Maritime structures —

Part 1: Code of practice for general criteria

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Contents

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Foreword

This revison of BS 6349-1 has been prepared under the direction of the Technical Sector Committee for Building and Civil Engineering (B/-) and supersedes BS 6349-1:1984, which is withdrawn.

BS 6349-1:1984 was published in advance of the subsequent parts of the code, which are listed below. In the new edition subject matter that is now duplicated or dealt with more fully in other parts of the code has been curtailed or deleted. All sections of part 1 have been updated in order to take account of new developments or increased knowledge and all amendments previously issued by BSI have been incorporated.

In part 1 recommendations are given to assist clients and engineers to obtain the basic data relevant to the design of any maritime structure.

Offshore structures and structures in inland waters are not covered by this code, although certain aspects might be relevant to such projects.

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

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

In general, reference is made to British Standards and not to European Standards, because most relevant European Standards are still in the pre-standard (ENV) form. Only in the case of structural steel, for which ENs have been published, is reference made to European Standards (see clause **59**).

This code of practice contains information and guidance for engineers and recommendations on good practice. As such, conformity with its recommendations is not obligatory and variations from its recommendations might well be justified in special circumstances. Engineering judgement should therefore be applied to determine when the recommendations of the code should be followed and when they should not.

This code of practice is intended for use by engineers who have some knowledge of the subject. It embodies the experience of engineers successfully engaged in the design and construction of the particular class of works, so that other reasonably qualified engineers can use it as a basis for the design of similar works.

A code of practice represents good practice at the time it is written and, inevitably, technical developments can render parts of it obsolescent in time. It is the responsibility of engineers concerned with the design and construction of works to remain conversant with developments in good practice that have taken place subsequent to the publication of this code.

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.

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The standard is issued in seven parts as follows:

— *Part 1: General criteria;*

— *Part 2: Design of quay walls, jetties and dolphins;*

— *Part 3: Design of dry docks, locks, slipways and shipbuilding berths, shiplifts and dock and lock gates;*

— *Part 4: Code of practice for design of fendering and mooring systems;*

— *Part 5: Code of practice for dredging and land reclamation;*

— *Part 6: Design of inshore moorings and floating structures;*

— *Part 7: Guide to the design and construction of breakwaters.*

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Summary of pages

This document comprises a front cover, an inside front cover, pages i to x, pages 1 to 189 and a back 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 maintenance of structures set in the maritime environment.

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 because 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 Normative references

The following normative documents contain provisions, which, through reference in this text, constitute provisions of this part of this British Standard. For dated references, subsequent amendments to, or revisions of, any of these publications do not apply. For undated references, the latest edition of the publication referred to applies.

BS 11, *Specification for railway rails.*

BS 12, *Specification for ordinary and rapid-hardening Portland cement.*

BS 105, *Light and heavy bridge-type railway rails.*

BS 146, *Portland blastfurnace cement.*

BS 500, *Steel railway sleepers for flat bottom rails.*

BS 812 (all parts), *Testing aggregrates.*

BS 882, *Specification for aggregates from natural sources for concrete.*

BS 1211, *Specification for centrifugally cast (spun) iron pressure pipes for water, gas and sewage.*

BS 1370, *Specification for low heat Portland cement.*

BS 1400, *Specification for copper alloy ingots and copper alloy and high conductivity copper castings.*

BS 1579, *Specification for connectors for timber.*

BS 1881-124, *Testing concrete — Part 124: Methods for analysis of hardened concrete.*

BS 2870, *Specification for rolled copper and copper alloys: sheet, strip and foil.*

BS 2871, *Specification for copper and copper alloys — Tubes.*

BS 3148, *Methods of tests for water for making concrete (including notes on the suitability of the water).*

BS 3410, *Specification for metal washers for general engineering purposes.*

BS 3468, *Specification for austenitic cast iron.*

BS 3892-1, *Pulverized-fuel ash — Part 1: Specification for pulverized-fuel ash for use as with Portland cement.*

BS 4027, *Specification for sulphate-resisting Portland cement.*

BS 4246, *Specification for high slag blastfurnace cement.*

BS 4449, *Specification for carbon steel bars for the reinforcement of concrete.*

BS 4466, *Specification for scheduling, dimensioning, bending and cutting of steel reinforcement for concrete.*

BS 4482, *Specification for cold reduced steel wire for the reinforcement of concrete.*

