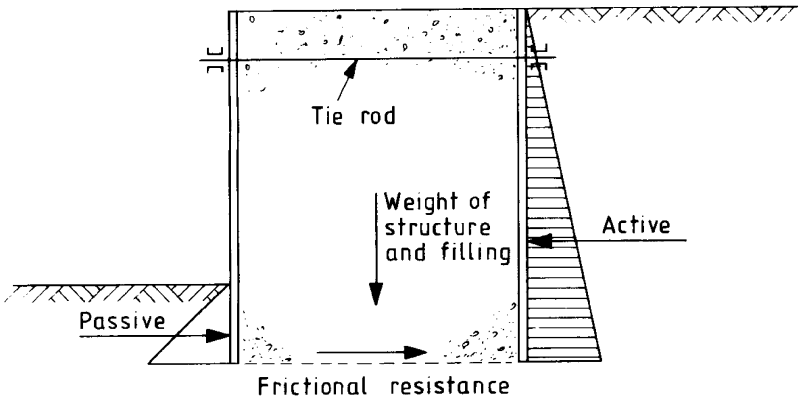


Figure 61 — Distribution of active earth pressure and passive earth resistance on double-wall sheet pile structure

Investigations of the stability of cellular sheet pile structures should consider the following modes of failure:

a) tilting due to failure in vertical or horizontal shear within the fill material [see Figure 63(a)];

b) tilting due to rotational failure on a curved rupture surface at or near the base of a cell [see Figure 63(b)];

c) interlock separation [see Figure 63(c)];

d) shear failure of the soil beneath the base [see Figure 63(d)];

e) horizontal sliding on the base [see Figure 63(e)];

f) tilting accompanied by escape of soil from beneath the sheet piles when structures are not backed by retained soil [see Figure 63(f)];

g) tilting without loss of filling material [see Figure 63(g)];

h) general shear slide of the structure and the soil retained by and beneath the structure [see Figure 63(h)].

Methods of analysing the stability of cellular structures against failure modes a) to g) and guidance on determining the factor of safety against failure mode h) are given elsewhere [36, 37].

Where the sheet piles are driven into a soft clay, it is necessary to remove this material from within the cells before placing the filling. The possibility of heave of the soil within or beneath the cells should be investigated [35].

Methods for determining the required sections of sheet piling to resist external and internal earth pressures and water pressures are to be found in subsequent Parts of this code.

51.6.5 *Anchored double-wall and cellular structures.* If anchors are required to provide an additional restraining force against sliding or overturning of double-wall or cellular structures, the distribution of lateral pressure on the sheet pile walls and the forces on the anchors will depend on the flexibility of the sheet piles, as in the case of anchored single-wall structures (see **51.6.1**).

Guidance on the calculation of bending moments and shears on the sheet piling and forces on the anchors is to be found in subsequent Parts of this code.

52 Gravity structures

52.1 General. Gravity structures for wharves and dock walls can be divided into five groups as follows:

a) masonry or brickwork;

b) mass concrete;

c) precast concrete blocks;

d) monolith;

e) caisson.

Gravity structures may be considered with advantage where skilled supervision and labour are not readily available. The height of the wall may be limited by the allowable pressure on the foundation. They also provide advantages where heavy abrasion or impact is likely or the water has a high sulphate content.

The problem of assessing earth active pressures and passive resistance is applicable to all the groups.

Soil exploration should be carried out on the following:

1) the earth to be retained;

2) the ground in front of the wall;

3) the foundation layer;

4) the underlying soil layers.

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The shape of the wall may be influenced by factors other than stability, e.g. use of the space in front of structure, appearance and constructional methods. Reduction in cost may result if the wall is inclined or battered back so that the resultant compressive stress is uniform over any section, including the base and the foundation. An alternative is to step the back of the wall where it is not practicable to form a batter.

Investigations of the stability of gravity structures should consider the following modes of failure:

i) tilting due to failure of the soil beneath the base;

ii) horizontal sliding on the base;

iii) rotational failure due to a curved rupture surface near the base;

iv) general shear slide of the structure and the soil retained by and beneath the structure.

Failure may be caused by an increase of active pressure due to inadequate drainage or excessive loading giving rise to tension in the back of the wall. Sliding may then occur along the planes of cracks or tilting through crushing of material at the face of the wall.

52.2 Masonry and brickwork. In the past, where the site could be dewatered and an adequate foundation existed, masonry and brickwork provided an economical form of construction. Refinements introduced to reduce the cost include the use of buttresses, arched forms or cellular construction but no important structure of this type has been built for many years.

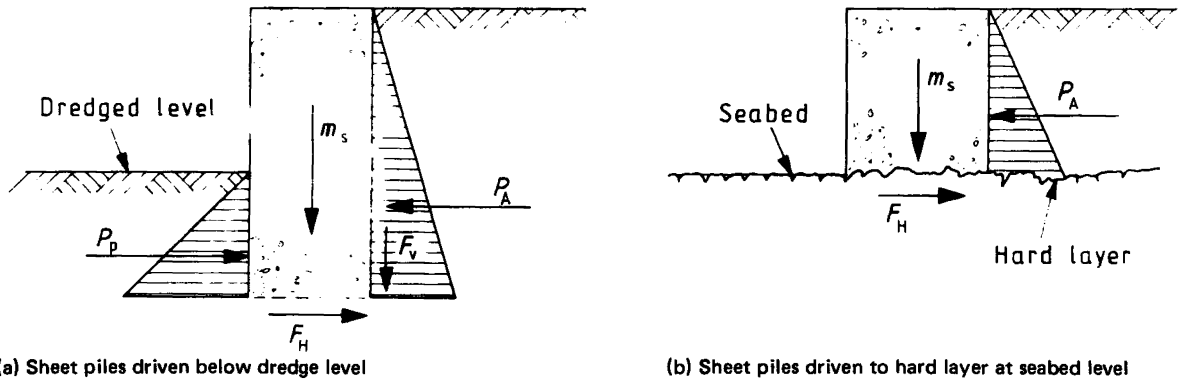
52.3 Mass concrete walls. The simplest form of gravity wall is mass concrete placed in a trench under dewatered conditions, the ground in front of the wall being excavated or dredged afterwards. In favourable ground conditions, trenches can be taken out to a depth of about 30 m.

Large pieces of clean stone or old cement concrete may be introduced into thick mass concrete walls as displacers or plums. These should be evenly distributed and so placed as to avoid steeply inclined junction planes in the concrete and not be too close to faces.

Construction joints in mass concrete walls should be kept to a minimum. Horizontal joints should be made with a longitudinal groove of a width of about one-third of the thickness of the wall. Vertical joints should coincide with contraction or slip joints. Slip joints should be placed at steps in the foundation, changes of section or where the nature of the foundation changes.

The foundation should be designed to avoid differential or excessive settlement, horizontal movement or tilting.

52.4 Concrete blockwork. The use of precast concrete blockwork provides advantages when time available on site is limited and it can be placed under water. Conditions required for this technique include the presence of hard material near foundation level, a long length of construction and the availability of heavy plant to handle the blocks. Concrete blockwork may be placed on rock or a prepared rubble base with horizontal and vertical joints and sometimes it is inclined with nearly vertical joints, i.e. slice work.



(a) Sheet piles driven below dredge level

(b) Sheet piles driven to hard layer at seabed level

Figure 62 — Distribution of active earth pressure and passive earth resistance on cellular sheet pile structure

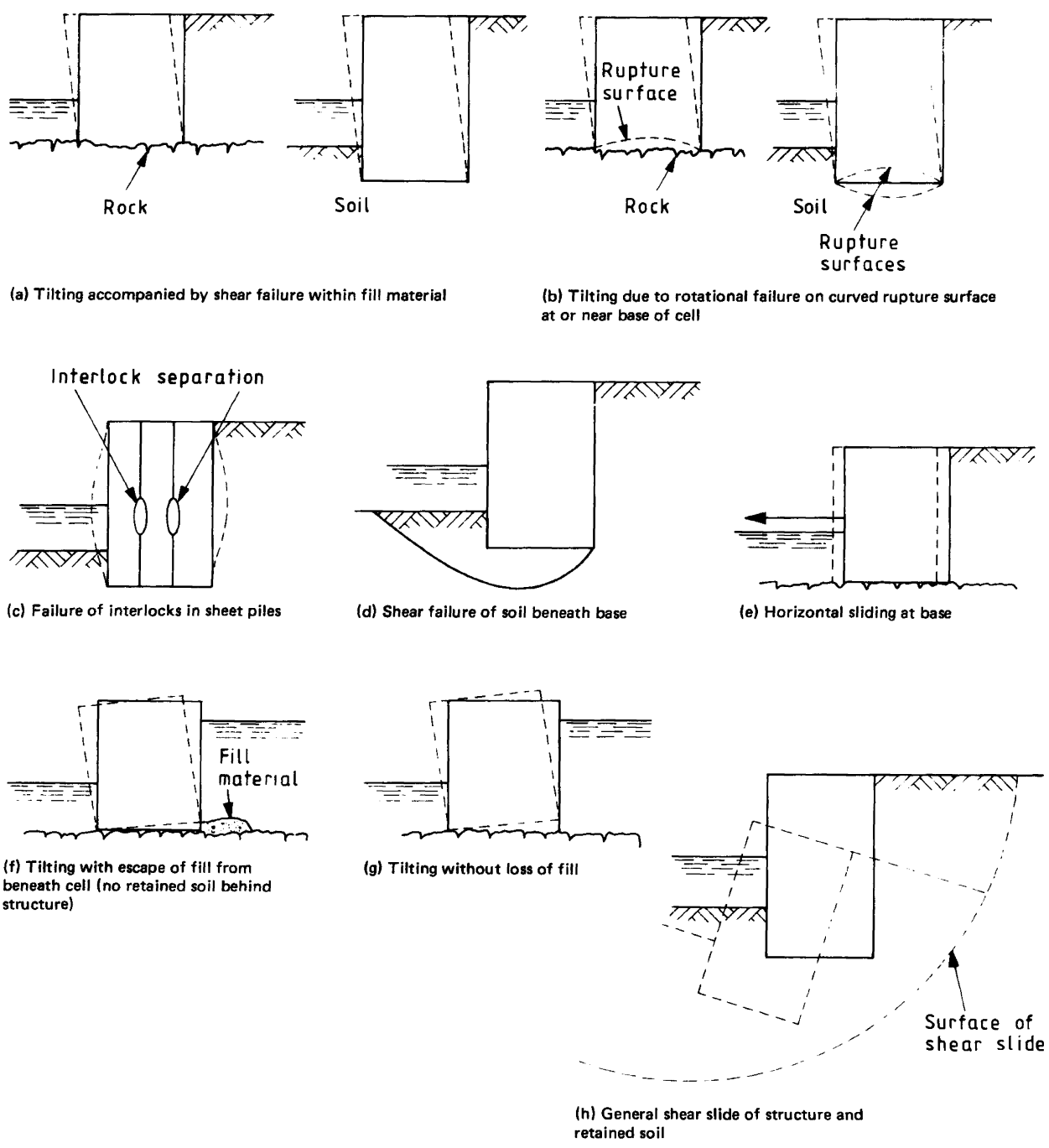


Figure 63 — Modes of failure of cellular sheet pile structure

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52.5 Monoliths. In poor ground and where great depth is required, concrete monoliths may be adopted. These may be of precast blocks or cast in situ as the monolith is sunk. Monoliths are usually square in plan, about 15 m × 15 m, and constructed in cellular form, the rear cells being filled with concrete hearting and the front cells being left to fill with water. To assist in sinking, they are loaded with kentledge and to lessen resistance due to friction, the lowest sections are sometimes increased in dimension to form a space which is filled with bentonite, which acts as a lubricant.

To allow for possible sinking out of true vertical, gaps of up to about 3 m are left between monoliths. The gaps are closed by piling after sinking to the desired level.

52.6 Caissons. Where special facilities, such as an old dock, are available, reinforced concrete floating caissons may be used. These are similar in form to open topped boxes and may be divided into cells. They are constructed in the dry and launched and floated to the site, where they are sunk on a prepared foundation, which may consist of piles or have been formed by levelling an area of the bottom. The retained ground may be formed by subsequent reclamation. After sinking of the caissons, filling is essential to provide mass for stability.

52.7 Design of gravity structures. Guidance on the detailed design of the various types of gravity wall is to be found in subsequent Parts of this code.

53 Anchorage of structures

53.1 Function and location of anchorages. Anchorage systems are used in maritime structures to restrain the structures against movement caused by earth pressure, hydrostatic pressure, wave-impact forces, berthing-impact forces and mooring-rope pull. Wind loads, earthquake loads, loads due to thermal stresses and pipe anchor forces may also need to be considered.

In the case of earth-retaining structures such as quay walls, lock chambers and dock walls, the anchorages are placed behind the waterside face of the structure in such a manner as to mobilize the gravitational and passive resistance of a body of ground between the anchor and the structure (see Figure 59).

Resistance to hydrostatic uplift beneath the floor of a lock chamber or dry dock can be provided by anchors which mobilize the gravitational weight of a body of ground beneath the floor (see Figure 64).

Resistance to wind and wave forces, the impact of berthing ships and mooring rope pull on jetties and dolphins is provided by the passive resistance of the soil on the embedded portions of vertical piles forming these structures or by combined base friction and passive resistance on the embedded portions of caissons or monoliths used as an alternative, or supplement, to piled structures. Where passive resistance or skin friction resistance is adequate to restrain movement against lateral forces it may be expedient, nevertheless, to provide additional resistance in the form of raking piles (see Figure 65).

53.2 Methods of anchorage

53.2.1 *Buried blocks and walls.* Restraint to forward movement of deadman anchorages in the form of buried blocks or walls is provided by the passive resistance of the soil against the face of the member.

Consideration should be given to the location of the anchorage in relation to potential failure planes within the mass of the soil retained by the structure. An anchorage placed within the active failure wedge (see **51.3**) will be ineffective [see Figure 66(a)]. The anchorage will be partially effective if placed at a location where there is overlapping of the zone of passive resistance with the active failure zone [see Figure 66(b)] and fully effective if placed beyond zones of potential failure [see Figure 66(c)].

The passive resistance of an anchor block or wall should be taken as the net resistance when the active pressure on the rear face of the block or wall is deducted from the total passive resistance and wall friction (see **51.3**) should not be allowed for on the vertical faces of the anchor structure.

In the case of buried blocks, the shearing resistance of the soil against the sides of a wedge of soil adjacent to the front face of the block may be allowed in addition to the passive resistance of the soil in front of the block. This additional resistance should not be allowed within the top metre of a cohesive soil in situations where tension cracks can develop in the soil in a direction parallel to the tie rod. In the case of a row of closely spaced blocks, the total of the net passive resistance on the front face of all the blocks, together with the additional resistance of the soil on the side of the wedges in front of the blocks, should not exceed the net passive resistance as calculated for the case of a continuous wall of the same length as the row of blocks.

When calculating the passive resistance on the face of a block or wall and the resistance of the wedge of soil in front of a block, the overburden pressure should not include any surcharge loading applied to the ground surface. The position of any surcharge loading should be considered in relation to producing the maximum lateral pressure on the earth-retaining structure, without applying vertical pressure to the soil above the anchorage.

In selecting soil parameters for calculating passive resistance, the effects of cyclic loading on the anchorages should be considered. This form of loading may be caused by cyclic variations in hydrostatic pressure on the structure due to varying tidal levels.

Where backfill is placed against the vertical faces of buried block or wall anchorages, care should be taken to compact the fill in such a manner that the required passive resistance to forward movement of the anchorage will be obtained in a uniform manner.

Consideration should be given to the possibility of bending stresses being induced in the tie rod due to settlement of the soil behind the earth-retaining structure. Methods of relieving these bending stresses and guidance on the methods of transfer of lateral forces from the structure to the anchorages are to be found in subsequent Parts of this code.

53.2.2 *Injected tendons.* Injected tendon anchors used to restrain forward movement of an earth-retaining structure should be inclined at an angle below the horizontal in order to mobilize the resistance of the largest possible volume of soil in the zone beyond the active wedge. The tendons may be placed in a single row [see Figure 67(a)] or in multiple rows [see Figure 67(b)]. Load transfer from the tendon to the soil is effected by the bond between the grout surrounding the bar or cable and the soil. The following factors should be considered in the design of the anchorages:

a) the tensile strength of the tendon;

b) the allowable bond stress between the tendon and the grout used to fill the annulus between the tendon and the soil;

c) the allowable bond stress between the grout and the soil;

d) the passive resistance of the body of soil surrounding and stressed by the tendons;

e) the vertical compression force induced in the earth retaining structure by the tensile force in the anchor;

f) the corrosion protection to the anchors and tendons.

Injected tendon anchors may be installed as unstressed, or dead, tendons or as stressed anchors. In the latter case a tensile force is applied to the anchor after allowing a suitable period of time for the grout to harden. The tensile force should be 1.5, or other suitable multiple, times the design working load in the anchor.

Injected tendon anchors may be used to restrain uplift forces on the underside of the floors of lock chambers, dry docks or other below ground structures. The tendons may be installed vertically or as a combination of vertical and inclined members (see Figure 64). The length of the tendons should be such as to mobilize the gravitational force of the mass of soil or rock stressed by the system. The volume of soil or rock stressed by a single anchor may be considered to act as a cone with its apex at the tip of the anchor (see Figure 68). The half angle of the cone will depend on the angle of shearing resistance of the soil or, in the case of anchors installed in rock, on the inclination of bedding, joint spacing and degree of weathering of the rock mass (see Figure 69). Half angles between 30° and 45° are commonly employed but consideration should be given to ground conditions where only a limited volume of soil or rock can be mobilized in resistance to anchor pull. An example of this is given in Figure 69(c) [38].

Allowance should be made for the overlapping of the cones of stressed soil or rock where tendons are installed at a close spacing. Where stressed anchors are used to restrain uplift on floor slabs, consideration should be given to deformation of the slab due to compression of the soil beneath the floor. Large deformations may occur where the floor is underlain by soft clay.

In selecting soil parameters for the design of injected tendon anchors to resist lateral or vertical uplift forces, consideration should be given to the effects of cyclic loading on the soil. Cyclic loading may be caused by variations in hydrostatic pressure on an earth-retaining structure or beneath an anchored floor slab due to variation in tidal levels.

Guidance on allowable stresses in tendon anchorages and on the procedure for installation and stressing is given in DD 81.

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53.2.3 *Raking piles.* The application of a lateral force to the head of a pair of raking piles induces axial forces in the piles. The compressive force is resisted partly by skin friction on the pile shaft and partly by the end resistance of the pile and the tensile force is resisted only by skin friction on the pile shaft and by the gravitational force from the pile and pile cap. Where raking pile anchorages are used to restrain forward movement of an earth-retaining structure, the load from the structure may be transferred to the pair of raking piles by means of a slab cantilevered from the facing wall [see Figure 70(a)] or through a tie rod [see Figure 70(b)].

Guidance on the method of determining the skin frictional and base resistance of piles is given in CP 2004. A reduction in the depth of embedment to achieve resistance to compression or tension loading may be achieved by forming an enlargement at or near the toe of the piles. Where raking piles carrying axial tension forces are driven on to a rock formation, the depth of penetration of the rock may not be sufficient to develop the required resistance in skin friction. In such cases the resistance can be provided by using a hollow open-ended pile driven to rock. The soil and loosened rock within the pile is then cleaned out and bar or cable anchors are installed by grouting them into holes drilled in the rock. The required depth of anchorage into the rock may then be determined taking into account the considerations set out in **53.2.2**.

Raking piles may be required to carry vertical forces from surcharge loading applied to the deck of a wharf structure or quay wall. This loading is beneficial in reducing the tension forces in the raking piles but it may produce unacceptably large axial forces in the compression piles when taken in conjunction with the compression loading from lateral forces in the pile group.

The conditions giving rise to maximum compression and tension loads should be considered for the purpose of determining the size and depth of embedment of the raking piles and any supplementary anchorages.

When raking piles are driven through layers of fill or soft clay, settlement of these layers under their own weight or under the influence of surcharge loading may cause axial dragdown forces and bending stresses on the piles. Dragdown conditions should not be taken into account when calculating the resistance of piles to tension forces.

If a large number of closely-spaced horizontal ground anchors is unacceptable (as by inhibiting freedom of subsequent foundation development in the anchor zone) the spacing can be increased to the extent that the stiffness of the capping beam can tolerate. In that case the tendons would preferably be of prestressing cable. An advantage of the system which has been proved in newly-placed granular fill is that such cables, if anchored into concrete boxes, can be overstressed so that the anchorage is moved forward into the fill until an excess of passive resistance has been developed. When relaxed for the working condition the system then has a proven factor of safety on stability.

Raking piles may be used in dolphins to provide resistance to the pull of mooring ropes. The effects of dynamic loading due to the ranging of ships and wave-impact forces on the cohesion and angle of shearing resistance of the soil should be considered when selecting parameters for the purpose of determining the required depth of embedment of the piles. Consideration should be given to the effects of seabed erosion and of vibrations or oscillations of the piles on the effective depth of embedment, in deep water conditions the effects of wave-induced pore pressures should also be taken into account [40].

53.2.4 *Vertical anchorages.* As an alternative to injected tendon anchors for restraining hydrostatic uplift on the floors of lock chambers, dry docks and the like, vertical anchorages in the form of tension piles or buried plates may be used. Tension piles taken down to a relatively incompressible stratum are a useful expedient for restraining uplift where a dock floor is immediately underlain by a weak compressible soil. The piles act in support of the floor and walls at times when the dock is impounded or when the empty dock carries a heavy ship load bearing on keel blocks.

The required depth of embedment to carry the uplift loads in skin friction on the pile shaft can be determined by the methods described in CP 2004. Where the piles are driven on to a hard stratum and the depth of embedment is insufficient to obtain the required uplift resistance in skin friction, anchorages can be provided below the interior of hollow piles as described for raking piles in **53.2.3**.

Buried plate anchorages are installed by drilling a hole to the required depth. A circular plate is lowered to the base of the hole by means of a rod or cable which forms the anchor tendon. Soil is placed in the hole and compacted in layers up to the ground surface. Alternatively the buried plates can be installed by driving or vibrating them into the soil by means of a withdrawable mandrel or by screwing a spiral plate into the soil.

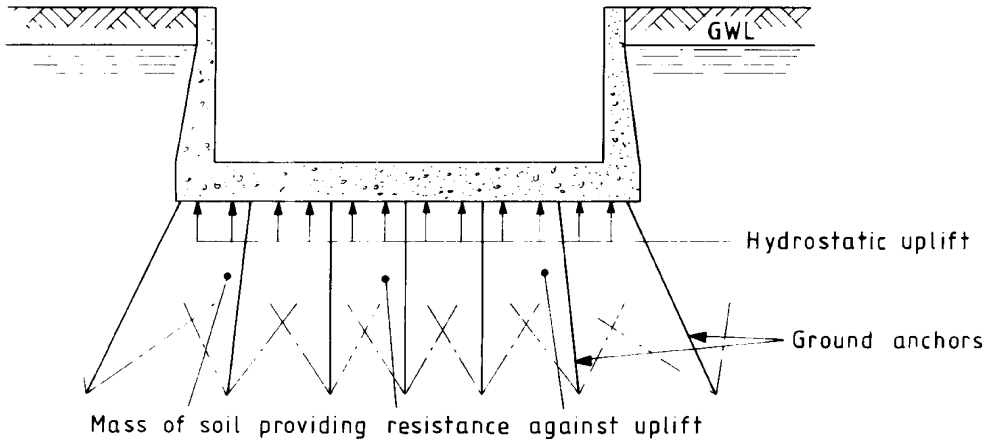

Figure 64 — Anchorage of lock or dry dock against hydrostatic uplift

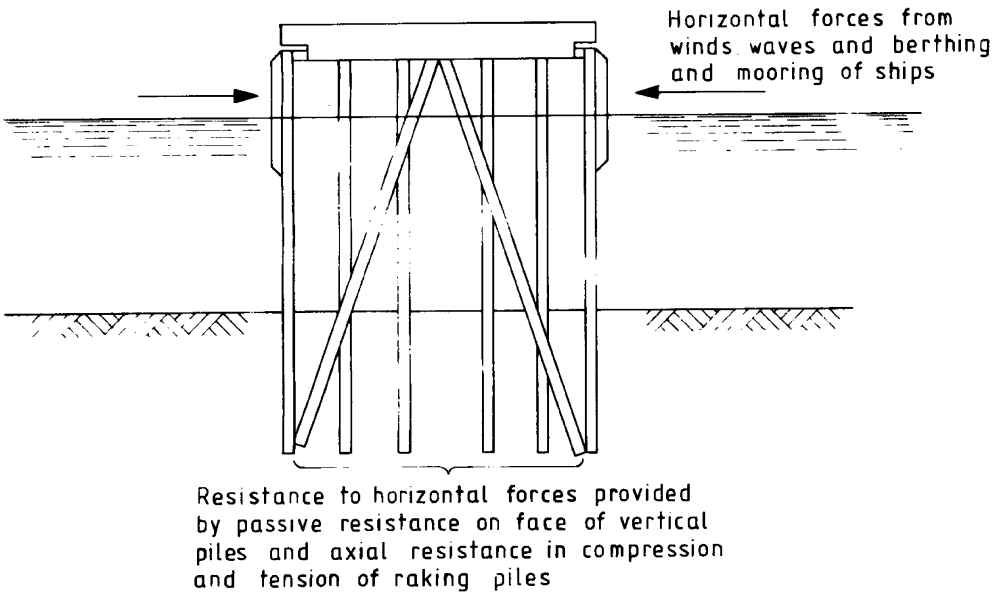

Figure 65 — Raking piles used to restrain horizontal movement of jetty

The uplift resistance of buried plate anchors is provided by the weight and skin friction on the cylinder of soil above the plate. The method of Meyerhof and Adams may be used to calculate the uplift resistance [41].

The effects of cylic loading due to tidal variations in the hydrostatic pressure beneath the anchored slab and the possibility of reversals of stress in the anchors due to ship loading in dry docks should be considered.

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54 Slope stability and protection

54.1 Environmental factors

54.1.1 *Pore pressure effects.* The gravitational forces acting on the mass of soil forming a slope together with any loads applied to the surface of the slope or beyond the crest act as disturbing forces tending to create instability in the form of a shear slide (see **54.2**). The disturbing forces are resisted by restoring forces which are partly or wholly in the form of the frictional resistance of the soil on the plane of sliding. In terms of effective stresses the resistance is given by:

$$R = c' + (\sigma - P_u) \tan \phi'$$

where

R is the resistance per unit area to shear along an actual or potential slip surface;

c' is the effective cohesion of the soil;

σ is the stress normal to the plane of sliding;

P_u is the pore water pressure;

ϕ' is the effective angle of shearing resistance of the soil.

When considering the stability of slopes in the long term, the effective cohesion of all soil types should be taken as zero. Consequently the pore pressure has a critical effect on the shearing resistance which can be mobilized on the potential plane of sliding.

Consideration should be given to the effect on pore pressures in the soil behind the slope due to variations in level of the waterway. In the case of partly submerged slopes, the effects of variation, in level of the ground water from landward sources should also be considered.

The fabric of the soil, that is the presence of fissures or layers or laminations of permeable soil interbedded with impermeable soils, has an effect on variations on pore pressure and the manner in which these discontinuities provide a means of drainage from a soil or a means of ingress of tidal water should be taken into consideration.

Variations of water level due to wave action on a slope can also cause variations in pore pressure behind the slope and the depth of soil affected by these rapid fluctuations in pore pressure should be assessed. In the case of submarine slopes in deep water, consideration may be given to numerical analyses to determine the wave-induced pore pressures and effective stresses [40].

Increase in pore pressure in the soil behind a slope may be caused by constructional operations such as displacement of the soil by pile driving, or by dumping materials on to or beyond the crest of a slope.

54.1.2 *Changes in slope profile.* The possibility of instability due to steepening of a slope should be considered. Steepening may be caused by erosion of the toe of a slope due to the effect of tidal or river currents, or the wash from ships (see **54.5**). Wave action can also cause changes in slope profile due to the effects of undercutting and deposition of loose material by uprush of waves.

Consideration should be given to the effect of steepening of the upper part of the slope by dumping material at the crest or by the deposition of soil by the natural processes of accretion.

54.1.3 *Other effects.* In areas of known seismic activity, the effects of earthquakes on the stability of slopes in soils sensitive to reduction in shear strength by disturbance should be examined.

In tidal waters, blocks of ice adhering to the soil at the water line can cause degradation of a slope on the falling tide.

Large scale instability due to flow slides on a regional scale in loosely deposited littoral sands or in deltaic deposits require special consideration. Information on these conditions is given elsewhere [42, 43].

54.2 Modes of failure

54.2.1 *General.* Modes of failure of slopes formed by excavation below the seabed or by dumping fill on to existing surfaces are described in **54.2.2** to **54.2.10** and are similar to the modes of failure of embankments and cuttings described in **6.3** of BS 6031:1981.

54.2.2 *Rotational slide.* Failure by rotation of a body of soil on a circular surface of sliding [see Figure 71(a)] is the usual mode in uniform soil conditions in structure-less or heavily jointed rock masses. Rotational slides act three-dimensionally when a spoon shaped mass of soil slips on a concave surface [see Figure 71(b)]. Rotational slides may be non-circular in form in anisotropic soil conditions or in the presence of a relatively strong layer at a horizontal or shallow inclination when sliding takes place on this layer [see Figure 71(c)].

Rotational sliding also takes place where an embankment or structure is placed on weak soil which is unable to support the imposed loading (see Figure 72).

54.2.3 *Compound slides.* Compound slides are partly rotational and partly translational in character and occur in soils where heterogeneity results in shear failure taking place on preferred surfaces which may not have any regular pattern in relation to the geometry of the slope. Compound slides are typical of stiff over-consolidated clays where surface shrinkage cracks and natural fissures form multiple sliding surfaces. These multiple slips can occur at random positions and be of varying dimensions on a slope (see Figure 73) or failure may take place near the toe followed by successive slips, working back to the crest. The latter are known as multiple retrogressive slides.

54.2.4 *Translational slides.* Translational slides occur as a result of weakness in a soil or rock mass at a fairly shallow depth beneath the slope surface. The slide involves the bodily movement of a shallow mass on a planar surface roughly parallel to the slope.

54.2.5 *Slab and block slides.* Slab and block sides are forms of translational movement where the sliding mass remains more or less intact. A slab slide (see Figure 74) typically occurs on the weathered surface of an existing slope. In rock slopes, large slabs of rock can slide on a weak clay-filled joint parallel to the slope. A block slide occurs when a block of relatively strong rock or stiff to hard clay moves down slope as a unit on a plane of weakness in the form of a fissure or joint roughly parallel to the slope.

54.2.6 *Wedge failures.* Wedge failure is three-dimensional in form, when a wedge of rock or stiff clay slides bodily forward and downward on two or three well defined joint planes which intersect behind the slope (see Figure 75).

54.2.7 *Debris flows.* Debris flows occur when water has access to debris forming a mantle on a slope [44, 45]. The water and debris in a random unsorted form move down the slope, either slowly in a creep movement or rapidly at times of saturation of a slope by the uprush of high waves, by heavy rainfall or as a result of diversion of surface water on to a slope.

54.2.8 *Flow slides.* Flow slides occur in loose to medium-dense saturated sands as a result of a sudden increase in pore pressure in the mass. In loose uniform fine sands flow slides can occur on relatively flat slopes, the bodily movement occurring over long distances at a high velocity [42, 43]. Sudden pore pressure increases can be caused by earthquakes, by rapid drawdown in water levels, or by disturbance of the soil mass consequent on failure due to local steepening by over dredging or erosion. Flow slides in cohesive soils are usually termed mud flows.

54.2.9 *Toppling.* Toppling failures occur in rock slopes when discontinuities behind the face are steeply inclined (see Figure 76). They may occur as a result of water pressure behind the slope.

54.2.10 *Internal erosion.* Internal erosion can occur behind the face of an excavation in a water-bearing layered soil formation consisting of interbedded permeable and impermeable soils. Water flows through the permeable layers and erosion occurs if the water emerges at the face with sufficient velocity to cause movement of soil particles. The erosion works backwards into the slope until it reaches the stage when undercutting and collapse of the overlying material occur. The collapsed materials then move down the slope in the form of a debris flow.

54.3 Design considerations for slopes and embankments

54.3.1 *Stability analysis.* Guidance on the methods of analysing the stability of slopes, including the slopes of embankments formed on the seabed, can be obtained from BS 6031. Particular attention should be paid to first the effects of variations in pore pressure within the soil mass due to the factors already described (see **54.1.1**) and secondly to the other environmental effects referred to in **54.1.2** and **54.1.3**.

In most cases only the long term stability of dredged slopes and embankments will need to be considered when the stability analyses, both in cohesionless and cohesive soils, should be made in terms of effective stresses. Stability analyses in terms of total stresses based on the undrained shear strength of the soil should be limited to cases where only short term stability is required, for example in excavations for placing box caissons or monoliths for quay walls.

In cuttings, consideration should be given to the effects of removal of overburden pressure due to dredging or above water excavation to the design formation level.

When constructing embankments on cohesive soils, the gain in shear strength due to consolidation of the soil under the imposed loading should be taken into account and the rate of placing the embankment fill should be controlled to allow time for dissipation of pore pressures which results in the gain in strength of the soil.

When preparing designs for the alignment and profile of a slope formed by dredging or by placing material to form an embankment, the possibility of local slips or falls occurring on the face should be considered in addition to the overall stability against the various forms of failure described in **54.2**.

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Local slips or falls may occur due to the presence of random pockets of weak or erodible soils or thin layers of weak or shattered rocks. In the case of underwater slopes, occurrences of local instability cannot be readily detected and remedial works are limited to dumping of material to form a stable profile or to surface protection by mattressing (see **54.5**). A conservative approach should therefore be adopted in the selection of a profile for the permanently submerged area.

Local instability can be detected in the areas above high water and in inter-tidal zones when the appropriate remedial action can be taken as described in **54.5.3** and **54.7**. Therefore an overall flattening of above-water slopes to avoid the occurrence of local failures is rarely justified.

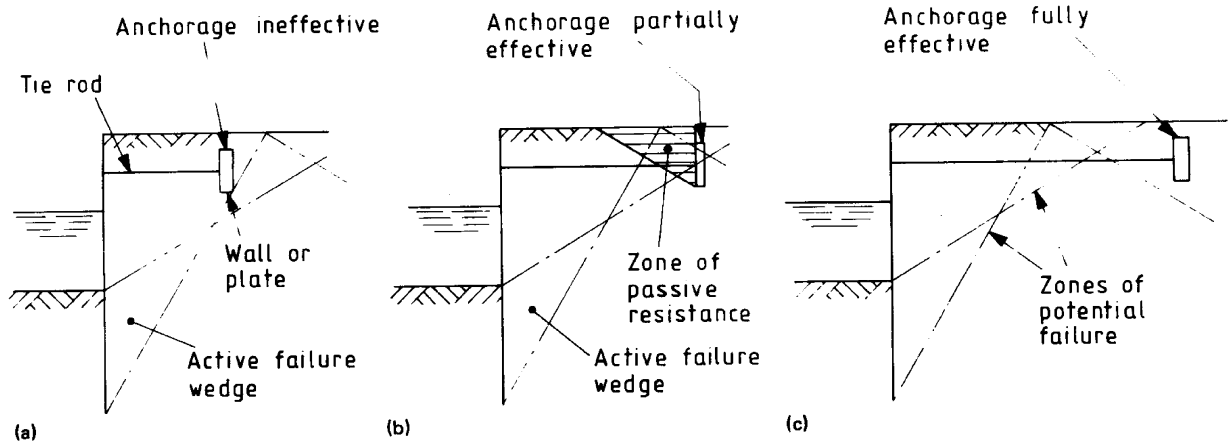

Figure 66 — Location of wall or plate anchors in relation to failure surfaces

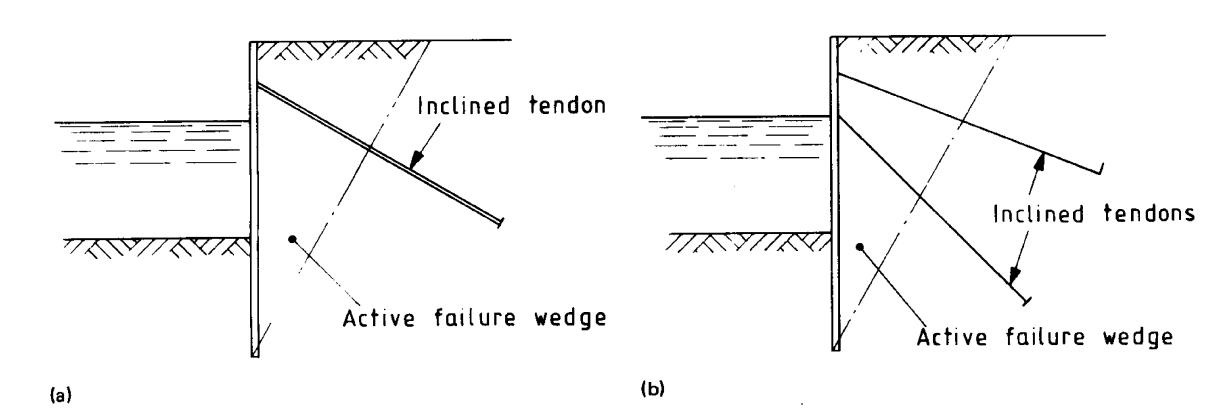

Figure 67 — Location of injected tendon anchors in relation to active failure wedge

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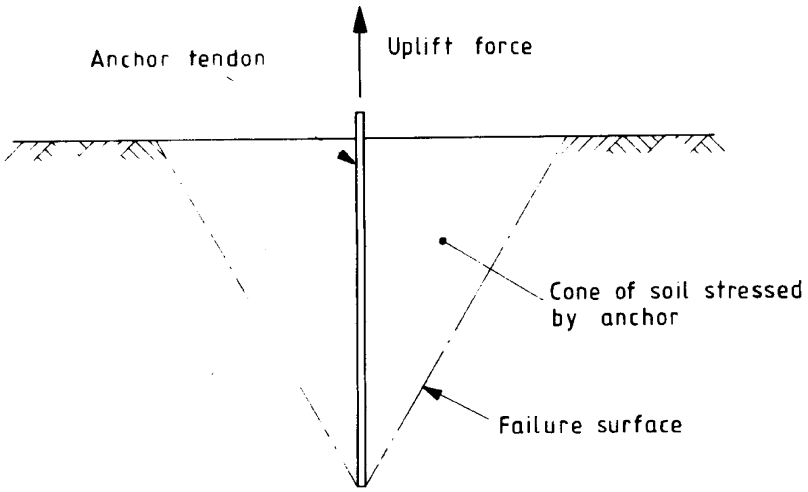
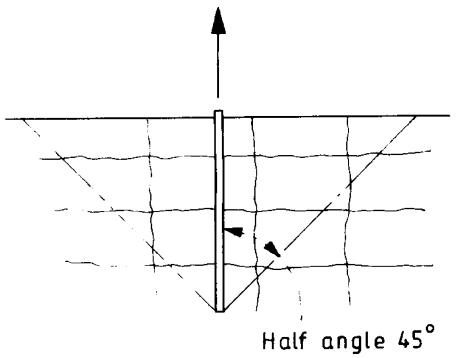
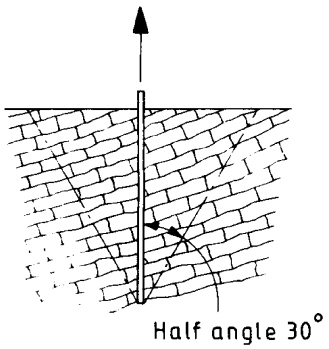


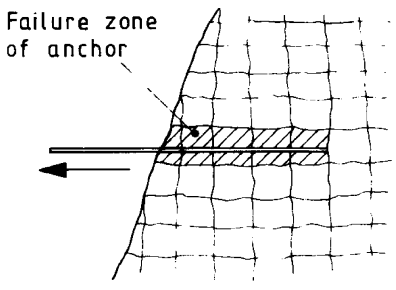
Figure 68 — Failure of vertical anchor tendon due to uplift of cone of soil or rock



(a) Massively bedded rock



(b) Thinly bedded rock with horizontal or near-horizontal bedding joints



(c) Horizontal anchor in horizontal joint system

Figure 69 — Modes of failure of anchor tendons in rock

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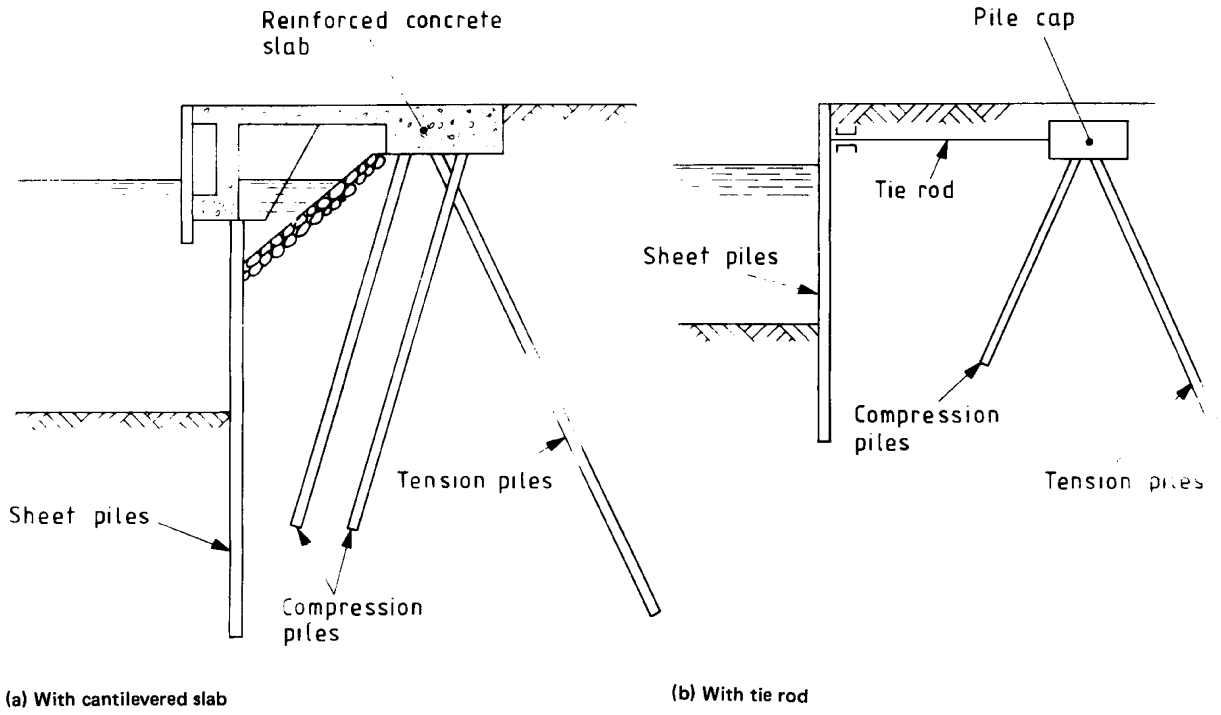

Figure 70 — Anchorage of single-wall sheet pile structure by raking piles

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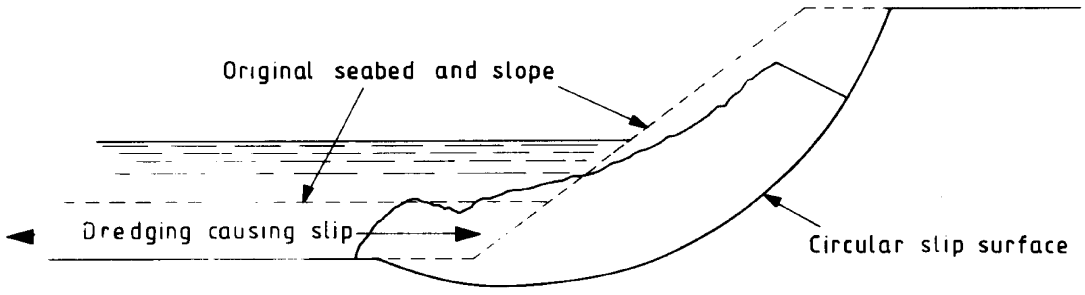
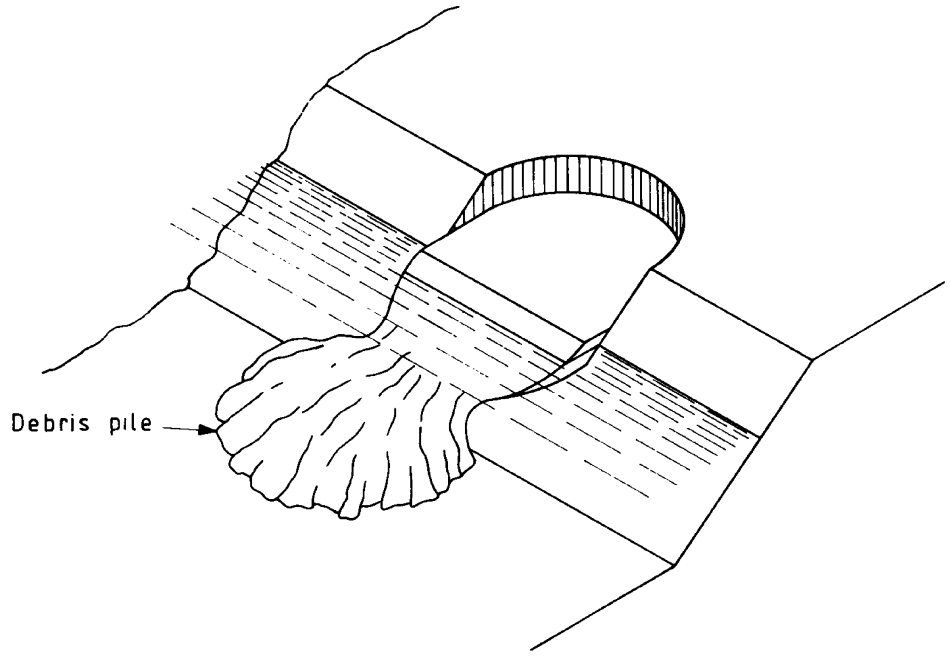
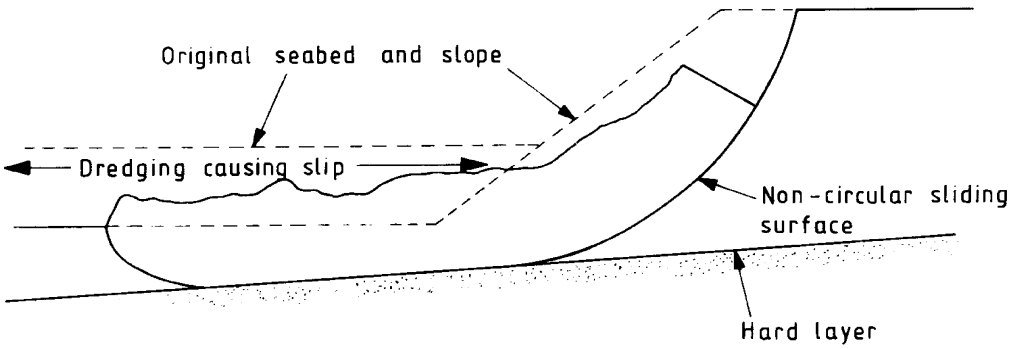