BS 4483, *Steel fabric for the reinforcement of concrete.*

BS 4844, *Specification for abrasion resisting white cast irons.*

BS 4976, *Specification for polytetrafluoroethylene (PTFE) tubing extruded from coagulated dispersion powder (metric and imperial units).*

BS 5268-2, *Structural use of timber — Part 2: Code of practice for permissible stress design, materials and workmanship.*

BS 5290, *Determination of lead content of gasoline — Volumetric chromate method.*

BS 5328-2, *Concrete — Part 2: Methods for specifying concrete mixes.*

BS 5391-1, *Specification for acrylonitrile-butadiene-styrene (ABS) pressure pipe — Part 1: Pipe for industrial uses.*

BS 5392, *Specification for acrylonitrile-butadiene-styrene (ABS) fittings for use with ABS pressure pipe.*

BS 5400-2, *Steel, concrete and composite bridges — Part 2: Specification for loads.*

BS 5409-1, *Specification for nylon tubing — Part 1: Fully plasticized nylon tubing types 11 and 12 for use primarily in pneumatic installations.*

BS 5480, *Specification for glass reinforced plastics (GRP) pipes and fittings for use for water supply or sewerage.*

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BS 5567, *Specification for apertures in tinplate containers to receive plug-in plastics closures.*

BS 5589, *Code of practice for preservation of timber.*

BS 5996, *Specification for acceptance levels for internal imperfections in steel plate, strip and wide flats, based on ultrasonic testing.*

BS 6399-1, *Loading for buildings — Part 1: Code of practice for dead and imposed load.*

BS 6399-2, *Loading for buildings — Part 2: Code of practice for wind loads.*

BS 6588, *Specification for Portland pulverized-fuel ash cement.*

BS 6699, *Specification for ground granulated blastfurnace slag for use with Portland cement.*

BS EN 485-1, *Aluminium and aluminium alloys — Sheet, strip and plate — Part 1: Technical conditions for inspection and delivery.*

BS EN 485-2, *Aluminium and aluminium alloys — Sheet, strip and plate — Part 2: Mechanical properties.*

BS EN 485-3, *Aluminium and aluminium alloys — Sheet, strip and plate — Part 3: Tolerances on shape and dimensions for hot-rolled products.*

BS EN 485-4, *Aluminium and aluminium alloys — Sheet, strip and plate — Part 4: Tolerances on shape and dimensions for cold-rolled products.*

BS EN 499, *Welding consumables — Covered electrodes for manual metal arc welding of non alloy and fine grain steels — Classification.*

BS EN 515, *Aluminium and aluminium alloys — Wrought products — Temper designations.*

BS EN 573-1, *Aluminium and aluminium alloys — Chemical composition and form of wrought products — Part 1: Numerical designation system.*

BS EN 573-2, *Aluminium and aluminium alloys — Chemical composition and form of wrought products — Part 2: Chemical symbol based designation system.*

BS EN 573-3, *Aluminium and aluminium alloys — Chemical composition and form of wrought products — Part 3: Chemical composition.*

BS EN 573-4, *Aluminium and aluminium alloys — Chemical composition and form of wrought products — Part 4: Forms of products.*

BS EN 586-1, *Aluminium and aluminium alloys — Forgings — Part 1: Technical conditions for inspection and delivery.*

BS EN 586-2, *Aluminium and aluminium alloys — Forgings — Part 2: Mechanical properties and additional property requirements.*

BS EN 603-1, *Aluminium and aluminium alloys — Wrought forging stock — Part 1: Technical conditions for inspection and delivery.*

BS EN 603-2, *Aluminium and aluminium alloys — Wrought forging stock — Part 2: Mechanical properties.*

BS EN 604-1, *Aluminium and aluminium alloys — Cast forging stock — Part 1: Technical conditions for inspection and delivery.*

BS EN 754-1, *Aluminium and aluminium alloys — Cold drawn rod/bar and tube — Part 1: Technical conditions for inspection and delivery.*

BS EN 754-2, *Aluminium and aluminium alloys — Cold drawn rod/bar and tube — Part 2: Mechanical properties.*

BS EN 754-7, *Aluminium and aluminium alloys — Cold drawn rod/bar and tube — Part 7: Seamless tubes, tolerances on dimensions and form.*

BS EN 754-8, *Aluminium and aluminium alloys — Cold drawn rod/bar and tube — Part 8: Porthole tubes, tolerances on dimensions and form.*