(a) Rotational shear slide on circular sliding surface



(b) Three-dimensional rotational slide



(c) Non-circular rotational slide

Figure 71 — Types of slope failure

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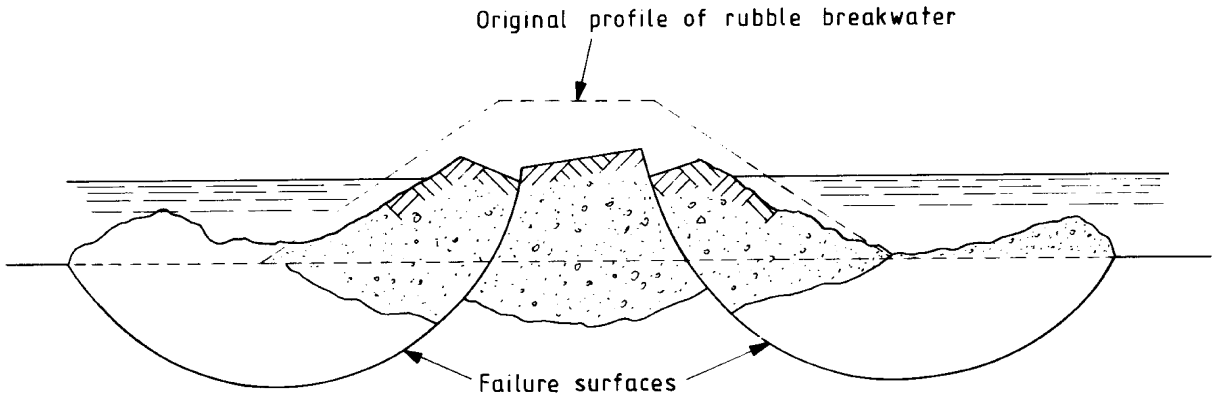

Figure 72 — Failure of embankment due to shear failure of supporting soil

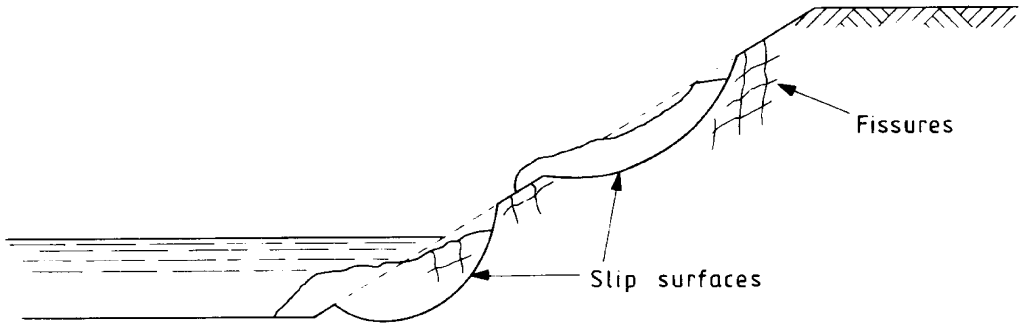

Figure 73 — Compound slide

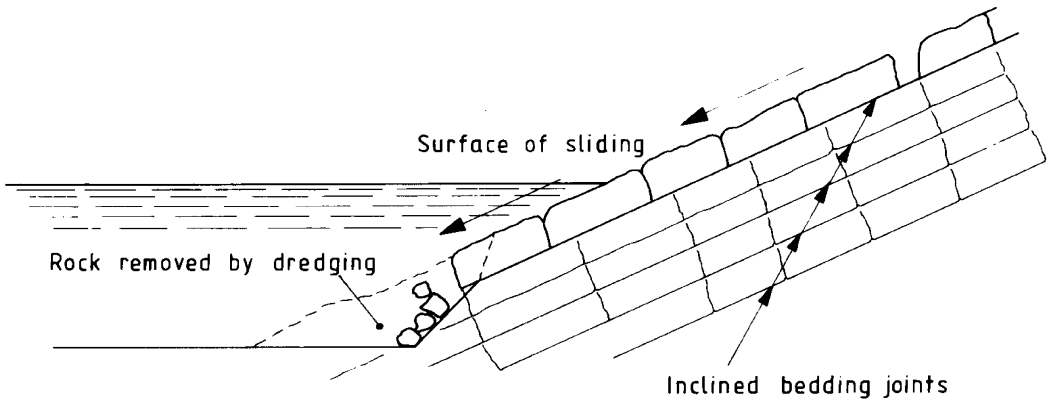

Figure 74 — Slab slide in rock

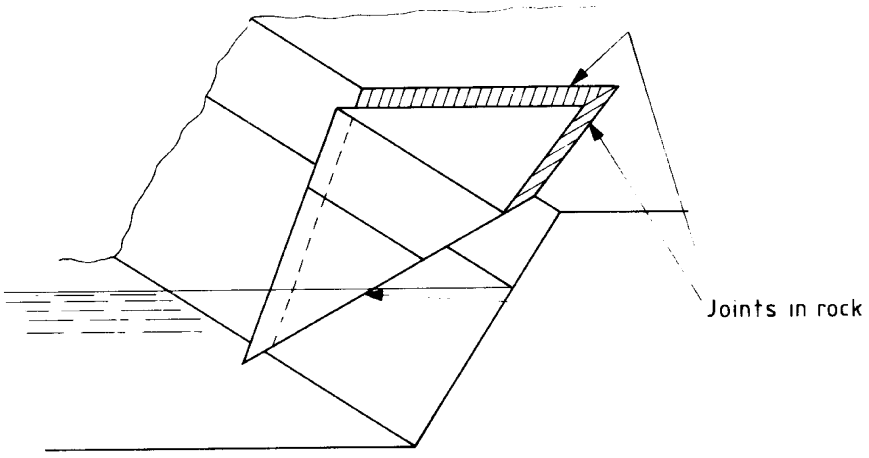

Figure 75 — Wedge failure in rock

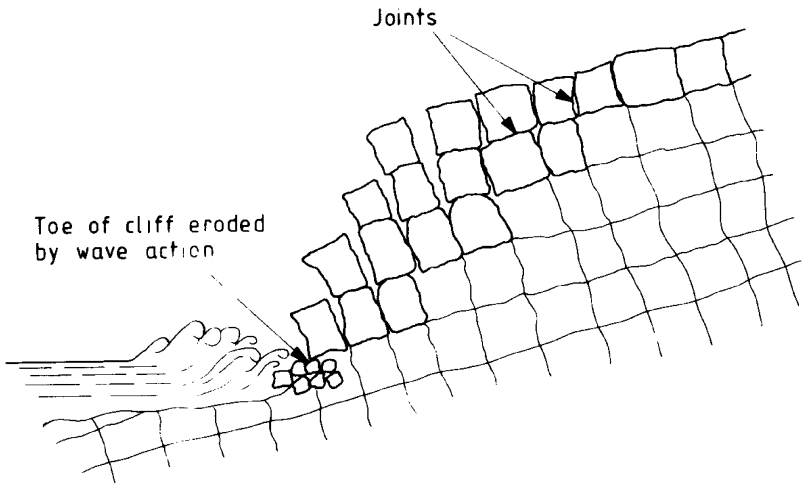

Figure 76 — Toppling failure in rock

54.3.2 *Factors of safety and risks of failure.* Guidance on suitable values for the safety factor of slopes is given in BS 6031. This guidance is essentially in relation to the stability of above-water slopes.

Consideration should be given to the consequences of underwater slope failure on maritime structures. A slip caused by dredging for a berth could result in collapse of a jetty installation or quay wall with loss of expensive equipment and revenue. Similarly blockage of a dredged channel could result in closure of a port. Mobilization of equipment and materials for remedial works involving dredging and restoration of profiles by dumping may be slow and the remedial work is costly in relation to the volume of material involved in a slip. Therefore a conservative approach should be adopted and a factor of safety of 1.5 should be allowed for first-time slides on underwater slopes where a good standard of site investigation has been achieved.

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Where embankments are constructed to form breakwaters on weak soils, the consequence of shear failure and subsidence of the embankment followed by overtopping by waves at times of storms should be considered in relation to the effects on harbour installations protected by the breakwater.

54.3.3 *Slope profile.* The required slope angles are obtained by the analytical methods referred to in **54.3.1** or by empirical methods. Angles flatter than those required from considerations of a minimum factor of safety may be desirable in order to avoid frequent maintenance dredging or in order to meet aesthetic criteria for above-water slopes.

Where underwater slopes are formed in erodible loose sands and silts the profile is likely to be governed by considerations of local steepening caused by erosion. The required slope profile is then established from local knowledge and experience based on the geometry of the underwater excavations and the presence of obstructions to flow, such as piles, moored ships, quays and the like (see **54.5**).

Where slopes are formed in layers of soil or rock of significantly differing characteristics the slope angles may be varied to conform to the engineering behaviour of each formation. Slope angles may also be varied in uniform pervious water-bearing soils by adopting a steep slope approaching the angle of repose of the soil located above the highest ground water level or the highest level affected by tides or uprush of waves and a flatter slope in the zone affected by varying tidal levels and wave action.

Where steep upper slopes are adopted, consideration should be given to the overall stability of the earthworks and where necessary a berm should be introduced between the two differing slope profiles.

In above-water slopes, a berm should be provided at the level of the interface between an impervious formation and an overlying water bearing soil. An open channel or piped drain may be provided on the berm to collect seepage from the upper slope. The surface of the berm should be sloped back to prevent water spilling down the lower slope at times of heavy surface water run-off.

A berm or other space should be provided at the toe of rock or steep earth cliffs to trap boulders or falls of soil from the face of the cliff where such falls would cause danger to persons or property.

Guidance on the required width of the berm or debris trap can be found elsewhere [46]. If insufficient space is available for the calculated width, a suitable fence or wall should be constructed along the outer margin.

The profile of the slope required for the face of a breakwater or training wall is governed not only by consideration of the factor of safety against failure in the underlying soil and consideration of differential water pressure within and on each side of the embankment but also by the need to avoid erosion and overtopping of the structure by wave action. Guidance on the design of breakwaters and training walls is to be found in subsequent Parts of this code.

54.3.4 *The effects of construction procedure.* The procedure adopted for dredging of berths and channels should not be such as to endanger the stability of slopes. In particular, the practice of dredging in a series of vertically-sided steps allowing the slope to slump to its natural angle of repose should not be followed if it results in a general weakening of the soil behind the slope such that the design profile cannot be maintained.

Where fill is placed on an existing slope for the purpose of reclaiming ground behind a wharf or quay wall or from the foreshore, care should be taken to avoid excessive surcharge by heaping up the fill on or beyond the crest of the slope in a manner which could cause instability or the formation of mud waves at the toe of the slope which could cause problems in removing the heaved soil from below water level.

Embankments and training walls may be constructed by dumping fill on to weak soils below the seabed in situations where a period of time can be allowed between successive stages of filling for the purpose of dissipating excess pore pressure. In such cases, care should be taken in the placing of the underwater fill to avoid local excessive surcharge. It may be necessary to place the main mass or core of the fill between outer embankments designed to retain the filling and to protect the core material against wave attack and erosion (see Figure 77). Where the outer protective embankments are placed in successive stages as shown in Figure 77, the height of each stage should be controlled to prevent the formation of mud waves which could become trapped within the core material and cause instability of the embankment.

The size of the stone or mattressing used to protect the retaining embankments should be such as to ensure stability in the worst storms expected during the construction period and subsequent to construction.

Any proposals for constructing embankments by end tipping from the shore should take account of the consequences of surcharge due to dumping material on to a steep slope and to erosion of the seabed soil beneath the advancing toe of the slope.

Where dredging or reclamation is undertaken in weak unstable soils, the effects of rapid pore pressure increase due to blasting or pile driving for associated works should be considered.

Suitable drainage measures should be taken to prevent accumulation of surface water or diversion of subsoil water on to areas at or near the crest of abovewater slopes if the resulting rise in pore water pressure would have an adverse effect on the stability of partly completed or completed earthworks.

Guidance on suitable forms of surface and subsoil drainage is given in BS 6031.

54.4 Monitoring stability. Where experience or stability analyses give reasonable assurance of stability, no special measures are required for monitoring stability. However, it is good practice to make periodic inspections during construction and in the early months following completion when regime conditions in the waterway are becoming established, and where grassing or other methods used to control erosion in abovewater slopes are attaining the desired conditions of stable growth.

Periodic inspections should include the following observations.

a) *Deformation.* Settlements in the upper part of the slope and bulging towards the toe may indicate incipient failure by a rotational shear slide (see **54.2.2**).

b) *Cracking.* A series of cracks parallel to the crest and parallel ridging towards the toe may indicate incipient translation failure on above-water slopes (see **54.2.4**). Hexagonal or random pattern cracking indicates drying shrinkage of cohesive soils.

c) *Fissuring.* Opening of joints and fissures in a rock slope may indicate incipient translational failure (see **54.2.4** and **54.2.5**) or toppling failure (see **54.2.9**).

d) *Seepage.* Water carrying soil particles seeping from a slope is indicative of internal or seepage erosion (see **54.2.10**).

e) *Gullying.* Channels eroded on a slope face indicate the need for protection against surface erosion.

Inspection should be made after storms, periods of heavy rain, snow or severe frost. Clay slopes should be inspected during or immediately after rainfall or wave attack following a period of dry weather to assess the effects of water entering surface cracks.

Inspection of the position and inclination of pegs or beacon poles driven into a slope is a simple means of detecting gross deformations.

Careful inspection of steeply cut temporary slopes for foundation excavations or trenches is required to ensure safe working conditions for operatives and to avoid damage to partly constructed works or existing structures adjacent to the excavation.

Where there are doubts concerning the short or long term stability of earthworks, it may be desirable to install instrumentation to give warning of incipient instability, enabling suitable remedial measures to be undertaken before the stage of failure is reached. Guidance on instrumentation suitable for above-water earthworks is given in BS 6031.

Instrumentation, including apparatus for pore water pressure observation, suitable for installation in earthworks above high-water mark is unlikely to be practicable for underwater slopes, particularly in areas where access for vessels is required. In these areas monitoring may need to be limited to detecting deformations by taking soundings or making observations on beacon poles.

54.5 Slope protection

54.5.1 *General.* Measures should be taken to ensure the overall stability of the slope against the modes of failure referred to in **54.2**. Consideration should then be given to the need to protect the surfaces of slopes and waterways against erosion by currents, waves, surface and subsoil water.

54.5.2 *Underwater slopes.* Erosion of the soil on the seabed is caused by disturbance and transport of soil particles by flowing water (see clause **14**). Erosion may be aggravated by oscillatory currents produced by waves and by vortices caused by obstruction to uniform flow.

Consideration should be given to the effects on vortex formation of the geometry of slopes formed by dredging schemes for berths and navigable channels or by reclamation from the foreshore. Sharply projecting spurs or re-entrant slopes should be avoided. Conditions giving rise to severe scour can occur in the presence of moored ships, as a result of restriction in the area of flow alongside and beneath the hull and vortex formation at the bow or stern. Moving ships can cause wave action due to the bow wave or propellor wash. Deep scour can occur around obstructions to flow such as piles or the protecting corners of quay walls. Protection of the seabed in the form of dumped rock or prefabricated mattresses may be needed in these areas if the currents are strong. Guidance on the design of such forms of bed protection is to be found in subsequent Parts of this code.

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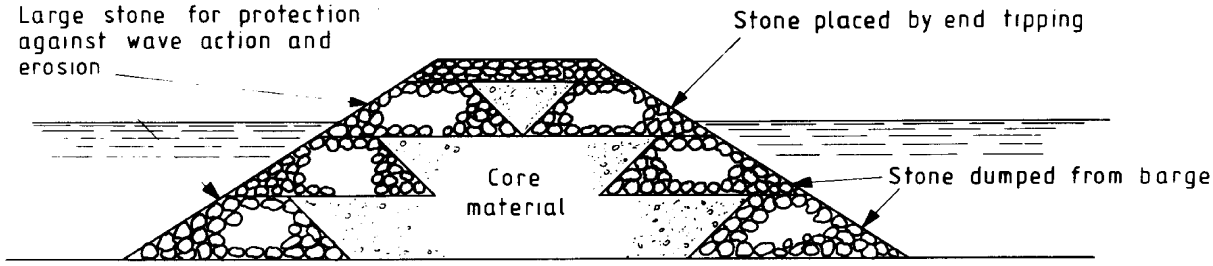

Figure 77 — Embankment built in stages with core material protected by dumped stone

54.5.3 *Above water and partly submerged slopes.* Agencies causing erosion and instability of slopes within the influence of the rise and fall of tides and wave action are as follows:

a) currents with associated vortex formation as described in **54.5.2**;

b) scour by waves and wash from ships;

c) movement of soil particles due to the egress of water on a falling tide or retreat of waves;

d) egress of subsoil water;

e) action of winds;

f) action of surface water.

Where protection of the surfaces against any of the above influences is provided by means of a layer of rock or precast concrete blocks on the slopes, consideration should be given to the effects of a varying water pressure on each side of the protective layer. Protection against wave action and severe scouring conditions may require the provision of large blocks of rock or special precast concrete moulded shapes to absorb and dissipate wave energy. Such cases require the provision of means to prevent the flow of finer soil particles into the interstices of the large units. These may consist of a blanket of filter material interposed between the rock or precast concrete armouring and the soil forming the slope. The filter should be designed to prevent the movement of the finest particles from the soil under the influence of water flowing out of the slope on the falling tide or on retreat of waves. Because of the large interstices between the blocks forming the armouring to the slope, the filter should consist of several layers graded from coarse to fine material, each layer being designed so that the finest filter material will not move into the adjacent coarser filter layer under the influence of flowing water. Alternatively, the filter may consist of other materials, including plastics fibre mesh or brushwood mats protected by a layer of crushed and graded stone and then the stone or precast concrete block armouring (see Figure 78). Guidance on the design of single or multi-layer filters is given elsewhere [47].

Ground surfaces beyond the influence of waves or tidal water movements may be protected against the erosive effects of winds and surface water by blanketing with stone, by paving with concrete or with bituminous materials or by vegetation.

Guidance on the design of protective measures is to be found in subsequent Parts of this code.

54.6 Maintenance of earthworks. Guidance on inspection and maintenance of above-water slopes is given in BS 6031.

In the zone subject to wave attack, water that cannot drain immediately will have a disruptive effect on the structure.

The design of any filter layers, for maintenance and during construction, therefore demands careful attention.

Maintenance of underwater slopes, other than dredging in areas of accretion, is not possible. However, periodic soundings should be taken in order to determine the trend of changes in seabed levels following the construction of maritime works and enable the appropriate remedial action to be taken before major instability develops.

54.7 Remedial works. Guidance on treatment to remedy failure of above-water slopes is given in BS 6031.

In the case of submerged slopes, the type of remedial treatment adopted will depend on the consequences of failure and the effects of remedial work on navigable depths in berths and channels. Thus, dumping rock fill at the toe of a partly submerged slope to apply a counter-weight against a rotational shear slide would not be feasible if this prevented access to a berth. In such a case the alternative of restraining the slip by anchored sheet piling would need to be considered. Toe weighting should, however, be considered as a temporary measure if the slide endangers the stability of important shoreside works.

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Where a rotational shear slide is caused by scouring of seabed material from the toe of a slope it should be feasible to dump rock into the scour hole without adverse effect on navigation. The desirability of laying mattresses on the seabed to control further erosion around the affected area should be considered.

Adjustment of the slope profile by removing material at the crest of the slipped area may be a feasible remedial treatment for submerged or partly submerged slopes.

Consideration may also be given to providing groynes or training walls to control scour and encourage accretion in areas where there is a trend towards increased scour which could result in instability of slopes.

55 Use of thixotropic liquids in excavations

55.1 Types of structure. Thixotropic liquids, referred to as bentonite slurry throughout this clause, are used as a means of stabilizing excavations in weak ground for shoreside structures such as quay walls and lock chambers, thereby avoiding the need for support by timbering or sheet piling.

Trenches for the foundations and walls of these structures are supported, during excavation, by the pressure from the slurry. Concrete is then placed through a tremie pipe thus displacing the slurry which is then reused or discarded.

Substructures for quay walls and locks may consist of a series of rigid gravity walls in box form, or counterfort retaining walls consisting of a continuous facing wall forming the waterside or berthing face and a series of counterforts projecting from the landward side of the wall. Quay walls may also be in the form of buttress walls with rectangular or arched vaults forming the earth-retaining members.

Foundations in weak ground for heavy shoreside structures such as fixed cranes or silos may be constructed on rectangular or cruciform trenches excavated with the support of bentonite slurry. Foundations of this type are known as barettes.

Bentonite slurry may also be used as a lubricant to assist the sinking of caissons used for the construction of quay walls or berthing structures.

In all cases the excavation is made from the ground surface above the highest level of the ground water table. Guide walls are constructed to retain the slurry in permeable ground above the water table and to assist maintenance of vertical and horizontal alignment on the concrete substructure.

Where quay walls and the like are sited on the foreshore or in tidal waters it is necessary to place fill to form a temporary working surface above the tidal height and to accommodate the guide walls. The filled areas should be protected from erosion on the seaward side by dumping rock fill or by temporary sheet piling.

55.2 Lateral earth pressure and earth resistance. Earth-retaining structures constructed using bentonite slurry techniques have a measure of rigidity and in the case of massive counterfort or buttressed walls on an unyielding foundation, consideration should be given to the adoption of at-rest conditions for the calculation of earth pressure. Cantilevered or tied-back retaining walls may be designed for active earth pressure conditions or for a state intermediate between the active and at-rest state depending on the rigidity of the structure and the amount of yielding expected in the anchors or at the toe of the structure. For anchored walls consideration should be given to the effects of arching of the soil on the distribution of earth pressure (see **51.3**).

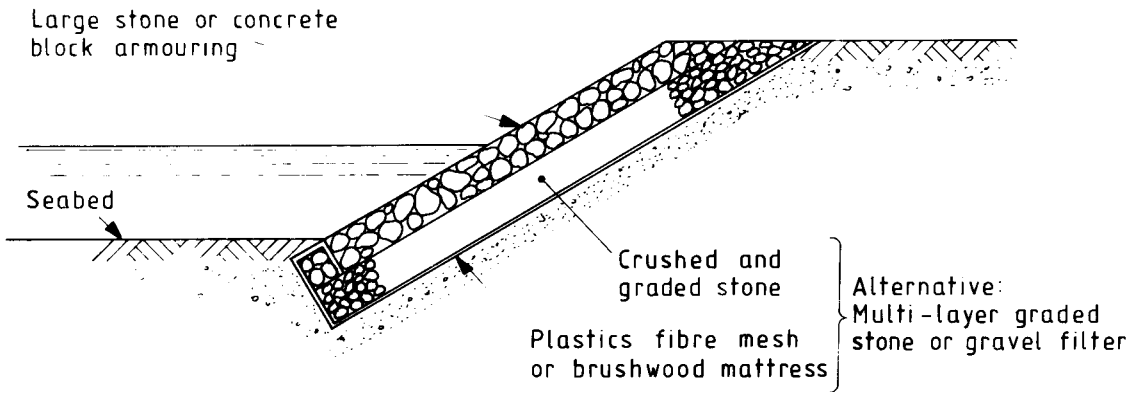

Figure 78 — Slope protection by rock or concrete armouring backed by filter layer

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The concreting of slurry trench wall panels in a single operation results in an increasing intensity of the interface pressure between the unit and the soil towards the bottom of the panel, effectively increasing the resistance of the panel to horizontal movement or rotation, through the ground, under pressure from retained backing. This effect enables panels to be set normal to a screen wall (as buttresses or fins) to provide anchorage to the structure and, particularly if associated with a relieving platform, provides self stability without the need for horizontal ground anchors.

This increase in interface pressure from the head of wet concrete may also encourage the development of horizontal arching in the ground behind the wall.

Adequate keyed joints should be provided between adjacent panels or units to ensure uniformity in distribution of earth pressure and earth resistance over the full length and depth of the structure.

55.3 Design of excavations for support by bentonite slurry. The vertical face of a trench excavated under bentonite slurry is supported by the hydrostatic pressure of the fluid which is designed to have a density higher than that of the surrounding ground water. The general principles which govern the stability of slurry-supported trenches are reviewed elsewhere [48]. It is essential to maintain a head of slurry in the trench above the level of the ground water table with an adequate margin of safety to provide for a rise in ground water level at the time of high tides or surges.

Temporary lowering of the slurry level due to overbreak in the excavation or loss of fluid to seepage through the soil or through interstices in open gravel or fill material should also be provided for.

The walls forming an earth-retaining structure or a foundation should be constructed in alternate short lengths or panels. Excavation in intermediate panels should not be commenced until the concrete placed in the adjacent panels has attained sufficient strength to prevent damage by the excavation equipment.

The length-to-width ratio of the panels should not be so great as to reduce appreciably the arching action of the soil surrounding the excavated trench. The dimensions of the panel are also governed by the capacity of the available concreting plant, bearing in mind that the concrete has to be placed in a continuous pour.

In addition to the ratio of the length-to-width of the panel, the stability of the trench excavation is also governed by the depth of the trench and the density of the slurry. A method of calculating the stability has been published by Aas [49]. Consideration should also be given to the dimensions of the excavating equipment in relation to the length and width of the panel, in order to avoid erosion and abrasion of the soil as the equipment is lowered down or raised from the trench.

If a heavy slurry is needed to maintain stable conditions, it may be necessary to reduce the density immediately before placing concrete in order to permit the concrete to flow freely from the tremie pipe along the base of the trench. This is particularly necessary in the case of a load-bearing wall, in order to avoid the base of the wall bearing on a layer of thick slurry contaminated with soil particles.

The density of the slurry should be considered in relation to the permeability of the soil forming the walls of the trench. The density should be such as to allow a filter-cake to form on the trench wall which will permit excess water to bleed from the slurry to the soil without loss of solid constituents of the bentonite. In highly permeable soils or fill material containing voids it may be necessary to extend the guide walls below the permeable zone or to use additives such as Portland cement in the slurry, until such time as the thickened slurry has become stabilized within the larger voids of the ground, when the slurry in the trench may be replaced by a thinner fluid.

Periodical checks should be made on the density and other properties of the slurry to ensure that it does not become flocculated or excessively diluted by ground water or contaminated by soil particles to the extent that instability could occur in the trench or deterioration take place in the supporting capacity of the ground immediately beneath the base of the completed substructure (see **55.4**).

55.4 Materials. The constituent materials and methods of mixing and testing of the bentonite slurry should conform with the recommendations of current specifications issued by recognized specialist organizations.

NOTE In the absence of a relevant British Standard specification, guidance on the specification of materials and methods of mixing and testing of bentonite slurry may be obtained from the Federation of Piling Specialists, Dickens House, 15 Took's Court, London EC4A 1LA.

Attention should be paid to the effect of saline water on the properties of the slurry and where necessary special forms of bentonite or processing methods should be used to prevent undesirable flocculation of the slurry in the trench.

Measures should be taken to prevent discharge of used slurry or uncontrolled escape of slurry from the trench to adjacent waterways, watercourses or sewers where these discharges would result in pollution or blockage of drainage systems.

Section 7. Materials

56 General

The materials covered by this section include the basic materials used in civil engineering construction and composite or manufactured materials where these are generally considered to be materials in their own right.

Some elements of maritime construction such as pavements and piling which may utilize a variety of materials are also included because the special requirements of the maritime situation have a bearing on the choice of materials which are the most appropriate.

The materials covered in this section are as follows:

a) stone for armouring or protection works;

b) concrete;

c) structural steel and other metals;

d) timber;

e) piles;

f) pipes;

g) pavements;

h) rail track;

i) brickwork, blockwork and masonry;

j) bituminous materials;

k) rubber and plastics.

Protective measures and treatments cover a wide range of methods which can be applied in the construction, operation and maintenance of maritime structures. Those covered in this section include the following:

1) coating systems;

2) concrete protection;

3) Monel 400 sheathing;

4) steel wear plates;

5) wrappings;

6) cathodic protection.

57 Stone for armouring or protection works

57.1 General. Natural stone has been used traditionally in the construction of protection works in marine conditions and is used in such structures as breakwaters, training walls, groynes and in pitched and revetted slopes.

Stone for protection works should be hard and of best quality, with good durability. It should be free from laminations and weak cleavage planes and should be of such a character that it will not disintegrate or erode from the action of air, water, wetting and drying, freezing and thawing and impact due to wave action. It should be capable of being handled and placed without undue fracture or damage. Individual pieces should be prismoidal in shape and the maximum dimension of stones should generally not exceed twice the minimum dimension and should never exceed three times.

In considering alternative sources of suitable stone, the first choice should be igneous rocks which are generally the most durable. Sedimentary and metamorphic rocks require more care in selection but the best sources can provide suitable materials.

57.2 Tests of quality. In determining the quality of rock available for use in protective structures, a number of methods of testing and assessment are available. The tests listed below are intended for the primary armour stone and suggested acceptable results are given. Lower standards of quality may be acceptable for stone to be used in layers other than the primary armouring. Except where noted, the test methods should be as described in BS 812.

a) *Apparent relative density.* A minimum value of 2.6 is desirable but lesser values are acceptable because armour stone size can be adjusted to compensate.

b) *Water absorption.* Value should be not more than 3 % (*m*/*m*).

c) *Aggregate impact value.* Value should be not less than 30 % for standard test fraction.

d) *Ten per cent fines.* Value should be not less than 10 t.

e) *Soundness.* Loss in mass after 5 cycles should be not more than 12 % for sodium sulphate or 18 % for magnesium sulphate, by the test method given in Appendix B. Where necessary, a representative sample of large stone should be crushed to provide sizes acceptable for testing by this procedure.

f) *Aggregate abrasion value.* Value should be not more than 15 %.

Of the above tests, the soundness test is the most important.

57.3 Specification of size

57.3.1 *General.* A number of methods of varying precision have been in common use for specifying the size and grading of stone and in order to provide a common standard the methods described in **57.3.2** and **57.3.3** are recommended.

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57.3.2 *Single-sized materials.* Single-sized materials generally refer to large stones and should be specified as individual stones having a stated nominal mass (in tonnes). All stones should be required to fall within stated upper and lower percentage tolerances of the nominal mass and a stated percentage of the number of all stones should be required to exceed the nominal weight. At least 75 % (m/m) of the stone in each grade should consist of stones within the top half of the range specified.

57.3.3 *Graded materials.* Graded materials include riprap, bedding, filter and core materials.

Riprap should be well graded and should be specified as falling between stated maximum and minimum masses. Riprap is frequently specified in terms of its median mass m_{50} and the typical figures for maximum and minimum masses of material are 3.6 m_{50} and 0.22 m_{50}, respectively.

Bedding and filter materials are both required to prevent the scouring of one material through another. Well graded materials are required for this purpose and maximum and minimum sizes with a number of grading points should be specified according to the materials lying both above and below the filter or bedding layer. The size and grading characteristics are normally specified by stating values for the median size, D_{50} and the ratios D_{85} to D_{50} and D_{15} to D_{50}.

The core of a protective structure has to be capable of achieving a relatively high density without compaction when dumped in water. Scour of the core material by being washed out through overlying layers has to be prevented by correct specification of the sizes of the respective materials.

58 Concrete

58.1 General. BS 5328 covers the methods for specifying and producing concrete as a constructional material up to the point where it is placed in the construction. BS 8110 covers the structural use of reinforced and prestressed concrete both cast in situ and precast and also gives recommendations for concrete materials, specification and construction.

In this clause the supplements and amendments, to the requirements of BS 5328 and BS 8110, which are necessary in maritime conditions are given.

During construction, maritime works are particularly sensitive to adverse weather conditions which may prevent use of floating plant, hinder access to the site and cause damage to work well above high-water level and while durability of concrete in maritime structures is largely dependent on the specification of appropriate materials, mix design and workmanship it has to be emphasized that long term durability depends on an understanding of the various ways in which damage and degradation can occur. A balance between the many inter-related factors involving materials, design, construction and costs can then be achieved. Some of the factors affecting the choice of construction whether, in situ, precast or a combination of the two, are as follows.

a) The work may be permanently underwater where access is difficult and visibility is negligible.

b) The work may be within reach of waves at every tide and therefore subject to scour.

c) The periods between tides in which the work is accessible may be very short and it may be necessary to work during low tides at night as well as during the daytime.

d) Temporary works should be simple and capable of rapid erection during tidal access and have to be strong enough to protect immature concrete and resist high temporary loadings in adverse weather.

e) The shape of concrete structures and members should be such that these can be formed by simple shuttering which can be easily fixed and will be grouttight, rigid and strong. Complicated shapes should be avoided, as should thin cross sections and cross sections in which the cover to reinforcement is sensitive to the accuracy of steel fixing.

f) Steel reinforcement should be carefully detailed so that it can be rapidly and accurately fixed while having adequate rigidity to resist displacement during placing and compaction of the concrete. Choice of the appropriate cover to reinforcement and ensuring that it can be achieved in situ are important.

g) Reinforcement should be designed to limit cracking of concrete due to thermal and drying movements as is the practice for water-retaining structures. It is not necessarily correct to provide excessive cover as this may give rise to increased surface cracking.

h) Erosion of concrete due to abrasion can be a cause of serious damage to maritime structures. It may occur at water level with floating objects or at bed level where beach materials may continually be washed against the structure.

i) In cold climates, freezing and thawing and impact of ice can affect concrete adversely.

j) Concrete can suffer chemical attack.

58.2 Chemical attack. The durability of reinforced concrete depends fundamentally on the quality and impermeability of the concrete in preventing steel reinforcement from corroding. Normally concrete provides an alkaline environment which passivates corrosive reactions. If this breaks down and corrosion occurs, the structure not only loses strength due to loss of steel but the concrete is spalled away due to the volume increase of the corrosion products, thereby increasing the exposure of the steel and increasing the rate of breakdown. The presence of salts in seawater makes maritime conditions most aggresive to steel reinforcement and therefore it is vital that the alkaline environment be preserved. Chlorides present in seawater pose the main threat of corrosion to reinforcement. In the presence of moisture and oxygen, local concentrations of chloride ions in the concrete can set up electrical potential differences which can promote serious corrosion. Alternate wetting and drying with salt water encourages salts to be deposited as crystals in concrete and high concentrations can build up internally in porous zones. The splash zone is the area where this effect is most severe.

The best method of ensuring that the natural alkaline protective mechanism is maintained is by providing concrete which has the lowest possible permeability and this can be obtained by adopting mixes designed to produce a concrete which readily compacts to its highest practicable density. Care should be taken that requirements for high strength and low permeability do not result in practice in the workability being restricted to less than that required for adequate compaction.

Substances more aggresive than seawater may be brought into contact with maritime structures by wave and current action. In such cases measures required to provide protection depend upon the nature of the substances involved and in extreme cases may include the use of impermeable surface coatings.

Sulphate ions are present in seawater at relatively high concentrations and can react with the tricalcium aluminate (C_3A) in hardened Portland cement. The rate of attack, which is greatest in warm or polluted waters, can be reduced by limiting the proportion of tricalcium aluminate in Portland cement. In UK waters a maximum of 10 % is recommended, when determined by the method described in BS 4027. The tricalcium aluminate content should not be less than 4 % in order to avoid attack of steel reinforcement by chlorides. Alternative approaches would be either to replace at least 25 % of the Portland cement with pulverized-fuel ash (pfa) or to replace at least 70 % of the Portland cement with ground granulated blastfurnace slag (ggbs). Either a blended cement may be used or the substitution may be made at site provided the blended cement or the individual components comply with the relevant British Standard.

The adoption of the above limits on tricalcium aluminate content in Portland cement should only be used as an added precaution and not as a substitute for concrete of low permeability which is a fundamental requirement irrespective of cement type. Further guidance is given in BS 5328.

Alkali-aggregate reaction, although rarely experienced to date in the British Isles, is a problem which is not unknown to maritime works. Its effects can be very damaging and it may be necessary to take precautions to prevent it. Reference should be made to BRE Digest 330 and to BS 5328.

58.3 Specification. Except where supplemented or amended by the provisions of clause **58**, the specification of materials for concrete in maritime structures should comply with BS 5328 and BS 8110.

58.4 Constituent materials

58.4.1 *Cement.* The cements to be used should comply with the British Standards given in Table 20A subject to the stated limits on tricalcium aluminate content.

The use of sulphate-resisting Portland cement (as specified in BS 4027) is not recommended for reinforced concrete in contact with seawater because it has a maximum tricalcium aluminate content of 3.5 %. If sulphates in ground water dictate its use it may be used in plain concrete.

There is insufficient long-term experience in maritime structures of those cements listed in BS 5328, but not listed in Table 20A, to provide guidance as to their use.

The use of high alumina cement manufactured to comply with BS 915 should be strictly confined to non-structural uses such as in screeds designed to protect concrete from aggressive substances.

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Table 20A — Cements

Type	Standard	Other requirements
(a) Portland cement		
Ordinary Portland cement	BS 12	Tricalcium aluminate content
Low-heat Portland cement	BS 1370	by weight not less than 4 % and not more than 10 %
b) Cement containing ggbs or pfa		
Portland blastfurnace cement	BS 146	
Low-heat Portland blastfurnace cement	BS 4246	
Portland pulverized-fuel ash cement	BS 6588	
c) Combinations of Portland cement with ggbs or pfa	Combination should comply with	
1) Ordinary portland cement (BS 12) in combination with ggbs complying with BS 6699	BS 146 or BS 4246	
2) Ordinary Portland cement (BS 12) in combination with pfa complying with BS 3892-1	BS 6588	

In order to minimize cracking at an early stage, when member thicknesses exceed 300 mm and cement contents exceed 350 kg/m^3, consideration should be given to methods of avoiding excessive temperature differentials due to heat of hydration. These may include partial cement replacement methods and the use of appropriate construction techniques.

58.4.2 *Aggregates.* Aggregates should comply with BS 882.

Lightweight and other aggregates not complying with BS 882 should only be used if there is satisfactory data on the properties of concrete made with them.

It is unlikely that lightweight aggregates would be satisfactory where resistance to abrasion is required.

When high strength concrete is required, the sources as well as the type and characteristics of the aggregate need careful selection based on the results of trial mixes.

Aggregates should pass the soundness test given in **57.2** e).

Porous aggregates should only be used if their use is unavoidable. The maximum absorption permitted should be 3 % as measured by the procedure described in BS 812.

Special care is required in the selection and quality control of aggregates of marine origin to ensure that the limits set by BS 5328, on the amount of chloride ion in the concrete, are not exceeded.

If severe abrasion of the concrete by pebbles or sand is expected, the coarse aggregate should be at least as hard as the material causing the abrasion and the fine aggregate content of the mix should be kept as low as is compatible with the production of dense and impermeable concrete.

58.4.3 *Water.* Water should be clean and free from harmful matter and, where tests are required, these should be as described in BS 3148. Seawater should not be used in reinforced concrete or grout but it can if necessary be used in mass concrete. Seawater should not be used with chloride accelerators.

58.4.4 *Admixtures.* Any admixture used should be one of those permitted by BS 5328. If admixtures are used, laboratory and production batch tests should be performed to demonstrate their properties and their capability with the cement to be used. Their possible effects on the durability of the concrete should be considered carefully.

The use of air-entraining agents in the concrete should be considered for the splash zone and for any other part of the structure which might be subjected to freezing and thawing when in a saturated condition. Controlling the required amounts of air is often difficult in concretes where the cement content exceeds 400 kg/m^3. Where this is so, long term durability may be obtained with more confidence by the use of a uniform well compacted concrete.

58.4.5 *Pozzolanas.* Pozzolanas may be used provided that tests are made to demonstrate their properties and their compatibility with the cement to be used. Their possible effects on the durability of the concrete should be considered carefully.

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58.5 Hardened concrete. The minimum requirements for the strength and durability of the concrete in the hardened state should be determined from consideration of the limit state requirements of section 2 of BS 8110-1:1985, but if, in addition, a special property or a particular surface finish is required, these minimum requirements may have to be considerably exceeded.

The grade of concrete required will depend partly on the particular use and the characteristic strength needed to provide the structure with adequate ultimate strength and partly on the exposure conditions and the cover provided for any reinforcement or tendons.

The cement content should not be less than the minimum given in Table 21 depending upon the exposure condition.

The water content should be the minimum required to give adequate workability of the concrete. Concrete with greater cement content that given in BS 8110 will exhibit greater thermal movement than is allowed for in that code and when reinforced will require proportionately more crack-control reinforcement.

58.6 Fresh concrete. In accordance with **6.3** of BS 8110-1:1985 special variations in mix design may be necessary to overcome deficiencies in the characteristics of local aggregates, i.e. flakiness, elongation, porosity and alkali reactivity. In many locations the shape of aggregates may present problems in obtaining adequate workability at reasonable cement and water content and it is not uncommon for aggregates to have to be brought considerable distances where local materials are inadequate for maritime structures.

Requirements of concrete mixes, the required arrangements and conditions for the production of concrete, and quality control and conditions for compliance of the concrete with specified requirements should be in accordance with BS 5328. When air-entrained concrete with a cement content of more than 350 kg/m^3 is specified, trials should be made to ensure that the required strength, compaction and surface finish can be achieved.

58.7 Curing. The minimum periods of curing and protection recommended in BS 8110 should be increased by 50 % to ensure low permeability.

Plain concrete may be cured with seawater.

58.8 Reinforcement. Reinforcement should comply with BS 4449, BS 4461, BS 4482 and BS 4483. Cutting and/or bending of reinforcement should comply with BS 4466. The reinforcement should not be surrounded by concrete unless it is free from salt deposits. Where, during the construction process, exposed reinforcement is immersed in seawater, it should be checked for corrosion other than surface rusting and cleaned if necessary.

58.9 Prestressing tendons, sheathing and grouting. The use of prestressing steels, sheathing, grouts and procedures to be taken when storing, making up, positioning, tensioning and grouting tendons, should be in accordance with BS 8110 or BS 5400.

Table 21 — Minimum cement content

Exposure conditions	Reinforced		Plain	
	Minimum cement content	Maximum ratio of free water/cement	Minimum cement content	Maximum ratio of free water/cement
	kg/m^3		kg/m^3	
Internal use only or protected from direct exposure to marine atmosphere	275	0.65	180	0.80
Continuously submerged in sea water up to a level of 1 m below lowest low water	350	0.50	300	0.60
Directly exposed to sea water spray or in tidal zone down to 1 m below lowest low level	400	0.45	350	0.50
Subject to severe abrasion	400	0.45	350	0.50

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58.10 Prevention of reinforcement corrosion. Cover in maritime structures should be preferably 75 mm but not less than 50 mm. Cover to reinforcement refers to the minimum distance from the surface of any steel reinforcement links, tendons, or sheath to the surface of the concrete. The thickness of binding wire is not included but ends of binding wire should be turned away from the cover zone and into the body of the member (see also **58.1**).

Where problems exist due to difficulties of access, visibility, etc., cover may be increased to provide a safety factor to cover wider tolerances of assembly but the element concerned should be designed with controlled crack widths in accordance with BS 6349-2. It should be remembered that the provision of excessive cover might not be beneficial if it results in reduced control over cracking. However, in seawalls and similar structures subject to severe abrasion, up to 300 mm of cover should be provided to give adequate life.

Where the risk of reinforcement corrosion is likely to be extreme and particularly in circumstances where the natural protection provided by high quality concrete cannot be ensured, e.g. where aggregate quality is unreliable, consideration may be given to the use of reinforcement protected by fusion bonded epoxy coating to ASTM A775 or hot-dip galvanizing, which should comply with BS 729. It should be noted that galvanized and nongalvanized reinforcement should not be interconnected.

If cathodic protection is applied to superimposed or adjacent steelwork, the dangers of corrosion of reinforcement or prestressing steel in the concrete structure should be borne in mind. If an impressed current system is to be used, care should be taken to prevent stray currents from causing such corrosion.

Cathodic protection should be in accordance with CP 1021.

58.11 Underwater concrete. Underwater concreting can be carried out in situ either by placing conventionally mixed fresh concrete by the methods listed in this subclause or by the process of pumping cement grout into previously placed aggregate.