BS EN 755-1, *Aluminium and aluminium alloys — Extruded rod/bar, tube and profiles — Part 1: Technical conditions for inspection and delivery.*

BS EN 755-2, *Aluminium and aluminium alloys — Extruded rod/bar, tube and profiles — Part 2: Mechanical properties.*

BS EN 755-3, *Aluminium and aluminium alloys — Extruded rod/bar, tube and profiles — Part 3: Round bars, tolerances on dimensions and form.*

BS EN 755-4, *Aluminium and aluminium alloys — Extruded rod/bar, tube and profiles — Part 4: Square bars, tolerances on dimensions and form.*

BS EN 755-5, *Aluminium and aluminium alloys — Extruded rod/bar, tube and profiles — Part 5: Rectangular bars, tolerances on dimensions and form.*

BS EN 755-6, *Aluminium and aluminium alloys — Extruded rod/bar, tube and profiles — Part 6: Hexagonal bars, tolerances on dimensions and form.*

BS EN 755-7, *Aluminium and aluminium alloys — Extruded rod/bar, tube and profiles — Part 7: Seamless tubes, tolerances on dimensions and form.*

BS EN 755-8, *Aluminium and aluminium alloys — Extruded rod/bar, tube and profiles — Part 8: Porthole tubes, tolerances on dimensions and form.*

BS EN 932-6, *Tests for general properties of aggregates — Part 6: Definitions of repeatability and reproducibility.*

BS EN 1301-1, *Aluminium and aluminium alloys — Drawn wire — Part 1: Technical conditions for inspection and delivery.*

BS EN 1301-2, *Aluminium and aluminium alloys — Drawn wire — Part 2: Mechanical properties.*

BS EN 1301-3, *Aluminium and aluminium alloys — Drawn wire — Part 3: Tolerances on dimensions.*

BS EN 1367-2, *Tests for thermal and weathering properties of aggregates — Part 2: Magnesium sulfate test.*

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BS EN 1744-1, *Tests for chemical properties of aggregates — Part 1: Chemical analysis.*

BS EN 10088 (all parts), *Stainless steels.*

BS EN 12163, *Copper and copper alloys — Rod for general purposes.*

BS EN 12164, *Copper and copper alloys — Rod for free machining purposes.*

BS EN 12165, *Copper and copper alloys — Wrought and unwrought forging stock.*

BS EN 12166, *Copper and copper alloys — Wire for general purposes.*

BS EN 12167, *Copper and copper alloys — Profiles and rectangular bar for general purposes.*

prEN 206, *Concrete — performance, production, placing and compliance criteria.*

DD ENV 1991-1, Eurocode 1: *Basis of design and actions on structures — Part 1: Basis of design (together with United Kingdom national application document).*

DD ENV 1992-1-1, Eurocode 2: *Design of concrete structures — Part 1-1: General rules for buildings.*

PD 6484, *Commentary on corrosion at bimetallic contacts and its alleviation.*

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

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

3.1.4

spring tides

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

3.1.5

neap tides

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

3.1.6

mean high water springs (MHWS)

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)

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)

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)

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

3.1.10

mean sea level (MSL)

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 that would exist in the absence of tides

3.1.11

lowest astronomical tide (LAT)

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)

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)

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)

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.

3.3 displacement

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.

3.4 belting

substantially horizontal continuous narrow rigid fender that projects from a vessel's side above the water line

3.5 Waves

3.5.1

wave height

height of a wave crest above the preceding wave trough

3.5.2

wave period

time for two successive wave crests to pass a fixed point

3.5.3

wave length

distance between consecutive wave crests

3.5.4

phase velocity

speed at which a wave propagates

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

3.5.5

wave diffraction

sharp change of direction and loss of energy of a wave as it passes a breakwater head due to diffraction when encountering an obstacle

3.5.6

wave refraction

slow change of direction and closer alignment with the seabed contours when a wave enters shallow water

3.5.7

wave gradient

wave height divided by the wave length

3.5.8

group velocity

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

3.5.9

significant wave height

average height of the highest one third of the waves

3.5.10

significant wave period

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

3.5.11

zero-crossing wave period

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

3.6

spectral density

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

3.7

design working life

assumed period for which a structure is to be used for its intended purpose with planned maintenance

3.8

return period

period that, on average, separates two occurrences

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_e	Breadth of earth-retaining structure.
B_v	Beam of vessel.
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
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.

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D_{50}	Median size of graded stone (other subscripts for different percentages passing).
d	Still water depth.
d_A	Depth of lowest trough below wave chart mean line.
d_B	Depth of second lowest trough below wave chart mean line.
d_b	Depth of water $5 \times H_s$ from wall (H_s calculated for depth "d").
d_c	Maximum displacement under cyclic loading.
d_e	Effective depth of sheet pile penetration.
d_m	Depth of water over mound.
d_m	Mean draught of vessel.
d_p	Sheet pile penetration.
d_t	Depth of tension crack.
d_x	Effective embedment of sheet piles.
d_0	Deep water depth.
d_1	Water depth at one wave length from wall.
d'	Depth below still water level of bottom of wall.
E	Effective kinetic energy of berthing vessel.
e	The exponent function.
f	Wave frequency.
F_B	Bed friction stress.
f_c	Frequency of cyclic loading.
F_D	Drag force.
F_H	Horizontal soil friction force.
F_I	Inertia force.
F_{LC}	Longitudinal current force.
F_{LW}	Longitudinal wind force.
f_m	Frequency at which peak occurs in spectrum.
f_N	Natural frequency of structure or member.
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).
g	Acceleration due to gravity.
H_A	Height of highest crest above wave chart mean line.
H_a	Wave height at arrival point.
H_B	Height of second highest crest above wave chart mean line.
H_b	Breaking wave at structure.
h_c	Height of top of wall above still water level.
H_D	Design wave height.
H_{inc}	Incident wave height.
H_L	Limiting wave height in probability distributions.
H_{max}	Maximum wave height.
H_n	The nth height or height interval in a rank-ordered set of n_x values.
H_R	Retained height of structure.
H_r	Height of retained soil.
$H_{r,\theta}$	Wave height at position (r,θ).
H_s	Significant wave height.
H_{sd}	Significant height of set-down.
$H_{s,n}$	The nth significant wave height in a set.
H_{so}	Significant wave height offshore
$H_{s,1}$	Component significant wave height from wave chart (similarly $H_{s,2}$).
H_w	Height of wave run-up above still water level.
H_z	Height of wind measurement above sea surface.
i	Number designating a stress range.
$I(\mathrm{f},\varphi)$	Spectral density function.
$I_D(\theta)$	Wave diffraction intensity factor.
$I_o(\mathrm{f},\varphi)$	Offshore spectral density function.
K	Stiffness of equivalent spring.
K	A constant.
k	Permeability.
K_A	Coefficient of active earth pressure.
K_b	Bed friction factor.
K_d	Wave diffraction coefficient.
K_f	Wave height reduction factor due to bed friction.
k_J	Coefficient in JONSWAP spectral density function.
k_P	Coefficient in Pierson–Moskowitz spectral density function.
K_p	Coefficient of passive earth resistance.
K_r	Wave refraction coefficient.
K_s	Wave shoaling coefficient.
K_0	Coefficient of at-rest earth pressure.

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

L Length of vessel.

L Wave length.

l Length of cylinder.

L_{BP} Length of vessel between perpendiculars.

l_e Effective length of anchorage.

L_F Fetch length.

L_s Submerged length of member.

L_W Waterline length of vessel.

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

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

L_0 Wave length in deep water.

l_0 Ineffective length of anchorage.

m_c Machine payload capacity.

m_D Displacement of vessel.

m_e Equivalent mass of structure.

m_L Mass per unit length of member.

m_s Mass of cellular structure and contained soil.

m_{50} Median mass of graded stone.

N Number of waves in design condition duration.

n A number between 1 and n_x

N_i Number of waves in i^{th} stress range during design life.

n_i Number of waves in the i^{th} stress range during design life.

N_s Scale factor.

n_T Total number of stress ranges.

n_x A number.

N_z Number of zero up-crossings on wave record.

P Maximum applied cyclic load.

P Probability.

P_A Active soil force per unit length of wall.

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

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

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

P_n Probability of nth value being equalled or exceeded.

P_u Pore water pressure.

P_u Effective uplift wave pressure at foot of wall.

P_{w1} Average maximum wave pressure on wall at still water level.

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

q Proportion of critical damping.

R Resistance per unit area to shear along soil slip surface.

r Radius.

r Polar co-ordinate.

Re Reynolds number.

s Bed slope (tangent of angle relative to horizontal).

$S(f)$ One-dimensional spectral density function.

T Wave period.

t Variable used in normal distribution.

t Time variable.