The principal methods of placing fresh concrete underwater are by the following:

a) tremie;

b) skip or similar device;

c) pumping;

d) bagwork.

A characteristic strength not greater than 25 N/mm^2 should be used for design purposes for concrete placed under water irrespective of the cement content unless it can be shown that a higher value can be achieved.

Concrete bagwork should be limited mainly to temporary work.

Concrete can also be placed underwater as precast units which may be used either as structural units in themselves or as permanent shuttering for underwater concrete placed fresh.

Plain concrete can generally be placed underwater but reinforced concrete should only be used where absolutely necessary because of the difficulties of ensuring sound results and the problems of inspection.

There is a risk of cement being washed out when concrete is placed underwater and this should be reduced by keeping the area of the concrete in direct contact with the water to a minimum and by avoiding any agitation. It is also necessary to prevent materials so denuded of cement from being trapped in the body of the pour. For these reasons, the concrete is required to be cohesive in texture and with flow properties such that the pour can be expanded progressively as concrete is injected into its centre. Good workability and freedom from segregation are therefore required. Compaction by vibration is generally impracticable underwater.

Materials for underwater concrete should comply with the same general requirements as for normally placed concrete which is to be subject to the same environment. Because of the requirement for high workability, rounded aggregates are preferred and crushed fine aggregates should be avoided because their particle shape and grading are usually unsuitable. Workability with poor aggregates can sometimes be improved by increasing the content of fines with imported materials or by increased cement content. Admixtures can be used with advantage to improve the properties of the fresh concrete and to improve long term durability.

The usual principles of mix design apply to underwater concrete. Maximum aggregate size should be as large as possible, consistent with the need to avoid obstructing flow in delivery pipes. In tremie work, maximum size should only exceed 40 mm in large pours when a large diameter tremie is used. Concrete should be wet by normal standards and a slump of 125 mm is necessary when measured in the dry.

Mix proportions may be adapted from those which are known to give the required strength at the workability appropriate for concreting in the dry by increasing the cement content by 25 %. This enables adequate properties to be achieved with increased workability and offsets any losses of cement due to washing out. Mixes should normally have a cement content of at least 350 kg/m^3.

So far as possible, horizontal construction joints should be avoided but, where necessary, suitable surface preparation is essential before placing the next lift. Laitance which forms on the top of pours should be cleaned away by hacking and wire brushing and be hosed off to leave a clean hard surface.

In the grouted aggregate method of placing underwater concrete, the forms are filled with coarse aggregate with sand-cement grout being injected to fill the voids completely. Patented processes are usually employed and it is important that adequately experienced personnel should be used to carry out the work.

Grouted aggregate concrete can have advantages over fresh concrete placed underwater when:

1) the physical conditions of access make the deployment and control of skips and tremies difficult;

2) the velocity of flowing water is such that the risk of losing cement from fresh concrete is high, because water flow is much reduced within the body of aggregate and the colloidal grouts used do not dilute easily without mechanical agitation;

3) reinforcement is required which would tend to impede the flow of tremied concrete or skip work.

Disadvantages of the grouted aggregate process are that:

i) there is a danger that the aggregate filled form may become contaminated with silt or algae before grouting;

ii) special plant and specially trained supervisors are required.

For grouted concrete, coarse aggregate should be such that not more than 10 % passes a 40 mm sieve. Maximum stone size is limited only by the need to economize on the quantity of grout required. Aggregate should be clean but its shape characteristics are not important. Normal concreting sand is suitable for grout. The grading is important and will depend on the particle shape of both coarse and fine aggregates. Field tests should be carried out with the selected sand. Ordinary Portland cement is normally used. Admixtures may be advantageous. PFA may be used to improve fluidity of the grout but its quality is important.

Grout should be mixed in a special mixer capable of producing a grout with colloidal characteristics before it is pumped into place. The equipment should include screens to prevent large particles entering grout pipes. Mix proportions should be about 1 part cement to 1.5 parts of sand by mass. Pozzolanas or ggbs should be regarded as partly replacing cement providing a delay in the time for achieving strength is acceptable. Water to cement ratios are varied by the use of admixtures.

58.12 Other applications of concreting materials. There are a number of methods of constructing or repairing structures which fall into specialized categories and are usually carried out by firms having the appropriate expertise. The general difficulties of access and aggressive environment experienced in maritime works should be taken into account when these processes are considered for application to both repairs and new construction and the basic requirements for maritime concrete should be considered when materials for the application are selected.

Sprayed concrete is widely used for the repair of reinforced concrete maritime structures but a hand process which pays special attention to the cleaning and treatment of corroded steel reinforcement may be preferred.

Pressure grouting using Portland cement based grouts is used in a large number of ways in maritime works in addition to the application described in the grouted aggregate method of placing underwater concrete (see **58.11**). It is used in the stabilization of earthworks and foundations both for temporary works and as part of the permanent works of projects. Pressure grouting also has many applications in the repair of maritime structures.

59 Structural steel and other metals

59.1 General. The most commonly used metals in maritime structures are weldable structural steels which should comply with BS 4360. Other metals which are sometimes used in maritime structures, usually for special reasons, are copper alloy steels, stainless steels, cast steels, cast irons, wrought irons, brasses, bronzes, aluminium alloys and Monel (see **59.4**).

All metals suffer more from corrosion in a maritime environment than on shore. This is mainly due to:

a) the formation of galvanic cells, with the metals of the structure acting as anodes and cathodes and the solution of salts in the seawater acting as the electrolyte, particularly in the splash zone;

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b) the steady erosion of the corrosion products, such as rust in the case of structural steels, by wave action and cyclic deflections, especially if the structure is designed to absorb energy by deflection;

c) the inadequacy of preventative maintenance.

Corrosion varies according to the locality but typical values for different exposure zones are given in Table 22.

The need to control corrosion and to predict losses of metal due to corrosion plays a major part in the selection of the most suitable metal. As an example of this, it may be preferable to choose grade 50 steel to BS 4360 used at the stress levels of grade 43 steel to BS 4360. This will yield a thicker section with a greater allowance for corrosion before the structure is endangered.

Corrosion and its prevention should be considered for all metalwork, whether or not that metal is the predominant material in the structure.

Fabrication details should be kept as simple as possible and be designed to avoid corrosion and facilitate maintenance, especially in respect of on-site connections where tolerances should be generous because of the difficulties associated with working in a maritime environment. For the same reason, as much prefabrication should be undertaken as possible, taking advantage of mechanized welding and early painting under factory-controlled conditions, provided that these operations are carried out where adequate inspection and supervision are possible.

Table 22 — Typical upper rates of corrosion for structural steels in UK maritime conditions

Exposure zone	Corrosion rate[a]
	mm/year
Below seabed or riverbed level	0.01
Continuous seawater immersion zone: seabed level to low spring tide level	0.08
Continuous fresh water immersion zone	0.05
Intertidal low water zone: between low-water at spring and at neap tides	0.20
Intertidal zone: between mean low-water and mean high-water	0.09
Splash zone: above mean high-water to a height depending on mean wave height and exposure to wind	0.14
Atmospheric zone: above splash zone and where direct wave or spray impingement is infrequent	0.10

[a] The rate is for each face exposed to the environment of the zone.

59.2 Structural steel

59.2.1 *Type of steel.* Welded connections between structural elements are in common use on maritime structures, either exclusively or in conjunction with bolted connections and therefore only steels which are suitable for welding should be used. In most cases, welding by manual metal arc will be favoured and steels which comply with BS 4360 should be selected rather than other steels which may require more specialized welding techniques.

59.2.2 *Fracture toughness.* The generally accepted method of comparing steel fracture toughness with approved standards is by the use of the Charpy V-Notch impact test carried out in accordance with BS 4360 for various design minimum temperatures and various thicknesses. The requirements specified in BS 4360 are adequate for normal maritime structures but stricter requirements may apply where brittle fracture at low temperatures is of particular concern.

59.2.3 *Chemical composition.* The chemical composition limits and carbon equivalent values specified in BS 4360 are adequate to provide satisfactory weldability for maritime construction where lamellar tearing risks are low.

Weather resistant steels such as those described in BS 4360 are not recommended for maritime structures because their better resistance to inland atmospheric corrosion is not realized in the presence of continuously damp or humid chloride-contaminated maritime environments.

Steel embedded in concrete is cathodic relative to the same steel in seawater. Rapid corrosion will therefore occur at the interface of a partly embedded member. In such cases, special precautions should be taken.

Chemical composition of steels has less influence on corrosion rates in maritime environments than physical factors such as the roughness of the surface finish of the steel and the presence of holes and re-entrant corners, all of which tend to promote the formation of galvanic corrosion cells. It should be recognized that heterogeneity of any kind in the materials of a structure can lead to the formation of electrolytic cells with one material being sacrificed to the other. For example, cold worked metals have a higher potential than unworked metals of the same chemical composition and are, therefore, anodic to the latter. Cold working can result from impact damage and this should be remembered when making repairs to existing structures.

Where set-on connections are made by welding, particularly using full penetration welds, lamellar tearing may occur. In such cases, requirements additional to the properties specified in BS 4360 should be added to the steel specification. These usually take the form of guaranteed through-thickness reduction of area values.

When the steel is to be painted, other than with a simple tar coating, the mill scale which normally coats steel sections, plates and bars on delivery should not be allowed to weather off. During the removal of mill scale by natural weathering, galvanic corrosion is promoted by cells formed between the mill scale and the parent metal and deep pitting may result.

59.2.4 *Quality of material.* The required quality of the steel in respect of discontinuities such as laminations in plate and forgings and blowholes and sand inclusions in castings should be considered and appropriate requirements specified in relation to service stresses and fabrication procedure.

Plate material required to have a quality level in respect of laminations and inclusions should be examined in accordance with BS 4336.

Castings and forgings may be examined by radiography, ultrasonics or both depending on the size and shape of the component. Such examinations should be carried out after any heat treatment but before any machining or welding takes place.

59.2.5 *Welds and welding.* Consumable materials for welding of steel in maritime structures should comply with BS 639, BS 2901 and BS 4165.

59.2.6 *Bolts and nuts.* The use of bolts and nuts should comply with BS 449 or BS 3294.

59.3 Aluminium and its alloys

59.3.1 *General.* Aluminium in its pure state has a low tensile strength but because of its ductility and excellent resistance to corrosion it is frequently used in both rolled and extruded forms where strength is unimportant. Pure aluminium can be strengthened by cold working but alloys of aluminium are usually used where increased strength is required.

Aluminium is alloyed with magnesium, manganese or silicon or combinations of some or all of the elements and the current standards classify alloys of aluminium according to their chemical composition, denoted by a number which is greater with increasing strength and various letters denoting whether the alloy has been heat treated or not and what method of fabrication has been used. This classification is fully described in BS 1470, BS 1471, BS 1472, BS 1473, BS 1474, BS 1475, BS 1490 and CP 118 which deal with the specification and use of aluminium alloys for general and structural engineering purposes.

59.3.2 *Selection of alloy.* CP 118 gives guidance on the selection of alloys. Although H30 of CP 118 is the alloy generally available for structural use it is not classed as sufficiently durable for immersion in seawater although when given paint protection it is acceptable above water level. Alloy N8 of CP 118, which is of lower strength, is classed as the only suitable grade for use in seawater.

59.3.3 *Structural properties.* For full details of the engineering properties of the principal alloys for structural use, reference should be made to CP 118.

The use of aluminium in structures requires the same consideration as steel but closer attention has to be paid to the stability of parts in compression, deflections, vibrations and expansion.

Aluminium has an elastic modulus which is one third of that for steel which means that particular attention has to be paid to deflection of laterally loaded structural elements and the buckling of in-plane loaded structural elements.

The coefficient of linear thermal expansion of aluminium is between 23×10^{-6} and 24.5×10^{-6} per °C which is approximately twice that of steel. Consequently, care is needed to ensure that forces arising from temperature effects are allowed for in design.

59.3.4 *Protection from environment.* The recommendations of section 7 of CP 118:1969 should be strictly observed.

Pockets and crevices likely to trap water, dirt or condensation should be avoided. The greatest care should be exercised in structures containing other metals, such as steel and copper based alloys, to ensure that water cannot flow from the metals on to the aluminium and that the aluminium elements are electrically insulated from the other metals.

59.4 Other metals. Other metals used in maritime structures (see **59.1**) should comply with the requirements of the following standards where appropriate.

a) *Stainless steel* (BS 970-1). Molybdenum bearing steels are preferable.

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b) *Cast iron* (BS 3468 and BS 4844).

c) *Wrought iron.* Has good corrosion resistance.

d) *Brass and bronze* (wrought materials BS 2870, BS 2871, BS 2872, BS 2873, BS 2874 and BS 2875; castings BS 1400). Brasses usually suffer from dezincification in seawater and should be avoided.

e) *Monel metal.* Has high corrosion resistance but is expensive (see **68.4**).

60 Timber

60.1 General. The use of timber in maritime structures should be in accordance with CP 112-2:1971.

A wide range of species of timber can be used in maritime structures. Tables giving data on timber properties, working characteristics, main uses, sizes, origins and availability are obtainable from several sources. Further details of one of these sources are given elsewhere [51].

60.2 Resistance to environmental hazards

60.2.1 *General.* Durability of timber in maritime structures is limited by a number of processes of degradation which generally fall into two main categories: mechanical damage (see **60.2.2**) and biological attack (see **60.2.3** to **60.2.7**).

60.2.2 *Mechanical damage.* Causes of mechanical damage include impact and abrasion by ships, cargo and floating debris or movement of cargo handling equipment, or the wearing away of timber by pedestrian and vehicular traffic.

Parts of structures adjacent to the seabed and on exposed beaches can be subjected to extreme abrasion due to the impact of mineral particles in motion under wave and current action.

Species of timber can be selected to resist the particular damage or wear that is expected and specific recommendations are given in **60.3**. In certain instances the effects of wear can be minimized by providing sacrificial sheathing or rubbing strips.

60.2.3 *Biological attack in general.* Biological hazards to timbers in maritime structures consist of two main types. Fungal decay or rot is the main hazard to timber structures in freshwater situations but it is also an important cause of degradation in the superstructure of maritime works. The major biological hazard to timber permanently immersed in the sea is attack by marine borers. On sandy beaches the growth of marine borers will be retarded on the exposed faces of timbers and decay will mainly proceed outward from joints and protected faces.

60.2.4 *Fungal decay.* The rise in moisture content which occurs when timbers are used in waterside conditions is normally followed by the onset of fungal decay unless the timbers used have natural durability or have been given a protective treatment. Exclusion of oxygen prevents the process of breakdown and therefore timber which is sufficiently deeply immersed in water or is embedded in impermeable soil will not be degraded in this way. The timber decks and upper works of wharves or jetties are frequently found to rot where freshwater is able to dilute salt depositions and to maintain an elevated moisture content in the timbers. Rot occurs typically on covered horizontal surfaces and in joints. Careful detailing of structures is therefore necessary to ensure drainage of surface water and to avoid places in which fresh water can accumulate and enter joints.

End grain in joints should be treated with sealing compounds having an appropriately long effective life and measures should be taken, wherever possible, both in the storage of timber and in the detailing of joints, to avoid splitting. Embedment of timber in concrete or brick or its use in other situations which will encourage retention of moisture should be avoided.

60.2.5 *Marine borers.* The two most common types of marine borer which attack maritime structures are the crustacean *Limnoria* (the gribble) and the mollusc *Teredo* (the shipworm). Both are capable of causing extensive damage quickly and preventative measures have to be adopted if timber structures are to achieve more than a very short life in regions where these organisms are active.

Shipworm is confined to marine sites and estuaries and is unable to exist in fresh water. Occurrence in British waters is sporadic and mainly in the sea around southern England although it has been known to appear in the Orkneys and Shetland.

60.2.6 *Resistance to biological attack.* In maritime conditions, the primary resistance required is to borers and resistance to attack should be provided by selection from a fairly restricted list of timbers which are capable of using wood preservatives or by the use of sheathing. The need to provide protection against fungal decay should be considered in the superstructure of maritime structures.

In freshwater situations, fungal decay is the major hazard and the use of preservatives or timber having appropriate natural durability is essential.

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Natural resistance to fungal decay varies from species to species. Variation is also observed within single species and in some timbers the heartwood core may be less durable than the outer zones. Sapwood of nearly all species is susceptible to fungal decay. Despite these variations, naturally durable timbers can be identified in practice and can be used successfully in conditions where perishable species could rapidly decay.

Natural resistance to marine borers does not exactly parallel resistance to fungal decay. No timber is known to be completely resistant to marine borers and only a few possess the high resistance necessary to give satisfactory lives in areas where activity is intense. Timbers are normally classified for their natural resistance to marine borers as follows.

a) Very durable: suitable for use under conditions of heavy attack by *Teredo* and *Limnoria* and should provide at least 20 years of life.

b) Moderately durable: suitable under conditions of moderate attack, mainly by *Limnoria*.

c) Non-durable: suitable for only short service life.

60.2.7 *Preservation.* Wood preservation covers a number of protective measures and treatments which will greatly increase the life of the less durable timbers when subject to both maritime and freshwater conditions. Protective treatments include various processes in which timber is impregnated with preservative solutions under pressure. These should comply with BS 5589. In making use of such treatments, it should be noted that species vary greatly in their ability to absorb the preservative solutions but in almost all cases sapwood is permeable and its resistance is improved. The incising of timber which is otherwise difficult to treat is necessary for effective treatment. Effective application of preservative treatments also depends on timber being seasoned to a moisture content of 20 % to 25 % and all shaping and cutting being completed prior to treatment.

60.3 Functional suitability

60.3.1 *Piling.* The principal species of timber used for piling are greenheart, basrolocus, balau, pitch pine, and Douglas fir. Many other species can be used, however, and choice will depend largely on availability in the region in which the works are situated.

60.3.2 *Structures.* The timbers commonly used in the main structural elements in superstructures are similar to those used for piling. The design of deck structures should take into account that horizontal surfaces may retain rainwater in positions where it cannot be quickly removed by evaporation. Timbers vary in their wear resistance to pedestrian and other traffic when employed as decking and the use of species such as Douglas fir, Western hemlock, Baltic redwood and dahoma is best confined to areas of light traffic. Pitch pine, dark red meranti, keruing, kapur, opepe, jarrah and karri, for example, have sufficient resistance to wear to recommend their general use for decking, except where resistance to heavy pedestrian traffic, severe trucking and resilience to impact loads is required, when oak, danta, belian, greenheart and okan, amongst others would be preferred.

60.3.3 *Kerbs and capping pieces.* To be effective, kerbs and capping pieces need to be of generous scantlings and sections in excess of 250 mm square are normal. Suitable species with the necessary impact and abrasion resistance are opepe, jarrah and karri.

60.3.4 *Fendering and rubbing strips.* Due to its resilience and soft contact, timber is used in a number of applications in fendering including fender frames and floating fend-offs. In all such applications, the qualities required are those described for fender piling.

A wide variety of timbers are successfully used as rubbing strips on piles, solid pier walls and jetties. The requirements are resilience, compressibility and resistance to splitting. As these rubbing strips can usually be built up to any required length, size is not an important factor. Resistance to splitting and abrasion is often associated with interlocking grain and the widespread use of elm for rubbing strips can be related to the variation in grain direction of this species. For similar reasons, tropical hardwoods with interlocking grain are also widely used. The species used for rubbing strips include elm, Douglas fir, pitch pine, oak, jarrah, opepe, ekki and okan. Choice will depend largely on a combination of availability and required durability.

The use of sacrificial rubbing faces is useful in that it allows the material of main members to be chosen for their structural properties, their availability in appropriate sizes and resistance to biological attack. The timber for rubbing strips may then be selected for impact and abrasion resistance.

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60.3.5 *Sea defences.* Among the timbers which are found to be satisfactory in sea defence works such as groynes are jarrah, greenheart, pynkado, ekki, opepe, and pitch pine. Of the UK grown timbers, oak is the best. The piling should be in greenheart or jarrah with pitch pine as an alternative. Timbers requiring protective treatment are not suitable as the treatment usually is effective only in the surface zone of the material which may be removed by abrasion relatively early in the life of the works.

60.3.6 *Other applications.* Timber is used traditionally for small to medium-sized lock gate construction. Ability to withstand shock loads, high strength to weight ratio, comparatively low cost and ease of in situ repair are the main requirements.

Species commonly employed on inland waterways include oak, pitch pine, Dutch and Wych elm, or greenheart where extra strength, abrasion resistance or long lengths are required.

Sluice paddles are traditionally constructed of timber. Sizes vary up to doors of four metres square and a variety of timbers is used. High friction is generated in the guide grooves which may be of masonry and good abrasion resistance is therefore required. Greenheart, balau, ekki and okan are suitable timbers in this application.

60.4 Fastenings. All steel jointing materials should preferably be hot dip galvanized and coated with a suitable paint or bituminous protective treatment. All bolts, nuts and washers should be similarly treated and bolts should be dipped in a suitable bituminous material immediately prior to assembly.

The design of joints using bolts, shear plates and ring connections is covered in CP 112. Bolted fastenings should comply with BS 916. Washers should comply with BS 3410 which includes coverage of large square and round washers for use on timber. Connectors for timber should comply with BS 1579.

Other fastenings which are used in timber maritime structures are screws, coach screws and spikes but their use should be confined to the fixing of planking and rubbing strips and similar connections of secondary importance. Screws should comply with BS 1210.

Rag bolts or indented bolts are used for fastening timber work to the face of concrete work. These items should be manufactured in mild steel, should be hot dip galvanized and should comply with BS 1494. Parts not embedded in concrete should be treated with suitable bituminous compounds or paint.

61 Piles

61.1 General. Piles in maritime works can be divided into two groups:

a) bearing piles as used in foundations, wharf, jetty and dolphin construction, or in fendering systems;

b) sheet piles as used in retaining walls or for wave protection.

Materials used for bearing and sheet piles include timber, concrete and steel. Fender piles are designed primarily to withstand or transmit horizontal impact forces and, for toughness, timber or steel is normally used. There are a number of specialized piling systems, some of which use combinations of materials.

61.2 Bearing piles

61.2.1 *General.* Descriptions of bearing piles are given in CP 2004 and particular considerations relevant to the maritime environment are given in **61.2.2** to **61.2.5**.

61.2.2 *Selection of bearing piles.* The selection of the most suitable pile depends on the following four factors:

a) location and type of structure;

b) ground conditions;

c) durability requirements;

d) availability and cost.

For a structure built over water, timber piles may be suitable but are limited by considerations of length and cross section. Solid precast or prestressed concrete piles can be used in fairly shallow water but in deeper water a solid pile becomes too heavy to handle and either a precast or prestressed tubular pile or a steel tube, box or H-section pile is more suitable. In exposed maritime conditions, steel tube or box piles are preferable to H-sections because of the smaller drag forces from waves and currents. Large diameter steel tube or box piles are also an economical solution to the problem of dealing with impact forces from waves and berthing ships. Bored and cast-in-place piles should not be considered for a maritime or river structure unless used in a composite form of construction, such as extending the penetration of a tubular pile, driven through water and soft soil, to a firm stratum.