T_m Mean wave period

T_o Period over which observations are taken.

T_p Period at which peak occurs in wave spectrum.

T_{po} Peak period offshore

T_R Return period.

t_{90} Time required for 90 % of original bacteria in a sample to die.

U Instantaneous water particle velocity normal to member.

$\dot{U}$ Instantaneous water particle acceleration normal to member.

u Horizontal components of water particle acceleration.

U_w Wind speed 10 m above sea surface.

U_z Wind speed at z m above sea surface (other subscripts for different heights).

V Incident current velocity.

v Kinematic viscosity of water.

$\dot{v}$ Vertical component of water particle velocity.

V_B Velocity of vessel normal to berthing face.

V_c Design current velocity.

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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_{crit}	Critical current velocity.
v_{c0}	Velocity of wave propagation in deep water.
v_s	Average fall velocity of sediment in still water.
V_W	Design wind speed.
V'_C	Average current velocity over mean depth of vessel.
$V\eta$	Vertical velocity of water surface.
v^*	Friction velocity.
w_F	Fetch width.
w_p	Water particle orbit width at surface.
w_s	Width or diameter of submerged structure or member.
x	Rectangular co-ordinate.
X_i	Component of radial in direction parallel to mean wind.
x_w	Wave prediction parameter.
y	Variable used in normal distribution.
Y_a	Dimension of submerged member.
Y_b	Dimension of submerged member.
Y_c	Dimension of submerged member.
Z	Tidal lag.
z	Pile wall thickness.
α	Angle of current off bow.
α_c	Angle of current relative to member axis.
α_o	Angle of wind off bow of vessel.
α_r	Angle of wind radial relative to mean wind direction.
α_s	Angle of slope from horizontal.
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_1^*, 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 groundwater.
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 co-ordinate.
θ_o	Angle of incident wave relative to breakwater axis.
λ	Term used in wave chart analysis (= $\log_e N_z$)
λ	Coefficient of local wave pressure.
μ	Coefficient of friction.
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_m	Mean wave direction.
φ_o	Offshore wave direction.
φ_r	Angle of soil shearing resistance.
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.

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Section 2. Environmental considerations

5 General

5.1 Design parameters

A fundamental prerequisite to designing a maritime structure is the understanding and assessment of the naturally occurring phenomena to which that structure is exposed. Information concerning these phenomena might already be available from existing sources, although such data can often be limited in scope and application, and further detailed investigations might be required to permit the selection of design parameters.

5.2 Environmental impact

The construction, operation, maintenance and decommissioning of maritime structures can cause a substantial impact on both the marine and terrestrial environment. Although the primary interface with the environment is through change in water regime in the vicinity of the structure, there are many other mechanisms by which environmental harm can result. In order to determine the potential effects of a structure an environmental assessment should be carried out in order to review all such mechanisms, whether they cause primary or secondary, direct or indirect, long term or short-term effects. The maritime structure can then be planned and designed so as to incorporate measures to minimize any undesirable environmental impact.

Consideration should be given to environmental conditions at all stages of construction, as well as for the completed structure. During construction, maritime works are particularly sensitive to adverse weather conditions, which can hinder access to the works, prevent the use of floating plant and cause damage to work both above and below high water level. Weather conditions can limit construction activity to certain seasons or “windows” and affect various transient load conditions such as towing, sinking and grounding of floating elements.

In evaluating environmental impacts specific expertise should be obtained from competent bodies. Detailed guidance on the types of impacts to be considered is available in a number of publications [1] [2]. World Bank Technical Paper No 126 [3] provides a checklist on the range of topics that should be considered.

5.3 Scope

This section describes the various environmental phenomena that 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 might suffice for the purposes of the initial investigations, it might at a later date be necessary to control other work connected with the structure, and at that stage a far more rigorous approach will be required.

6.2 Level control

Most investigations connected with surveys for maritime structures 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;

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 that are subject to settlement.

Types of gauges are described in **10.4.2**.

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 four main categories, namely:

— visual;

— satellite;

— radio-positioning;

— laser.

Range capabilities extend from a few hundred metres to several hundreds of kilometres.

6.3.2 *Visual methods*

The overall accuracy of visual positioning methods normally 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 co-ordinated 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 that inexperienced observers can produce erroneous results.

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, a higher order of positioning accuracy is not necessarily achieved. Lack of synchronism between the land-based and sea-based elements of the survey work is the main source of error. Use of theodolites also reduces the flexibility of a survey operation, because it might be necessary to have frequent changes in the position of the theodolites in order to maintain well-conditioned intersections.