The choice of pile for inshore or land work can be made from any of the three bearing pile categories, namely large displacement, small displacement or non-displacement. Of these, bored and cast-in-place piles are often the most economical and large diameter bored piles, sometimes with enlarged bases, are capable of carrying very high loads. Bored piles are often specified in environments where ground heave, noise and vibration are to be avoided but great care should be exercised in their use in waterside situations in which ingress of water to the pile excavation can cause very serious structural defects.

The second factor, ground conditions, influences both the material forming the pile and the method of installation.

Augered bored piles are suitable for firm to stiff cohesive soils but augering is not possible in very soft clays or in loose or water-bearing granular soils and in these cases driven or driven-and-cast-in situ piles would be preferable. Concrete driven and driven-and-cast-in situ piles are not suitable for ground containing boulders, nor can they be used in soils subject to ground heave, where this is to be avoided. Driven-and-cast-in situ concrete piles which use a withdrawable tube cannot be used for very deep penetrations because of the limitations of jointing and retrieving the driving tube. A driven pile or a mandrel-driven thin-walled steel pile would be suitable in these conditions. Thin-walled steel piles are liable to tearing when being driven through soils containing boulders. For hard driving conditions in boulder clays or gravelly soils, a thick-walled steel tube pile or a steel H-section can withstand heavier driving than a precast or prestressed concrete pile of solid or hollow section. Some form of drilled pile, such as a drilled-in steel tube, would be appropriate for piles taken down into rock for the purpose of mobilizing resistance to uplift from lateral loads.

The third factor, requirements of durability, affects the choice of material for a pile.

Timber piles are liable to decay above ground water level and in some situations they are damaged below water level by marine borers (see **60.2**). Precast concrete piles should not suffer corrosion in saline water below the splash zone and well compacted concrete can generally withstand attack from quite high concentrations of sulphates in soils and ground waters (see **58.2**). Piles can also be damaged by abrasion (see **58.1** h) and **60.2.1**) and protective measures are necessary, depending on the particular circumstances.

Cast-in-place concrete piles are not so resistant to aggressive substances because of difficulties in ensuring complete compaction of the concrete but protection can be provided against attack by, for example, placing the concrete in permanent linings of coated light-gauge metal or plastics.

Steel bearing piles in normal undisturbed soil conditions generally have an adequate resistance to corrosion. The portion of the piles above the seabed in maritime structures, in disturbed ground or in corrosive soils may be protected by cathodic protection or other methods as described in clause **68**, or be specified with a thickness suitable to allow for corrosion.

Further considerations, particularly in respect of timber piles, precast concrete piles and steel piles, are given in **61.2.3** to **61.2.5**.

61.2.3 *Timber bearing piles.* Round log or hewn piles are often acceptable in lieu of sawn piles but only certain timbers are suitable for maritime piling (see **60.3.1**). The resilience of such timbers and their high strength to weight ratio make them very suitable for bearing and fender piling but to obtain adequate durability the correct species and grading should be used, connections should be carefully detailed (see **60.4**) and appropriate preservative treatments (see **60.2.7**) should be adopted.

The abrasion of timber piles by sand or shingle on the seabed can be prevented by tipping stones around them and sometimes they are protected by strapping sacrificial timbers to them. Timber bearing piles in maritime structures are often protected against impact damage by sacrificial elm rubbing strips spiked or coach-screwed to them.

The working stresses in timber piles should not exceed the permissible green stresses, given in CP 112-2, for compression parallel to the grain for the species grades 50 and better. The special structural grades (SS and MSS) and the M50 or better grades of CP 112-2 should be adequate for both softwood and hardwood. When calculating the working stress on a pile, allowance should be made for bending stresses due to eccentric and lateral loading and to eccentricity caused by deviations in the straightness and inclination of a pile. Allowance should also be made for reductions in the cross-sectional area due to drilling or notching and to the taper on a round log.

61.2.4 *Concrete bearing piles.* Concrete in piles should be in accordance with clause **58**.

Precast concrete piles are often used in maritime structures because of the simplicity of manufacture and their suitability in a wide range of applications, but their main disadvantage is the difficulty of extending them when necessary.

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Particularly in maritime works, prestressed concrete piles are normally preferable to ordinary reinforced concrete piles because they are able to tolerate larger bending loads and are more durable. Their higher bending strength permits manufacture and handling in a wider range of lengths and greater resistance to lateral forces in operation.They have a greater resistance to tension stresses caused by uplift forces. A further advantage of prestressed concrete is that any cracks which occur during handling and driving are likely to close up. This, combined with the high quality concrete necessary for prestressing, gives the prestressed pile increased durability in the maritime environment. However, prestressed piles are less resistant than reinforced concrete piles to impact, as may arise from harbour craft or lighters.

The working stresses in the concrete during lifting, handling and pitching of precast piles should not exceed those recommended in BS 8110-1. If nominal mixes are adopted, a concrete of grade 40 of BS 8110-1 is suitable for very hard driving for all maritime construction. For normal or easy driving, a grade 30 concrete is suitable but if there is exposure in the splash zone, grade 40 of BS 8110-1 is preferable (see **58.5**). High stresses, which may exceed the handling stresses, can occur during driving and it is necessary to consider the serviceability limit of cracking. BS 8110-1 permits crack widths of up to 0.3 mm for members in aggressive environments but for severe exposure conditions which include piles in seawater, the crack widths near the main reinforcement should not in general be more than 0.004 times the cover to the main reinforcement.

The cover to the reinforcement should not be less than 50 mm in ordinary and prestressed concrete piles exposed directly to seawater.

61.2.5 *Steel bearing piles.* Types of steel bearing piles include tubes, box sections, H-sections and tapered and fluted tubes. They can be designed as large or small displacement piles which, in the latter instance, is advantageous in situations where ground heave and lateral displacement have to be avoided.

Steel bearing piles have the advantage of being robust and can be driven hard to a deep penetration to reach a bearing stratum or to develop a high skin frictional resistance. They can be readily cut down or extended where the level of the bearing stratum varies and the head of a pile which buckles during driving can be cut down and retrimmed for further driving. They have a good resilience and high resistance to buckling and bending forces. They are light to handle compared with precast concrete piles and this can be particularly advantageous in maritime construction over water but their cost per metre run is greater. Their lightness facilitates their use as raking piles.

The extending of steel piles for driving to depths greater than predicted is carried out easily by welding but the practice which is satisfactory in the leaders of a piling frame for land structures may need considerably more care in maritime works where the section of the pile above seabed level is subjected to high lateral forces and corrosive influences and the highest strength and best protective treatment are required. Conditions are not conducive to first class welding when the extension pile is held in leaders or guides on a floating vessel or on staging supported by piles swaying under the influence of waves and currents.

Hollow-section piles may be driven with a closed end in order to assist developing increased end bearing resistance over the pile base area. Where deep penetrations are required, they may be driven with open ends possibly with thicker driving shoes up to one metre long welded on the toes and then filled with concrete after cleaning out the soil. In some circumstances, the soil plug within the pile may itself develop the required bases resistance. A possible subsequent loss of strength, particularly by boiling in granular soils, can be prevented by filling the pile with water immediately after driving.

If the base resistance has to be eliminated when driving hollow-section piles to a deep penetration or to avoid ground heave, the soil within the pile can be progressively cleaned out by augers, by reverse water-circulation drilling or by airlift. It is not always necessary to fill hollow-section piles with concrete and each case should be considered in relation to its circumstances.

Hollow-section piles have an advantage when inspecting a pile for buckling. A light can be lowered down the pile and if it remains visible when lowered to the bottom, no deviation has occurred. If a large deviation is shown by complete or partial disappearance of the light, measurements can be made and measures taken to strengthen the buckled section by, for example, inserting a cage of reinforcement and placing concrete.

Circular-section piles have considerable strength as columns where piles project above the ground or seabed and are preferred when soil has to be cleaned out for subsequent placement of concrete, since there are no corners from which the soil may be difficult to remove. The circular shape is also advantageous in minimizing drag and oscillation from waves and currents.

Steel tubes for piles can be seamless, longitudinally welded or spirally welded. There is nothing to choose between the latter two types of welding from the point of view of strength to resist driving stresses.

61.3 Sheet piles

61.3.1 *Timber sheet piles.* Timber sheet piles are used for low earth-retaining structures and beach groynes, for replacing damaged sheet piles in existing timber structures or for temporary works such as cofferdams. Interlock can be achieved by tongue and groove joints or by lapping the joints of a double row of planks. The materials and preservative treatments which should be used are given in clause **60**.

Timber sheet piles are often fitted with steel shoes to prevent splitting of the toe and to assist in driving. Cutting of the toe on a longitudinal rake will assist in close driving.

61.3.2 *Concrete sheet piles.* Reinforced concrete sheet piles are often used for durability, for example in retaining walls in canal banks where they are quite short and hence of acceptable thickness. The height of any earth-retaining structure is limited by the depth to which it is possible to drive the piles without breakage in order to achieve the required passive resistance to earth pressure. It is also difficult to achieve a tight interlock because of warping and shrinkage in individual piles. In addition, with increasing length the thickness of concrete piles, even if prestressed, becomes excessive, making them very heavy and therefore expensive to manufacture and handle. Their use in UK conditions is generally regarded as uneconomic but they remain in use in developing countries in structures of modest size.

Steel shoes are not normally required on the toes of concrete sheet piles driven through soft or loose soils into dense sands and gravels or firm to stiff clays. A blunt pointed end is all that is required to achieve the desired penetration in these soils and if the blunt point is cast on a longitudinal rake it will assist in the close driving of adjacent piles.

Concrete mixes, working stresses, cover to reinforcement, aggressive conditions and the use of prestressed concrete in concrete sheet piles should comply with **61.2.4**.

61.3.3 *Steel sheet piles.* Steel sheet piling is used in all types of temporary works and permanent structures including cofferdams, retaining walls, river frontages, quays, wharves, dock and harbour works, permanent foundations, land reclamation and sea defence works. Piles are available in various sectional shapes and the stiffer sections are capable of being driven to a considerable depth in a wide range of ground conditions and into weathered rocks. The interlocks between adjacent piles are relatively watertight.

The more common types of interlocking steel sheet piles are as follows:

a) U-section;

b) Z-section;

c) H-section;

d) straight-web.

Types a) to c) are designed to have significant bending strength whereas piles of type d) are designed primarily to act in tension across the webs and are used in cellular configurations.

Each type is produced in a comprehensive range of sections in widely differing sizes and masses, thus enabling the most economical choice to be made to suit the conditions at any particular site. Other types of steel sheet pile and compound sections varying in size, mass and strength are also available.

Steel sheet piling can be rolled in lengths up to approximately 35 m, though handling facilities do not always permit the use of such long lengths. This is especially so with straight-web piling. Where it is necessary to increase the pile length during driving, site-welded joints can be used.

Although appreciable savings in steel content can result from the adoption of steels with high yield stresses, careful attention to the prevention of corrosion is essential (see **59.1**).

Certain sections of steel sheet piles, intended for use in permanent structures where stresses are not severe, such as cut-off walls, are of uniform thickness throughout in order to provide the maximum effective useful life.

The life of sheet steel piles may be extended by using steel containing between 0.25 % and 0.35 % of copper, as copper-bearing steel is more resistant than ordinary steel to atmospheric corrosion.

However, the extra cost of copper-bearing steels may not be justified as they do not afford any useful improvement in corrosion resistance in the intertidal zone (see Table 22).

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As only one side of sheet piling will normally be accessible for maintenance, protective coatings cannot easily be removed. Hence it is desirable to allow for greater corrosion.

Systems for protecting steel piling against corrosion are given in clause **68**.

61.3.4 *Other types of sheet pile.* Glass reinforced plastics or corrugated asbestos sheet piles are sometimes used for low earth-retaining structures where durability is required over a long period in an aggressive environment. These materials are brittle and have to be driven gently.

62 Pipes

62.1 General. Pipes used in the maritime environment can be divided into four main categories:

a) pipelines laid on or under the seabed;

b) pipelines laid below ground on land in quays, piers, marginal and inland storage areas;

c) pipelines supported above ground;

d) flexible pipelines for ship-to-shore connection.

Factors influencing the selection of pipeline materials for each of these categories are given in **62.2** to **62.5**.

Pipes are manufactured in a wide variety of materials, a list of pipes currently available being given below. These should comply with the requirements of the appropriate standard where such exists.

1) Aluminium.

2) Asbestos cement (BS 3656).

3) Concrete (BS 5911 and BS 5178).

4) Copper (BS 2871).

5) Ductile iron (BS 4772).

6) Glass reinforced plastics (GRP) (BS 5480).

7) Glazed vitrified clayware (GVC) (BS 65).

8) Reinforced plastics matrix (RPM).

9) Pitch fibre (BS 2760).

10) Plastics.

i) Polyvinyl chloride (PVC) (BS 3506, BS 4660 and BS 5481).

ii) Low density polyethylene.

iii) High density polyethylene.

iv) Polypropylene.

v) Acrylonitrile butadiene styrene (ABS) (BS 5391 and BS 5392).

vi) Nylon (BS 5409).

vii) Cellulose acetate butyrate (CAB).

viii) Polytetrafluoroethylene (PTFE) (BS 4976 and BS 5130).

11) Spun iron (BS 1211).

12) Steel (BS 534 and BS 3601).

13) Resin coated and bonded helically wound steel strip construction with resin coatings on inner and outer surfaces.

Selection of pipe material for a particular situation is governed by its suitability for the duty involved, taking account of availability, strength, durability and the construction methods to be adopted.

WARNING. The use of asbestos products is controlled by the Asbestos Regulations 1969, the Control of Asbestos at Work Regulations 1987 and the approved Code of Practice relating to those Regulations. The Regulations require that any dust liberated during the handling and working of these products should be maintained at a low level. Adequate method exist to control levels of dust during such operations and these are detailed in the Control and Safety Guides issued by the Asbestos Research Council.

62.2 Submarine pipelines. The selection of material for pipelines laid on the seabed will be influenced largely by the method of construction proposed. For sea outfalls, the most commonly used material is steel, which can be welded in long sections on the shore and pulled out into position from an anchored barge or placed by lay-barge. Protecting such pipes is very important and they are normally lined and surrounded on the outside with concrete armour coating which provides added weight as well as additional protection. The lining can consist of bituminous enamel, epoxy resin or cement mortar. Pipelines are often wrapped on the outside as well as being coated and cathodic protection is also provided in many cases. Steel pipes can be made in a wide range of sizes and steel is the most widely used material for large diameter submarine pipelines. Steel pipes can be jointed using couplings but welded joints are preferable.

PVC pipes are also suitable for submarine pipelines and can be towed into position in long lengths in much the same way as for steel pipelines. PVC pipes are currently available up to 600 mm diameter in the UK, but it is likely that larger sizes will be developed in the future.

Concrete pipes are suitable for submarine pipelines but the method of construction usually involves laying pipes in the dry, within a cofferdam. Attention should be given to providing sufficient cover to reinforcement in the pipes and it is normal to use pipes with greater wall thickness than would be applicable when pipelaying on land. If required, concrete pipes are generally manufactured with sulphate-resisting cement for maritime applications. The method of handling and storage of the pipes prior to laying is important and there have been cases of concrete pipes being damaged due to the abrasion effects of wind-blown sand when stored for long periods on the foreshore.

Spun iron and ductile iron pipes can be used for submarine pipelines, though their use is not common. It should be noted that the bolted-gland type of pipe jointing can be made by divers so that underwater pipelaying methods can be carried out with this type of joint.

The problems of catering for flexing of pipes coming off a lay-barge need to be considered. Particular attention should also be given to flotation problems when considering submarine pipelines and the method of laying and anchoring the pipe may well have an influence on the pipe material selected, particularly when larger pipe sizes are involved.

62.3 Underground pipelines. Particular points that need to be borne in mind in selecting materials for pipelines laid below ground in docks and port areas are as follows.

a) *Strength.* Surface point loads are likely to be higher in port areas than normally encountered and special consideration should be given to pipe material strength and cover to pipelines.

b) *Joints.* Land areas around docks are frequently formed by reclamation and it is quite likely that the ground will not be particularly well consolidated and settlement is to be expected. Wherever possible, flexible joints should therefore be used for pipes of rigid materials.

c) *Durability.* It is likely that the ground water within port areas will be saline and therefore more corrosive than conditions encountered in normal ground. In these circumstances, sulphate-resisting cement should be used where asbestos cement or concrete pipes are used. Special precautions for iron and steel pipes would include complete protection systems, probably involving cathodic protection as well as protective coatings on the pipes. Glazed vitirified clayware pipes, pitch fibre and plastics pipes have good resistance to deterioration in a maritime environment and are very suitable for use in such situations.

Both pressure and non-pressure pipelines are needed in areas near to docks. Pipe materials normally used for non-pressure pipelines include asbestos cement, concrete, glazed vitrified clayware, pitch fibre and PVC. Materials for pressure pipelines include asbestos cement, reinforced concrete, ductile iron, GRP, spun iron and steel. Plastics pipes, in addition to being corrosion resistant, have also a slight natural flexibility making them suitable for ground where settlement is likely to occur.

62.4 Surface pipelines. When pipes are supported above ground, the structural strength of the pipe materials assumes great importance as this will affect the spacing of the support system. Steel is suitable for pipelines above ground but corrosion protection assumes greater importance because the pipe is more exposed to the aggressive effects of sea spray. If steel pipelines are used, particular attention has to be paid to the selection of pipe fittings which are compatible and will not cause accelerated corrosion by electrolytic action. Consideration should be given to isolation of fittings from the pipeline by means of gaskets to prevent this condition.

Ductile iron and spun iron pipes are more resistant to corrosion than steel pipes but generally require to be supported at closer centres.

PVC and ABS pipes are also suitable from a corròsion resistance point of view but need supporting at more frequent intervals. They have the advantage of being able to withstand very heavy general usage and their slight natural flexibility is an added advantage. They can carry a wide range of corrosive liquids and therefore have a wide application.

62.5 Flexible hoses. Flexible hoses are used for loading and off-loading bulk liquid cargoes and can be of reinforced rubber, plastics or canvas. They need to be able to withstand very heavy general usage and should therefore be selected with attention to robustness and fatigue life.

63 Pavements

63.1 General. Paved surfaces are required in port and dockyard complexes for roads, circulation areas, hardstanding areas for the parking of vehicles and storage of cargo and shed floors. Other pavings include breakwater cappings and sea-walls where environmental conditions can be more damaging than traffic conditions.

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Particular points to be considered in selecting materials for pavements are:

a) loadings;

b) durability;

c) settlement characteristics.

63.1.1 *Loadings.* The wheel loadings imposed by mobile cargo handling equipment can be very large (see clause **45**) and the tendency is for such loadings to increase with the development of new cargo handling systems. Axle loads imposed by fork lift trucks handling loaded containers frequently exceed 50 t and systems in which four or more loaded containers are carried on a single chassis can give rise to single bogie loads of 150 t. These loadings are quite different from those experienced due to highway traffic where wheel loadings are more frequent, less intense and travel at higher speeds. The pavements required to withstand wheel loads of such high intensity will generally be significantly heavier in design and more costly than highway pavements and to achieve economy in planning port layouts it may be necessary to confine the heavy duty pavements to the working areas of the heavy equipment, the remaining areas being paved to a suitably reduced specification.

63.1.2 *Durability.* In addition to long term loading due to various kinds of bulk and break bulk cargo, pavements have to be capable of withstanding wear and tear by lorries and cargo handling equipment. In hard-standing areas where the small jockey wheels of parked semi-trailers give rise to intensive point loading of the surface and in areas where tight curves have to be negotiated by multi-axled vehicles, concrete construction should be adopted.

Modern cargo handling equipment may have solid rubber or plastics wheels or may use high pressure pneumatic-tyred wheels. Plant is almost always power steered. This combination can give rise to severe scuffing of the surface of pavements as the hard wheels are steered whilst the vehicle is at a standstill. Selection of an appropriate surface finish is necessary for such duties.

Another factor which, combined with the problem of scuffing, can give rise to rapid surface deterioration is attack by fuels, lubricating oils and hydraulic fluids dropped by mobile plant. Where such contamination cannot be avoided, susceptible paving materials such as asphalt should not be specified.

63.1.3 *Settlement characteristics.* Pavements in port and dockyard areas are frequently constructed on poor ground or in areas where land has recently been reclaimed and is subject to settlement. In such situations appropriate means of accelerating settlement and/or strengthening the ground such as pre-loading, various drainage methods, vibroflotation or dynamic consolidation should be considered. In combination with those methods, a pavement type which is tolerant of settlement, which can be regulated and repaired from the top or which can easily be taken up and relaid should be selected. Rigid pavements which are not tolerant of settlement and which can only be repaired or replaced at considerable cost should only be selected when the foundation is reliable. In situations where pavements have to be brought into use before the residual settlement is reduced to an acceptable amount, it may be desirable to provide a temporary pavement which will serve operational needs for an interim period while settlement continues. Such a temporary pavement may be designed to form one of the lower courses of the final design pavement after regulation.

63.2 Bitumen-bound flexible pavement. This pavement consists of flexible surfacing made up of a wearing course of hot rolled asphalt or dense bitumen macadam on a basecourse constructed of one of the materials. This flexible surfacing is found on a structural basecourse such as lean mix concrete, cement-bound granular materials, dry or wet bound macadam, dense bitumen macadam or rolled asphalt which in turn is laid on a layer of sub-base material.

A bituminous pavement is relatively tolerant of ground settlement and can be raised by the addition of regulating courses if settlement occurs. This type of pavement can be opened for access to service ducts and reinstated with relative ease but should not be used where there is regular spillage of petroleum oils.

Care should be taken in the use of bituminous pavements where high point loads are applied at random locations. The Marshall method [52] of mix design produces a dense well-graded surfacing, known as asphaltic concrete, which can be used in hot as well as temperate climates for heavy duty paving.

63.3 Tar-bound flexible pavement. A pavement made with a tar binder is similar to one made with a bitumen binder but has good resistance to fuel and oil droppings. Like bitumen-based paving it is susceptible to hydraulic fluid spillage. It is, however, less tolerant to extremes of temperature than bituminous paving. Tar-PVC compounds have been developed to overcome this deficiency.

63.4 Concrete block pavement. Interlocking block pavement consists of accurately cast blocks of high strength concrete either plain sided or in shapes which join to form an interlocking pavement when laid and vibrated into a sand or crushed stone bed. A base layer is required which can be in any of the materials normally used as bases for bituminous pavements. A lean mix concrete or cement-stabilized sand is commonly used. The thickness of the base layer is governed by the nature of the wheel loads to be carried and the quality of the formation soils.

Interlocking concrete block construction is particularly suitable where a pavement is required to tolerate large, uneven settlement. Where settlement occurs, blocks can be lifted, the base layer increased in thickness and the same blocks laid again with only minimal wastage. Block pavements also permit easy and inexpensive access for the installation and maintenance of buried services.

Details of a typical specification for concrete paving blocks are given elsewhere [53].