6.3.3 ***Satellite methods***

Global positioning systems have largely superseded visual and radio-positioning methods for the location and surveillance of traffic. These systems are based on the use of radio signals from earth satellites, which are under the control of military authorities. These authorities have the ability to deliberately degrade the radio signals so as to reduce location accuracy.

The deliberate degradation of system performance can be removed, however, by "differential" techniques. Differential corrections are determined by comparison of observed signal measurement at a known location with calculated data; the difference between the calculated and the observed is the correction. The corrections are then broadcast to a survey boat in the vicinity. Differential techniques also help to remove errors caused by other factors such as atmospheric conditions. The accuracy of the corrections diminishes with increasing range from the reference station.

One problem with satellite systems is that in some territories the relationship between the satellite and local co-ordinate systems has not been defined.

Even when defined, the accuracy achieved depends on the proficiency of the operator.

6.3.4 ***Radio-positioning methods***

Most radio-positioning methods operate in a range-range mode, measuring the ranges from shore reference stations to a master station on the survey vessel. The position of the vessel is given by the simultaneous measurement of two or more ranges, the position being that of the intersection of the range arcs.

The systems in general use for coastal work can be used for up to 30 km range with a positioning accuracy of from 1 m to 3 m.

The range-bearing type is used for relatively short range survey work, where the position of the vessel is given by simultaneous measurement of range and bearing from a single shore station. Accuracy is about 1 m at a range of 8 km to 10 km.

Radio-positioning methods are described in greater detail in BS 6349-5:1991.

6.3.5 ***Laser systems***

Certain types of laser device 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 tracking systems are useful in a variety of survey situations. A system set up on shore can determine range and angle to a vessel up to 10 km from shore.

Laser instruments have also been developed that can provide a single range and angle 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.

7 Meteorology and climatology

7.1 General

Clause **7** describes the meteorological and climatological considerations that 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 normally be obtained from the meteorological office covering the area under consideration. The Meteorological Office, Bracknell, Berkshire, England can also provide data for most overseas locations. Another valuable source of data for overseas locations is the Admiralty Pilot series of publications [4].

Of particular interest to the construction industry is the quick reply climatological service developed by the Meteorological Office, Bracknell, to assist tendering and planning in the UK especially in estimating the time likely to be lost because of adverse weather. The service 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 usually adequate for detailed planning and a further site-specific data search should be undertaken.

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7.2 Wind

7.2.1 *General*

Wind 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.

During coastal construction, long-term wind records are vital, for predicting weather windows and likely delays during site investigation, construction and operation. The effect of the wind on water set-up and storm surge generation is of importance.

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*

Wind speed and direction are usually measured by means of some form of anemometer and wind vane. The output can be either recorded or read at regular intervals.

7.2.3 *Exposure and level of measurements*

Care should be taken when siting an anemometer, because eddies created by obstructions such as trees and buildings can produce unreliable measurements of the wind speed and direction and therefore 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. This is defined as the height over open level terrain in the vicinity that, 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. This will vary with different averaging time intervals and site conditions, so expert advice should be sought before measured values are corrected.

7.2.4 *Presentation and use of wind data*

Standard wind records can be used as follows.

a) As a basis for wave prediction (see section **4**) and the design of moorings (see section **5**).

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 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, reference can be made to the sources of data given in **2.3.1**.

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 exerts load on a structure. The amount and expected duration of snowfalls should be considered.

c) *Cargo handling.* The intensity and amount of rainfall is important with reference to the type of cargo handled, handling rates and the storage facilities in a port.

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

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

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 that are likely to be encountered during the life of the structure. To provide suitable design data four temperatures should be measured at regular intervals, i.e. minimum and maximum, and wet and dry bulb.

7.5 Visibility

7.5.1 *General*

Poor visibility can have severe consequences in relation to navigation in inshore waters and estimates of the expected duration should be made. The reduction of atmospheric transparency and therefore 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.

Visibility 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 is usually not be experienced more than 3 km to 4 km inland and erroneous data can therefore be extracted for navigation and piloting purposes.

7.6 Atmospheric pressure

7.6.1 *General*

The pressure of the atmosphere at any point is the weight of the air that lies vertically above a unit area. The atmospheric pressure is recorded in millibars[1)] and is usually 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, therefore, sea state forecasts. In particular, the influence of pressure on water level is of importance. A high or low pressure decreases or increases, respectively, the depth of water to cause what is known as a storm surge, either negative or positive (see clause **10**).