The laying course should not contain more than 3 % (*m*/*m*) silt or clay and should comply with the grading requirement of a grade M fine aggregate complying with BS 882 where the compacted thickness does not exceed 50 mm. A crushed stone bedding consisting of 60 % of 0 mm to 8 mm crushed stone and 40 % of stone dust can be laid with a compacted thickness exceeding 50 mm. This is a desirable feature in cases where blocks are to be lifted and relaid in order to compensate for settlement. Maximum thickness should not exceed 100 mm to 120 mm.

The blocks should be vibrated to their final level with a plate vibrator. Generally sand bedding should be screeded to a thickness of 50 mm which is reduced to 35 mm after the blocks have been laid on the sand and vibrated. Sand for brushing into the surface joints should be grade M concreting sand complying with BS 882.

63.5 Precast concrete slab pavement. This pavement consists of precast slabs in standard sizes up to about 2 m square placed in position on a prepared bedding layer. Slabs should be framed with steel angles to prevent edge spalling.

Their thickness and reinforcement are chosen to withstand the largest wheel loads expected and a wearing layer containing specially abrasion-resistant aggregates may be specified. Precast slabs should be of concrete of at least grade 40 of BS 8110-1 concrete and should comply with clause **58**.

After grading and compaction of the sub-grade, a bedding layer of sand or 5 mm crushed stone is laid and screeded. A base layer is required in areas where there are heavy wheel loads and/or the sub-grade is weak.

Slabs should be laid with uniform gaps of about 5 mm and a gap-filling sealant may be used to prevent surface water saturating the bedding layer. It is necessary to ensure effective drainage of the sub-grade.

In areas liable to settlement, such slabs are unsuitable as uneven settlements under large slabs can cause cracking under heavy wheel loads due to loss of uniform support.

Precast slabs can be laid using a fork-lift truck and they may be readily lifted and relaid by the same means, for regulation or for the installation and maintenance of services.

63.6 In situ concrete pavement. This type of paving is only suitable where ground conditions are such that differential settlement will be minimal. It is difficult and expensive to open and reinstate in situ concrete paving and its use over service ducts should normally be avoided. As with other types of concrete paving, resistance to oil spillage is good.

Concrete paving should be constructed in rectangular bays, the length of which should take into account expected subgrade settlement, shrinkage and thermal expansion and in no rectangular bays should the length exceed the breadth by more than one third of the latter. The thickness required is a function of the maximum concentrated loads to be applied and the quality of the sub-grade materials.

Concrete should have a minimum grade of grade 30 of BS 8110-1 and should comply with clause **58**.

64 Rail track

64.1 General. Rail track is used in maritime facilities both as permanent way for rail traffic and as track for special rail-mounted equipment including cranes, hoppers, slipway carriages and lock and dock gates.

Requirements for materials for permanent way in port facilities are not normally different from those applying to permanent way generally and materials should comply with BS 11, BS 105, BS 500 and BS 536, as appropriate. Further standards cover the requirements of fittings, including fish-plates and fishbolts and are referred to in these standards.

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A large proportion of permanent way in port areas is constructed flush with pavements and care is necessary in their detailing and in the specification of foundations, drainage and method of installation since problems can arise due to differential settlement and movement between tracks and pavement.

Crane rails and rail track for special equipment may be required for a wide range of duties including the carriage of wheel loads much higher than those used on permanent way. The materials and details for special rail tracks should therefore be selected and specified according to the needs of each application. The rail section selected will be a function of the maximum wheel loading and the method proposed for supporting the rail. There are numerous methods for bedding, positioning and holding down crane rails and these are frequently supplied as proprietary systems, Factors which should be considered in selecting from available systems are given in **64.2** and **64.6**.

64.2 Bedding system. An effective bedding system should be provided. This will normally give continuous support to the rail and may involve a number of different components including a continuous rubber pad between the base of the rail and a steel base plate, a bedding medium such as sand/cement mortar, free flowing cement grout, or an epoxy compound which can be placed in situ between the steel pad plate and the load-bearing structure on which the rail is founded. It is important that the bedding gives full support over the whole area of the bed plate and that it is free from defects. Great care is therefore required from detailed design to supervision of installation.

Various cement-based and epoxy bedding media, usually supplied as proprietary materials, may be used. Many of these are of a pourable consistency and these are particularly convenient where the requirement for flush track necessitates installation of the rails in a deep groove. Particular attention should be paid to cleaning out the area between the base plate and the supporting structure before pouring and to see that no air is trapped between the bedding medium and the underside of the base plate.

Sand/cement mortar bedding should be proportioned 3 : 1 by mass and should be of a workability suitable for placing by ramming, preferably using pneumatic percussion tools. The mortar should be of the highest strength obtainable and should have a characteristic strength not less than 40 N/mm^2. The general arrangements for installing the track should be such that operatives have easy access to carry out the placing and ramming of the bedding. It is not easy to carry out the operation properly if the track is set down in a groove in the concrete deck.

64.3 Holding-down bolts. Where an in situ placed bedding medium is used, the system of holding-down bolts provided should allow the rails to be accurately adjusted to line, level and freedom from twist and be rigid enough to hold the rails in correct register whilst the bedding materials are placed and cured.

64.4 Rail clips. In selecting rail clips, consideration should be given to the need to give positive restraint to lateral and twisting displacement of the rail but at the same time it may be necessary for the clips to permit some linear displacement of the rail under thermal expansion.

64.5 Heavy duty crane rails. Heavy duty crane rails are normally continuous at structural expansion joints. The steel base plate should normally be discontinuous and movement between the rail and the base plate may be accommodated by the rail sliding under the rail clips, provided that these are of a suitable design to allow such movement.

64.6 Bolted joints. Bolted joints in crane rails tend to give trouble and rails should normally be jointed by welding.

65 Brickwork, blockwork and masonry

The uses of brickwork, blockwork and masonry in maritime works are usually similar to those in general building and civil engineering works. Materials and workmanship for maritime applications, however, should always be of the highest quality in order to provide satisfactory durability.

Sand for mortar should be naturally occurring sand or consist of crushed rock or gravel or a combination thereof with naturally occurring sand and comply in all respects with BS 1200.

Clay bricks and blocks should comply with BS 3921. They should have a high frost resistance and the soluble sulphate radical content measured according to BS 3921 should be less than 0.5 %. Engineering bricks should comply with class A or class B of BS 3921.

Brickwork should generally be in English bond with half bricks not being used unless they are required for bond. Joint thicknesses should remain constant and no four courses should rise above the same laid dry by more than 40 mm for 10 mm joints or 25 mm for 6 mm joints. Brickwork should be carried up in a uniform manner with no position being raised more than 1 m above another at any one time.

Fresh cement mortar joints in brickwork below tide level should be treated with a quick-setting water-proofing compound to prevent the leaching of joints when the tide rises above the brickwork level.

Concrete blocks and their use should comply with BS 6073-1.

Masonry is seldom used now in new construction but repairs to existing masonry structures are frequently required. Repair can often be effected more economically by alternative means but in some cases, either for engineering or aesthetic reasons, repairs in masonry are necessary.

66 Bituminous materials

66.1 General. Bituminous materials are used in maritime works for the following:

a) pavements;

b) coatings;

c) waterproofing;

d) sealing compounds;

e) coast and bank protection.

Their use in the construction of pavements is covered in clause **63** and for protective coatings in clause **68**. Their use for waterproofing and sealing compounds is similar to their applications in other structures and requires no special comment. The uses, mixes and application techniques of bituminous materials for coast and bank protection are described in **66.2** to **66.4** inclusive.

66.2 Bituminous materials available. Bitumen is employed in a number of ways for coats and bank protection work but not all bituminous materials used for other hydraulic applications can be used for this purpose. Pure bitumens, cutbacks and emulsions are only applied for tack coats and surface treatments.

Aggregate/cutback or emulsion mixtures are rarely used but extensive use is made of hot mixtures of bitumen and aggregate. These mixes are generally dense and it should be noted that impermeability is not an essential requirement of a bituminous protective cover for coast defence works and the impermeability of bituminous revetments is often a disadvantage from the design point of view. A low void content is nevertheless recommended owing to difficulty in constructing strong and durable open mixtures. Open stone asphalt is the one exception, however. Hot mixtures in common use are:

a) asphaltic concrete (gravel and sand asphalt and stone filled sheet asphalt also belong to this group);

b) sand mastic or asphalt mastic.

For special applications the following mixtures are also available:

1) stone asphalt;

2) lean sand asphalt.

66.3 Composition, mix design and application techniques

66.3.1 *Asphaltic concrete.* Asphaltic-concrete types of mixtures consist of stones or gravel with a maximum size of 25 mm, graded sand, filler and bitumen. These mixes generally have to be compacted after spreading and possess a certain stability. Normally they are applied under dry conditions.

Asphaltic concrete is normally used for revetments and it should therefore be impermeable and durable, have adequate stability to resist flow down a slope and be of good workability to facilitate compaction.

These requirements lead to a number of characteristics as follows.

a) The bitumen content will normally be higher, i.e. 6 % to 9 %, than for paving mixes in order to obtain good impermeability and durability.

b) Penetration grade bitumens 50 pen, 70 pen or 100 pen complying with BS 3690-1 are normally used to facilitate compaction, which is difficult on slopes, the softer grades being preferred.

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c) To minimize segregation, a continuous aggregate grading with a maximum stone size of 12 mm to 25 mm is commonly used. However, gap-graded mixes are sometimes used which need less compactive energy. Filler contents, i.e. materials smaller than 75 μm, are usually between 8 % and 13 %, limestone or cement being preferred. Aggregates should have good adhesion characteristics and be angular or crushed for revetments which are placed on slopes steeper than 1 : 2.5 and, to ensure that the mix is impermeable, i.e. K smaller than 10^{-6} mm/s, the void content should be below 4 %.

For mix design, use can be made of the Marshall test procedure [51]. It is, however, essential that the state of compaction achieved in the laboratory-made specimens be comparable with that obtained in the field.

The design procedure should be to select an aggregate grading, make mixes at varying bitumen contents and determine the void content after 5 to 10 blows. The aim should be to obtain a mix with minimum void content in the mineral aggregate and a total void content below 4 % with a bitumen content between 6 % and 9 %.

When a suitable mix has been selected, an uncompacted sample should be subjected to a flow test at the compaction temperature. A suitable form of test is to place the mix in a wooden form held at the same slope as the structure. The mix should remain in the form for half an hour without flow. A similar test should be carried out on a compacted specimen kept at a temperature of 40 °C to 70 °C, as appropriate, for 48 h. Any movement should be small, i.e. 3 mm maximum over a length of 300 mm, and should cease after 24 h. In both tests the sample depths should be representative of those to be used in the field.

Asphaltic concrete is normally made in a hot-mix plant and transported to the site by trucks or lorries. Various methods are used for spreading the mixture over the revetment. The simplest is by dumping the mix directly from a truck on the subsoil and spreading it to the desired thickness with hand rakes. Another possibility is to dump the mix into a shallow container, from which it is picked up and divided over the revetment by means of a crane, spreading usually being carried out by hand.

A modern trend is to lay asphaltic-concrete revetments 200 mm to 300 mm thick, in one layer, the thicker layer retaining heat and thus facilitating compaction. Further-more, problems related to adhesion between different layers are thereby eliminated.

66.3.2 *Sand mastic.* Sand mastic consists of sand filler and bitumen. The voids in the sand filler mixture are overfilled with bitumen so that the mix is pourable in its hot state and does not require any compaction. Larger aggregate is sometimes added but this has no effect on the viscous behaviour of this type of mix.

Sand mastic is used above and under water, either as a grouting material for stone revetments or as a carpet. This material can also be used for the construction of pre-fabricated mattresses for toe protection.

The composition of sand mastic can vary within wide limits and thus extensive use of local materials can be made. It should, however, comply with the following two requirements.

a) During application the mastic should be pourable and its viscosity low enough to permit sufficient penetration into the interstices between the stones but high enough to prevent excessive flow when applied hot on a slope.

b) During service conditions, the viscosity should be high enough, i.e. above about 1 MPa s to 1 GPa s, to keep long term flow within acceptable limits.

For a number of applications, it is desirable to limit the penetration depth of the sand mastic grout without affecting the viscosity appreciably and this can be done by adding fine rounded gravel to the mix.

The sand mastic for grouting purposes is usually made in a hot-mix plant and transported to the site in 5 t special containers or transporters, fitted with stirrers to prevent settlement of the mineral particles. At the site the hot mastic is poured through open channels or chutes and guided over the stone surface and into the voids with the aid of squeegees or shovels. In some cases the containers are lifted from the truck by crane so that the mix can flow directly into the work.

It is possible to apply sand mastic in deep water using a special apparatus which can lay an impermeable sand mastic carpet continuously on the seabed at depths of up to 30 m in 5 m wide strips. It is also possible to carry out stone grouting continuously and evenly at these depths. Prefabricated mattresses of sand mastic, which are easy to construct, can be used for toe protection, but in order to make transport and placing possible they are often reinforced with wire netting and steel cables.

66.3.3 *Stone asphalt.* Stone asphalt consists of bitumen and a mineral aggregate having a large maximum stone size and is characterized by a double mixing procedure. First, a slightly overfilled asphaltic mixture is made in a normal hot mix plant and, secondly, this mix is blended with dried preheated large stones. Due to this special manufacturing process, stone asphalt possesses some unique properties. For example the aggregate is gap-graded and the mix is just about pourable and no compaction is required. Stone asphalt can be used above and below water and it can be applied as a monolithic revetment or as a grouting material for very large stones. Because of its high stability, stone asphalt can be applied on slopes in much thicker layers than normal asphaltic concrete.

The composition of stone asphalt can vary between certain limits, depending on the type of application and the possibility of making strong, open yet still durable revetments with a special stone asphalt is very promising.

Heavy stone asphalt can be considered as consisting of large stones (maximum 500 mm) in which the interstices are filled with a slightly overfilled asphaltic concrete, whereas light stone asphalt consists of smaller stones (maximum 75 mm) with mastic.

When the material is applied as a revetment on relatively steep slopes above and below water level, the stability, and therefore stone content, for that part above water should be greater, because higher temperatures are expected in this area. When stone asphalt is used as an open mix, the stone content should be increased again to create sufficiently large microvoids.

Where this material is used for surface grouting very large stones, the composition should be modified in such a way that sufficient penetration is obtained but at the same time limiting the depth of penetration to prevent loss of material.

66.3.4 *Lean sand asphalt.* Lean sand asphalt consists of locally available sand and 3 % to 6 % of bitumen, and is therefore a relatively cheap material. Generally, when applied under water, no compaction is carried out, so the void content is very high but even when dry applications are carried out and compacted, the void content is rather high in most cases, due to the single-sized grading of most sands and the low binder content.

Lean sand asphalt can be made in a conventional hot-mix plant and transported to the site by lorry, where it is dumped, either above or under water.

Despite the relatively high void content of lean sand asphalt, it is well able to resist the scouring effect of water up to velocities of 3 m/s. Because of the high void content, durability and bearing capacity are limited but if the material is applied correctly it can form a reasonably cheap substitute for conventional materials.

66.4 Uses of bituminous materials

66.4.1 *General.* The choice of the various materials available for different maritime protection works is summarized in Table 23.

66.4.2 *Revetments for dykes, closure dams, dunes and sea-walls.* These structures are listed in the title of this subclause in order of decreasing water-retaining function and of increasing protective function against wave attack and scour. Asphaltic concrete and sand mastic grouted stones are extensively used for these structures, asphaltic concrete mainly for sealing and for protection against heavy wave attack and in those cases where a heavy layer is needed to withstand uplift pressures.

The problem of uplift pressures is one of the main reasons for asphaltic concrete seldom being used below the high water level, the thickness required in some cases generally being uneconomic.

66.4.3 *Underwater seabed protection.* Sand mastic in the form of a carpet is a suitable material for the protection of sandy seabeds. In general, waves do not play a significant role in the loading conditions. Uplift pressures may develop when the carpet is applied in a closure gap of an estuary or sea arm.

66.4.4 *Groynes and breakwaters.* These structures are frequently the subject of heavy wave attack and in many cases they reach far into the open sea where the water depth can be considerable.

The cost of obtaining, transporting and placing large stones is often very high and it is therefore sometimes more economical to use smaller stone or rubble and to bind these together to form monolithic structures that will have sufficient mass to resist movement and displacement by wave action.

One of the most satisfactory methods of binding stones together is to apply a grouting with a bituminous mixture. The monolithic mass thus formed is not only heavy but also flexible and will accommodate considerable movement due to settlement without cracking.

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Table 23 — Uses of bituminous materials in maritime protection works

Types of bituminous materials	Dykes and closure dams	Dune protection and sea-walls	Protection of sea-bed, river beds, canal bottoms	Groynes	Breakwaters and harbour dams	Sills in closure gaps
Asphaltic concrete	Revetment above high water level	Revetment above high water level	—	Special cases (capping)	—	—
Sand mastic grouting	Revetment	Revetment	—	Revetment and capping	Moderately attacked revetment	Heavily attacked sill
Sand mastic carpet (placed in situ)	Toe protection	Toe protection	Plain or stone weighted or stone roughened	Toe protection	Toe protection	—
Prefabricated mattresses	Toe protection	Toe protection	Special cases	Toe protection	Toe protection	—
Light stone asphalt	Revetment	Revetment either direct or by grouting of heavy rubble	—	Revetment either direct or by grouting of heavy rubble	Grouting of heavy rubble	—
Heavy stone asphalt	—	Revetment	—	Revetment	Revetment	—
Lean sand asphalt	Core, fill retaining wall, filter layer	Fill retaining wall, filter layer	—	Core	Special cases (core) filter layer	Core retaining walls

67 Rubber and plastics

67.1 General. New synthetic and rubberized compounds are continuously being introduced and the many applications of these compounds have affected all aspects of civil engineering construction.

The impact of these compounds on maritime construction has been no less than elsewhere and some of the uses which have become well established are given in **67.2** and **67.3**.

67.2 Natural and synthetic rubbers

67.2.1 *Deterioration.* Natural rubber deteriorates under the action of sunlight and, unless it is to be kept in darkness, should always be compounded with antiozonants and other chemicals to give it a life comparable with synthetic rubber compounds.

67.2.2 *Fendering.* Because of its energy absorbing characteristics, rubber is used as a fendering material on quays and dolphins in order to reduce the forces between structure and berthing vessel, thereby minimizing costly impact damage and reducing significantly the basic cost of the structure.

Manufacturers of rubber fenders produce a variety of shapes of rubber section each of which has its own particular application. Selection of the fender is generally based on data produced by each manufacturer.

Fenders should be manufactured in rubber having the necessary resistance to abrasion, ultra-violet light and other natural weathering processes. Synthetic rubbers such as styrene-butadiene rubber or ethylene propylene diene rubbers are recommended and such synthetics usually have additives to enhance their properties and give the required durability.

67.2.3 *Jointing materials.* As rubber is flexible and impervious, it is used extensively to produce watertight membranes at movement joints between structural elements. Such membranes, commonly known as water stops, are produced by a number of manufacturers and are made of vulcanized rubber. Jointing of water stops on site has to be made with special vulcanizing adhesives.

67.2.4 *Hoses.* Flexible rubber hoses for fire fighting, bunkering and product transfer should be manufactured to comply with BS 1435.

67.2.5 *Rubber bearings.* Structural beams and decks which are subject to significant rolling loads such as cranes and container handling plant are often supported by rubber bearings which enhance the load distribution characteristics of the structure and allow freedom of movement for rotation and thermal expansion.

Crane rails are often bolted down on rubber pads which minimize wear and stresses in the rail and its supporting structure under rolling loads.

67.3 Plastics

67.3.1 *Anti-friction facings.* In order to reduce wear and tear to both vessels and structures, resulting from acute angle impacts, it is usual to face the sides of dock gates and other vulnerable elements with rubbing strips traditionally made of timber but which of late have been effectively replaced by similarly shaped sections extruded from hardwearing polymers.

Similar materials have also been developed with coefficients of friction of 2 % to 4 % which are much lower than for traditional timber rubbing strips.

67.3.2 *Bearings.* Plastics bearings are used in similar circumstances to those in which rubber bearings are used but, whereas rubber bearings usually permit lateral movements by shear distortion, the low coefficient of friction associated with plastics bearings is utilized to allow lateral movements by sliding, usually against a stainless steel plate. Rubber bearings are, however, more suitable in dirty conditions.

67.3.3 *Glass reinforced plastics (GRP).* GRP is a versatile material which can be used for a variety of applications and is generally used where light strong construction is required. Caution is required in its use because it:

a) cracks and tears if it is overstressed, having no capacity to receive load stress concentrations by plastic deformation and is thus not so resistant to impact losses as ductile materials;

b) has a low modulus of elasticity therefore deflections are relatively large;

c) is relatively expensive;

d) has a poor fatigue performance and loses strength with age;

e) is inflammable unless inhibitors, which reduce the strength, are added and care should be taken to ensure compliance with the appropriate fire regulations.

67.3.4 *Plastics fabrics and membranes.* Plastics porous fabrics are being used increasingly as filter membranes in order to prevent fine material being washed out of embankments and similar structures. Impermeable membranes are often laid on the bottom of canals to waterproof them and under in situ reinforced concrete paving slabs to prevent loss of grout from the concrete mix into the sub-base.

Plastics fabrics have the disadvantage that with a relative density of less than unity they float and like rubber they tend to deteriorate in sunlight.

68 Protective measures

68.1 General. When designing structures which are especially vulnerable to deterioration due to the aggressive environment in which they are sited, the life of the structure may be increased by allowing either for planned maintenance as recommended in BS 5493 or for increased initial structural strength which will give the required life without maintenance.

The detailing of maritime structures also has an important effect on the subsequent effects of corrosion and the recommendations on the selection of materials and workmanship set down in clause **59** will minimize corrosion. Reference should also be made to Appendix A of BS 5493:1977 regarding detailing to minimize corrosion.

When selecting a protective system, the following factors should always be taken into consideration.

a) The costs of protective measures are repetitive in that the protective materials themselves deteriorate and regular maintenance and renewal of coatings has to be discounted to present day values in any cost analysis carried out.

b) The cost of down-time of a maritime installation can be significant and it is important to estimate the effective value of the facility during any periods when it has to be put out of commission for maintenance of protective coatings or equipment.

c) Corrosion does not proceed at a uniform rate over the whole structure and at certain corrosion points loss of the original material can be rapid. Thus any estimate of a corrosion allowance for extra material is bound to be excessive on some parts of the structure while being inadequate on others (see **61.3.4** and Table 22).

d) The cost of renewing a protective system can be much more than initial protection, even without allowing for monetary inflation. This is because marine growth and old paint have to be removed prior to renewal of the system and access is usually more difficult than during construction.

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e) Marine growth is prevalent on structures below mean high-water level. Evidence exists that fouling is protective against corrosion and should not, therefore, be removed as it is more effective and durable than any paint system that might replace it. The only exposure zones which might usefully be painted are the splash and atmospheric zones.