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 that can be directly related to the energy received at the sensor. Radiometers can be designed to be sensitive in particular wave lengths of interest, for example incoming short wave lengths or outgoing long wave lengths or in the specific bands of ultraviolet or infrared.

The duration of bright sunshine is normally 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 ultraviolet wave lengths. 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

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**.

More detailed guidance on bathymetric surveys in relation to dredging and land reclamation work is given in BS 6349-5:1991.

Automatic systems, which receive inputs for location and water depth, should be regularly calibrated as part of a quality control system. Care should be taken to ensure that raw data is stored and that any averaging of data is defined.

Published bathymetric charts provide the designer with very limited data for his purposes. Charts are produced for safe navigation and features that can be of particular significance to the engineer might not necessarily be shown. Charts are only as accurate as the data from which they were compiled.

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1) 1 mbar = 100 N/m^2 = 100 Pa
1 000 mbar ≈ 29.5 in Hg.

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Data is often 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.

8.2 Echo sounder

8.2.1 *General*

The prime method of measuring water depth is by echo sounder, which records a continuous profile of the seabed as the survey craft moves over it. Only purpose-built hydrographic echo sounders should be used if accuracies that are acceptable for engineering purposes are to be obtained. A typical hydrographic echo sounder produces a large-scale paper trace providing a permanent graphic analogue record of the seabed profile. The instrument is 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.

Alternatively, electronic methods can be used to determine the velocity of sound in water.

Whichever technique is used, the importance of regular calibration cannot be over-stressed. Further guidance is given in BS 6349-5:1991.

8.2.3 *Transmission frequency and beam width*

Considerations affecting the choice of transmission frequency and beam width are covered in BS 6349-5:1991.

Some echo sounders operate on dual frequencies, typically 33 kHz and 210 kHz in order to identify areas of soft sediments. Acoustic energy at 33 kHz passes through fluid mud and is reflected off the underlying strata; energy at 210 kHz is reflected from the top of a layer of fluid mud. It is thus possible to assess the thickness as well as the depth to fluid mud or other very soft sediments. (See **18.3** regarding effects of seabed density.)

8.2.4 *Swathe bathymetry*

Swathe bathymetry uses the principles of side-scan sonar, which is described as follows, but mathematically adjusts the soundings to create a band of soundings across the direction in which the boat is travelling. The system gathers more data than a conventional echo sounder and gives coverage under adjacent vessels. The equipment is however expensive and is only economic to use when large amounts of data are required.

8.3 Side-scan sonar

Working on similar principles as an echo sounder, side-scan sonar systems transmit a fan-shaped 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 can also be detected.

More specific information on side-scan sonar systems is given in BS 6349-5:1991.

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 normally 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.

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 depends 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 normally 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 can 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 1:2 000.

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 times to 10 times. These lines serve the purpose of check lines and reveal the presence of features such as sand waves, which, because of the original choice of direction, might not have been identified.

8.6 Wire sweep

A wire sweep survey might 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 fore and aft from the vessel a horizontal bar, 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 that an area is 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 (see also **10**).

The datum to which the soundings are reduced should always be noted on the drawing or chart, together with its relationship with the relevant land datum.

The bathymetric plan should preferably be plotted on the same grid system as any adjacent land survey.

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.

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.

The bathymetric plan should preferably be plotted on the same grid system as any adjacent land survey.

9 Geological considerations

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.

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 can be caused by such factors as wind, atmospheric pressure, wave effects, local run-off and evaporation. A further factor that should be taken into account is the long-term rise in general sea level.

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 might also be required in relation to soil pore water pressures, flood relief valve discharge capacities and for the prediction of tidal flows.

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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 [5]. These predictions are of universal application and are usually based on continuous observations of the tide over a period of at least one 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.

Harbour authorities and others also publish almanacs and tide tables. 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 Proudman Oceanographic Laboratory (formerly the Institute of Oceanographic Sciences), Bidston, Birkenhead, England when information is required in a more detailed or specific format than is provided by the Admiralty tide tables.