68.2 Coating systems. Sections two and three of BS 5493:1977 give valuable guidance on the choice and specification of coating systems available for maritime structures. Additional factors influencing the choice of coating are as follows.

a) Consideration should be given to primer coatings which can protect the steel for extended construction periods and can withstand abrasion associated with handling and fabrication with a minimum of damage.

b) Preference should be given to coating systems which can be applied and maintained with conventional and readily available application equipment.

c) For structures in locations where coatings are exposed to low temperatures, preference should be given to coatings which dry by evaporation of solvents and do not involve use of catalysts.

d) For deck areas of maritime structures exposed to weathering, abrasion associated with cargo handling operations and spills of diesel fuels, lubricants and corrosive compounds, consideration should be given to coatings which have high impact resistance and resistance to spills of solvents and corrosive chemicals, i.e. the thermosetting materials.

68.3 Concrete protection. Because of the low electrical conductivity of concrete, its alkalinity and its ability to exclude oxygen from the surface of the metal, it is common to encase steel members in the splash zone with a layer of dense concrete, usually reinforced with a light, welded cage of steel mesh. This method of splash zone protection is particularly applicable to structures where tidal ranges are small, i.e. 1.5 m to 3.0 m, and installation costs are acceptable.

The placing of concrete for protection is often done under-water by use of a tremie inside prefabricated shutters. In all cases, careful cleaning of the steel piles should be carried out immediately prior to placing of the concrete.

In some cases, special woven polypropylene jackets are tied around piles and very wet concrete poured into the jackets, thus displacing the water inside. A hydrostatic head is maintained on the concrete within the woven jackets which forces the excess water from the mix through the weave of the jacket material. A very dense concrete can result from this procedure.

68.4 Monel 400 sheathing. Monel 400 is an alloy (Ni 67 %, Cu 28 %, Fe 2.5 % max. and Mn 2 %) which is as strong, tough and ductile as steel and resists corrosion better than copper, gunmetal or bronze. It machines readily and can be rolled, drawn, cast, forged, soldered, brazed and welded. It is used for sheathing other members but is expensive. Where Monel 400 sheathing is used to clad steel structural members, the possibility of galvanic corrosion of the steel should be considered as noted in PD 6484.

68.5 Steel wear plates. Sufficient additional thickness of steel may be provided in the splash zone to compensate for the combined effects of corrosion and wear during the life of the structure. In addition to providing a corrosion allowance, wear plates add stiffness and strength, thereby providing greater impact resistance. Wear plates are typically coated with the same protective coating system used for the remainder of the structure.

68.6 Wrappings

68.6.1 *General.* Wrappings are described in section two of BS 5493:1977 but in connection with maritime work points given in **68.6.2** to **68.6.5** require special consideration.

68.6.2 *Structural members.* Structural members can be wrapped with glass fibre impregnated with a water repellant compound of good adhesive properties. Hessian reinforced wrappings should be used with caution as they are liable to microbiological attack.

68.6.3 *Wrappings.* The wrappings can be applied to structural members, which are preferably cylindrical, prior to installation below water or, in case of maintenance, they are applied by divers. In the latter case, careful surface cleaning is essential if adhesion is to take place between the waterproofing compound and the structural member.

68.6.4 ***Waterproofing compounds used for wrapping tapes***

68.6.4.1 *General.* The wrapping tapes described in **68.6.4.2** to **68.6.4.6** have been used extensively for the protection of tie rods on anchored, sheet piled structures. However, systems are being developed for cable ties of high tensile engineering cable protected by a polypropylene sleeve applied during manufacture.

68.6.4.2 *Petroleum jelly and neutral mineral filler.* Petroleum jelly and neutral mineral filler are used in conjunction with a petroleum jelly primer. This material is suitable for electrical insulation but is subject to damage by abrasion. Therefore, tapes impregnated with this compound should not be used where they are liable to impact, such as near the berthing face of a maritime structure.

68.6.4.3 *Synthetic resin or plastics tapes.* Synthetic resin or plastics tapes consist of PVC or polyethylene tapes 125 mm to 250 mm wide with a fabric core, coated on one side with a contact adhesive, normally of synthetic rubber base. This material is only suitable for dry application prior to installation of the structural member and is rarely, if ever, used in maritime structures.

68.6.4.4 *Coal tar and bitumen.* Coal tar and bitumen have a high resistance to water and good adhesion to steel and are usually applied after priming with coal tar or bitumen primer.

68.6.4.5 *Two-pack epoxy or polyester compounds.* Two-pack epoxy or polyester compounds can be applied with glass fibre tape. This protective treatment is very suitable for maritime protection both prior to installation of structural members and for long term maintenance. The tape is wrapped around the member immediately after it has been cleaned and primed with a two-pack resin compound and then several layers of resin are applied over the tape and the final surface trowelled smooth.

68.6.4.6 *Plastics sleeves.* Plastics sleeves are sometimes shrunk on to steel pipes to provide protection and a system is now being developed where a polypropylene or glass fibre sleeve of larger diameter than the pipe to be protected has the annulus between it and the pipe filled with expanded polyurethane foam.

68.6.5 *Special measures.* In flowing water, special measures have to be taken to prevent wrappings being torn off by turbulence.

68.7 Cathodic protection. The electrochemical processes which accompany corrosion of submerged structural elements in seawater are described in CP 1021, which also gives details of the way in which cathodic protection, both by sacrificial galvanic anodes or by impressed current systems, should be applied to combat corrosion.

68.8 Protection in the buried zone. Structural elements in this zone are those below seabed level, minus an allowance for scour and over-dredging, or those within structural earthworks such as soil dykes.

Where structural elements are driven into undisturbed ground below seabed or river bed level the rates of corrosion are negligible, irrespective of the presence of anoerobic sulphate-reducing bacteria. However, in filled ground, especially with certain clays, bacterial corrosion can occur locally and may be rapid.

The presence of such bacteria is rare and can be detected by taking measurements of the redox potential of the soil (see Appendix A of CP 1021:1973). By provision of adequate cathodic protection this form of corrosion can be eliminated.

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Appendix A Physical properties of commonly stored cargoes

Typical values of bulk densities and angles of repose are given in Table 24. Typical values of stacked densities are given in Table 25. These tables may be used in conjunction with the recommendations of clause **44**.

Table 24 — Typical dry bulk densities and angles of repose

Material	Dry bulk density	Angle of repose
	t/m^3	degrees
Ores		
Iron (Limonite)	2.24 to 3.00	35 to 40
Copper (Copper pyrites)	2.56	38 to 45
Lead (Galena)	2.56 to 2.76	35 to 40
Zinc (Zincblende)	1.50 to 1.79	38
Aluminium (Bauxite)	1.33	28 (when dry) 49 (in 8 % moisture)
Tin (Cassiterite)	1.63 to 1.99	35 to 38
Chromium (Chromic iron)	2.39 to 2.56	33 to 40
Magnesium (Magnesite)	1.44	35
Manganese (Manganite)	1.79 to 2.39	35 to 45
Basic chemicals		
Sulphur	1.12 to 1.20	35 to 40
Phosphate rock	1.03	30 to 34
Kaolin	0.90 to 0.94	30 to 35
Solid fuels		
Coal	0.72 to 0.90	30 to 45
Coke	0.36 to 0.51	37
Building materials		
Natural aggregates	1.28 to 1.60	30 to 40
Granite (chippings)	1.20 to 1.24	35
Sand	1.79 to 1.89	30 to 40
Limestone	1.63	34
Waste products		
Domestic refuse	0.56	
Scrap iron	1.0 to 1.6	35
Foodstuffs (normally stored in sheds or silos)		
Cereal	0.51 to 0.76	40
Sugar	0.78	40
Salt	0.90	45
Soya bean	0.82	35 to 60
Copra	0.51	35

Table 25 —Typical stacked densities for common commodities

Commodity	Stacked density
	t/m^3
Timber	
Softwood: Douglas fir	0.61
Hardwood: Oak	0.83
Greenheart	1.14
Timber products	
Paper (in bales)	0.80
Linerboard (in reels)	0.65
Chemical products	
Petroleum products (in barrels)	0.41 to 0.51
Fertilisers (in bags)	0.84 to 0.94
Foodstuffs	
Beers (in casks)	0.66
Dry sugar (in bags)	0.78
Tea	0.32 to 0.38
Potatoes (in bags)	0.72
Copra (in bags)	0.38
Soya beans (in bags)	0.72
Flour (in bags)	0.83
Metal products	
Aluminium ingots	1.24
Copper ingots	3.00 to 3.59
Copper coils	1.12
Steel bars	2.24 to 3.00
Pig steel	3.00 to 3.59
Steel coils	1.20 to 3.00

Appendix B Method of test for soundness of aggregate by use of sodium sulphate or magnesium sulphate

B.1 Principle. This method covers the testing of aggregates to determine their resistance to disintegration by saturated solutions of sodium sulphate or magnesium sulphate. It is based on the method described in ASTM C88 adapted to British practice.

The test provides information helpful in judging the soundness of aggregates subject to weathering, particularly in a maritime environment and when adequate information is not available from performance under similar conditions. It should be noted that test results obtained by use of the two salts differ considerably and hence it is essential to apply the appropriate criteria, [see **57.2** e)].

B.2 Reagents

B.2.1 *Sulphate detection.* A suitable laboratory solution of barium chloride ($BaCl_2$) shall be provided for proving the absence of sulphates in the washing process (see **B.7.8**).

B.2.2 ***Special solutions required***

B.2.2.1 *General.* Prepare the solution for immersion of test samples from either sodium or magnesium sulphate in accordance with **B.2.2.2** or **B.2.2.3**.

NOTE Some aggregates containing carbonates of calcium or magnesium are attacked chemically by fresh sulphate solution, resulting in erroneously high measured losses. If this condition is encountered or is suspected, repeat the test using a filtered solution which has been used previously to test the same type of carbonate rock, provided that the solution complies with **B.2.2.2** and **B.2.2.3** for specific gravity.

The volume of the solution shall be at least five times the solid volume of all samples immersed at any one time.

B.2.2.2 *Sodium sulphate solution.* Prepare a saturated solution of sodium sulphate by dissolving a general purpose reagent (GPR) or equal grade of the salt in water at a temperature of 25 °C to 30 °C. Add sufficient salt of either the anhydrous ($Na_2 SO_4$) or the crystalline ($Na_2 SO_4 10H_2O$) form, to ensure not only saturation but also the presence of excess crystals when the solution is ready for use in the tests.

NOTE 1 For the solution, 215 g of anhydrous salt or 700 g of the decahydrate per litre of water are sufficient for saturation at 22 °C. However, since these salts are not completely stable and since it is desirable that an excess of crystals be present, the use of not less than 350 g of the anhydrous salt or 750 g of the decahydrate salt per litre of water is recommended.

NOTE 2 The decahydrate sodium sulphate presents difficulties in compounding the required solution on account of its cooling effect on the solution.

Thoroughly stir the mixture during the addition of the salt and stir the solution at frequent intervals until used. To reduce evaporation and prevent contamination, keep the solution covered at all times when access is not needed. Allow the solution to cool to 21 ± 1 °C. Again stir and allow the solution to remain at the designated temperature for at least 48 h before use. Prior to each use, break up the salt cake, if any, in the container, stir the solution thoroughly and determine the specific gravity of the solution. When used, the solution shall have a relative density not less than 1.151 nor more than 1.174. Discard a discoloured solution or filter it and check for relative density.

B.2.2.3 *Magnesium sulphate solution.* Prepare a saturated solution of magnesium sulphate by dissolving a GPR or equal grade of the salt in water at a temperature of 25 °C to 30 °C. Add sufficient salt of either the anhydrous ($MgSO_4$) or the crystalline ($MgSO_4 7H_2O$) (Epsom salt) form, to ensure saturation and the presence of excess crystals when the solution is ready for use in the tests.

NOTE For the solution 350 g of anhydrous salt or 1 230 g of the heptahydrate per litre of water are sufficient for saturation at 23 °C. However, since these salts are not completely stable, with the hydrous salt being the more stable of the two, and since it is desirable that an excess of crystals be present, it is recommended that the heptahydrate salt be used and in an amount of not less than 1 400 g per litre of water.

Thoroughly stir the mixture during the addition of the salt and stir the solution at frequent intervals until used. To reduce evaporation and prevent contamination, keep the solution covered at all times when access is not needed. Allow the solution to cool to 21 ± 1 °C. Again stir and allow the solution to remain at the designated temperature for at least 48 h before use. Prior to each use, break up the salt cake, if any, in the container, stir the solution thoroughly and determine the specific gravity of the solution. When used, the solution shall have a relative density not less than 1.295 nor more than 1.308. Discard a discoloured solution or filter and check for relative density.

B.3 Apparatus

B.3.1 *Test sieves,* complying with BS 410 with aperture sizes appropriate to the range of sizes of the aggregate selected from the following.

a) *Woven wire:* 2.36 mm, 1.18 mm, 600 µm and 300 µm.

b) *Perforated plate* (square holes): 125 mm, 106 mm, 90 mm, 75 mm, 63 mm, 50 mm, 37.5 mm, 28.0 mm, 20.0 mm, 14.0 mm, 10.0 mm and 5.0 mm.

Additional sieves with aperture sizes of 31.5 mm, 16.0 mm, 8.0 mm and 4.0 mm shall be used to determine losses (see **B.7**).

B.3.2 *Containers,* for immersing the samples of aggregate in the solution perforated in such a manner as to permit free access of the solution to the sample and drainage of the solution from the sample without loss of aggregate.

NOTE Baskets made of suitable wire mesh or sieves with suitable openings are satisfactory containers for the samples.

B.3.3 *Temperature regulation,* for regulating the temperature of the samples during immersion in the sodium sulphate or magnesium sulphate solution.

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B.3.4 *Balances,* of the following sizes:

a) for fine aggregate, a balance or scale accurate to within 0.1 g over the range required for this test;

b) for coarse aggregate, a balance or scale accurate to within 0.1 % or 1 g, whichever is greater, over the range required for this test.

B.3.5 *Drying oven,* capable of being heated continuously at 110 ± 5 °C and the rate of evaporation, at this range of temperature, shall be at least 25 g/h for 4 h, during which period the doors of the oven shall be kept closed. This rate shall be determined by the loss of water from 1 L low-form beakers, each initially containing 500 g of water at a temperature of 21 ± 2 °C, placed at each corner and the centre of each shelf of the oven. The evaporation requirement is to apply to all test locations when the oven is empty except for the beakers of water.

B.3.6 *Hydrometers,* complying with BS 718, or a suitable combination of graduated glassware and balance, capable of measuring the solution relative density within ± 0.0001.

B.4 Samples

B.4.1 *Fine aggregate.* Fine aggregate for the test shall be passed through a 5.00 mm sieve. The sample shall be of such size that it will yield not less than 100 g of each of the following sizes, which shall be available in amounts of 5 % or more, expressed in terms of the sieves given in Table 26.

Table 26 — Sieve sizes for fine aggregate fractions

Passing sieve	Retaining sieve
5.00 mm	2.36 mm
2.36 mm	1.18 mm
1.18 mm	600 µm
600 µm	300 µm

B.4.2 *Coarse aggregate.* Coarse aggregate for the test shall consist of material from which the sizes finer than the 5.00 mm sieve have been removed. The sample shall be of such a size that it will yield the amounts of the indicated sizes shown in Table 27 that are available in amounts of 5 % or more.

Table 27 — Sample mass for coarse aggregate fractions

Size	Mass
mm	g
10 to 5	300 ± 5
20 to 10	1 000 ± 10
consisting of:	
14 to 10;	350 ± 5
20 to 14.	650 ± 5
37.5 to 20	1 500 ± 50
consisting of:	
28 to 20;	500 ± 30
37.5 to 28.	1 000 ± 50
63 to 37.5	5 000 ± 300
consisting of:	
50 to 37.5;	2 000 ± 200
63 to 50.	3 000 ± 300
Larger sizes by fractions sized by successive sieves detailed in **B.3.1** each fraction	7 000 ± 1 000

B.4.3 *Combined fine and coarse aggregates.* When an aggregate to be tested contains appreciable amounts of both fine and coarse material, having a grading with more than 10 % (*m*/*m*) coarser than the 10.0 mm sieve and also, more than 10 % (*m*/*m*) finer than the 5.00 mm sieve separate samples of the minus 5 mm fraction and the plus 5 mm fraction shall be tested in accordance with the procedures for fine aggregate and coarse aggregate, respectively. The results shall be reported separately for the fine aggregate fraction and the coarse aggregate fraction, giving the percentages of the coarse and fine size fractions in the initial grading.

B.5 Preparation of test sample

B.5.1 *Fine aggregate.* Thoroughly wash the sample of fine aggregate on a 300 µm sieve, dry to constant mass at 110 ± 5 °C and separate into the different sizes by sieving, as follows. Make a rough separation of the graded sample by means of a nest of the standard sieves specified in **B.3.1**. From the fractions obtained in this manner, select samples of sufficient size to yield 100 g after sieving to refusal.

NOTE In general, a 110 g sample will be sufficient.

Do not use fine aggregate sticking in the meshes of the sieves in preparing the samples. Weigh samples consisting of 100 ± 0.1 g out of each of the separated fractions after final sieving and place in separate containers for the test.

B.5.2 *Coarse aggregate.* Thoroughly wash and dry the sample of coarse aggregate to constant mass at 110 ± 5 °C and separate it into the different sizes shown in **B.4.2** by sieving to refusal. Weigh out quantities of the different sizes within the tolerances of **B.4.2** and where the test portion consists of two sizes, combine them to the designated total weight. Record the masses of the test samples and their fractional components. In the case of sizes larger than 20 mm, record the number of particles in the test samples.

B.6 Procedure

B.6.1 *Storage of samples of solution.* Immerse the samples in the prepared solution of sodium sulphate or magnesium sulphate for not less than 16 h nor more than 18 h in such a manner that the solution covers them to a depth of at least 12 mm.

NOTE Suitably weighted wire grids placed over the sample in the containers will permit this coverage to be achieved with very lightweight aggregates.

Cover the containers to reduce evaporation and prevent the accidental addition of extraneous substances. Maintain the samples immersed in the solution at a temperature of 21 ± 1 °C for the immersion period.

B.6.2 *Drying samples after immersion.* After the immersion period, remove the aggregate sample from the solution, permit it to drain for 15 ± 5 min and place in the drying oven. The temperature of the oven shall have been brought previously to 110 ± 5 °C. Dry the samples at the specified temperature until constant mass has been achieved. Establish the time required to attain constant mass as follows. With the oven containing the maximum sample load expected, check the mass losses of test samples by removing and weighing them, without cooling, at intervals of 2 h to 4 h. Make enough checks to establish required drying time for the least favourable oven location (see **B.3.5**) and sample condition.

NOTE Drying time required to reach constant mass may vary considerably for several reasons. Efficiency of drying will be reduced as cycles accumulate because of salt adhering to particles and, in some cases, because of increase in surface area due to breakdown. The different size fractions of aggregate will have differing drying rates. The smaller sizes will tend to dry more slowly because of their larger surface area and restricted interparticle voids, but this tendency may be altered by the effects of container size and shape.

Constant mass shall be considered to have been achieved when mass loss is less than 0.1 % of sample mass in 4 h of drying. After constant mass has been achieved, allow the samples to cool to room temperature, when they shall again be immersed in the prepared solution as described in **B.6.1**.

B.6.3 *Number of cycles.* Repeat the process of alternate immersion and drying until the required number of cycles is obtained.

B.7 Quantitative examination. After the completion of the final cycle and after the sample has cooled, wash the sample free from the sodium sulphate or magnesium sulphate as determined by the reaction of the wash water with barium chloride ($BaCl_2$). Wash by circulating water at 43 ± 6 °C through the samples in their containers. This may be done by placing them in a tank into which the hot water can be introduced near the bottom and allowed to overflow. In the washing operation, do not subject the samples to impact or abrasion which may tend to break up particles.

After the sodium sulphate or magnesium sulphate has been removed, dry each fraction of the sample to constant mass at 110 ± 5 °C. Sieve the fine aggregate over the same sieve on which it was retained before the test and sieve the coarse aggregate over the sieve shown in Table 28 for the appropriate size of particle. For fine aggregate the method and duration of sieving shall be the same as were used in preparing the test samples. For coarse aggregate, sieving shall be by hand, with agitation sufficient only to ensure that all undersize material passes the designated sieve. No extra manipulation shall be employed to break up particles or cause them to pass the sieves. Weigh the material retained on each sieve and record each amount. The difference between each of these amounts and the initial mass of the fraction of the sample tested is the loss in the test and is to be expressed as a percentage of the initial mass.

Table 28 —Sieve size to determine loss

Size of aggregate	Sieve used to determine loss
mm	mm
63 to 37.5	31.5
37.5 to 20	16.0
20 to 10	8.00
10 to 5	4.00

B.8 Qualitative examination of sizes larger than 20 mm. Separate the particles of each test sample into groups according to the action produced by the test.

NOTE Many types of action may be expected. In general, they may be classified as disintegration, splitting, crumbling, cracking, flaking, etc. While only particles larger than 20 mm in size are required to be examined qualitatively, it is recommended that examination of the smaller sizes be made in order to determine whether there is any evidence of excessive splitting.

Record the number of particles showing each type of distress.

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B.9 Report. The report shall include the following data.

a) Mass of each fraction of each sample before test.

b) Material from each fraction of the sample finer than the sieve designated in **B.7** for sieving after test, expressed as a percentage of the original mass of the fraction.

c) Weighted average calculated from the percentage of loss for each fraction, based on the grading of the sample as received for examination or, preferably, on the average grading of the material from that portion of the supply of which the sample is representative except that:

1) for fine aggregate, i.e. with less than 10 % coarser than 5.00 mm sieve, sizes finer than the 300 μm sieve shall be assumed to have 0 % loss and sizes coarser than the 5.00 mm sieve shall be assumed to have the same loss as the next smaller size for which test data are available;

2) for coarse aggregate, i.e. with less than 10 % finer than the 5.00 mm sieve, sizes finer than the 5.00 mm sieve shall be assumed to have the same loss as the next larger size for which test data are available;

3) for an aggregate containing appreciable amounts of both fine and coarse material tested as two separate samples as required in **B.4.3**, the weighted average losses shall be computed separately for the minus 5 mm and plus 5 mm fractions based on recomputed gradings considering the fine fraction as 100 %, and the results shall be reported separately giving the percentage of the minus 5 mm and plus 5 mm material in the initial grading;

4) for the purpose of calculating the weighted average, any sizes in **B.4.1** or **B.4.2** that contain less than 5 % of the sample shall be considered to have the same loss as the average of the next smaller and the next larger sizes, or if one of these sizes is absent, to have the same loss as the next larger or next smaller size, whichever is present.

d) In the case of particles coarser than 20 mm before test:

1) the number of particles in each fraction before test;

2) the number of particles affected, classified as to number disintegrating, splitting, crumbling, cracking, flaking, etc.

e) Type of solution, i.e. sodium sulphate or magnesium sulphate, and whether the solution was freshly prepared or previously used.

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