10.3 Meteorological effects

10.3.1 *General*

Tide predictions are based on average meteorological conditions. Variations in predicted heights 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 tends 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. This results 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 can cause oscillations in sea level. The period between such waves can 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 can be raised by river flow originating from surface water run-off or artesian sources. Special factors, such as the 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 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*

The basic type of tide gauge is the tide pole or staff. This is a simple graduated pole set up vertically on a fixed structure in the water so that the changing level of the sea surface can be read by an observer. It is important to install a tide pole, wherever possible, regardless of any other automatic gauge that might be installed, because it provides a calibration and checking standard for automatic gauges.

There are many types of automatic gauge that provide a continuous record, such as:

a) float operated;

b) pressure;

c) bubbler;

d) acoustic;

e) electric resistance (step gauge).

10.5 Tidal analysis

If a sufficiently large database is obtained, an analysis of the records can produce the astronomic tidal harmonic constituents, which can then be used to predict astronomic tidal heights. These predictions assume that no other changes occur at the relevant site and their accuracy depends on the size and accuracy of the original database. It is possible to obtain the basic harmonic constituents for a site from only 14 days tidal data but this requires favourable circumstances. Continuous records covering 28 days to 30 days give an adequate analysis and subsequent predictions. A full tidal analysis from a minimum of one year’s continuous data would produce more than 114 tidal constituents, which is a sufficient number to predict accurate astronomical tidal heights.

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Such analyses and predictions are explained in detail elsewhere [6]. 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 [4].

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 is 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.

11.2 Measurement of currents

11.2.1 *General*

Indirect methods of calculating water movement are normally 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 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 might be beneath the threshold of the instrument and transmitted vessel or moorings movement can 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 normally 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 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.

11.2.2.3 *Recording current meters*

Recording current meters are typically between 2 times 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, therefore 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 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 that 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. These meters normally measure the current velocity at frequent intervals, for example once or ten times per 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.

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11.2.3 ***Float tracking***

This method of tracing water movement involves the introduction of readily identifiable material that 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 therefore 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 lies and follows 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 is 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 is 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 floats vertically just under the water surface. Typical materials used are telegraph poles, tree trunks, bamboo poles or lengths of aluminium. Logships are particular useful in navigation studies and harbour layout planning because 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 therefore the 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 that 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 are not always entirely reliable, due to differences between the paths of the identified water layers.

As stated, the floats do not follow the same turbulent flow paths as the water itself and so 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 beachwalkers 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 many 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 [7]. An indicator was sought with physical properties similar to those of the water to be labelled.

Acceptable techniques that were developed were the use of dyes, bacterial organisms and radioactive materials. The last technique, which involves the use of radioactive materials, is no longer feasible because of public health restrictions on the strength of isotopes that could be released.

These techniques all have specific advantages and limitations and a problem can often best be solved by a combination of 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.

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 that 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 dispersion rates and mass movement for a proposed discharge at sea before construction. The timing of the release can be organized as considered previously, 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 might be of special interest.

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.

Batch releases of dye 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 zigzag 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 are wider than those estimated visually, because the sensitivity of the measuring instruments is greater than that of the eye.

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 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 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.

Considerable progress has been made in recent years in the development of computer models for studying water movement and in particular effluent mixing and dispersion. These models rely mainly on prototype information on currents and density structure and to a lesser extent on the dispersion characteristics of the location. Specific field studies are recommended in order to ensure that adequate data is available for model verification and calibration.

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 fieldwork, to avoid spending time and money on data that 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.

Raw data should be archived so that others can make use of it.

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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 only shows 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 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.

Although charts and pictorial presentations of data are still necessary, increasing use is made of computer graphics for data presentation.

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 viewpoints. 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. The influence of the structure on the water quality and other features of the surrounding environment, i.e. the environmental impact, is covered in section **2**. The environmental effects, which are reviewed in the following paragraphs, are with reference to the effect of water quality on a maritime structure.

13.2 Water temperature

13.2.1 *General*

In coastal regions there is usually a well-defined seasonal temperature variation, although throughout the year the water column tends to be isothermal due to strong turbulent mixing. Significant stratification can 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 can become a problem when the water temperatures fall below −2 °C. Floating ice masses that are formed elsewhere can survive though 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 sea surface temperatures fall below 6 °C.

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

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

d) *Effluent dispersion.* The density of seawater 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.

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 normally fully saturated with oxygen at the surface but if there is little vertical mixing the oxygen content decreases with depth. Under normal circumstances this decrease is unlikely to have a significant effect unless anaerobic conditions are reached but the local distribution and seasonal variation should be considered when siting outfalls to discharge effluents that could act as reducing agents.

The important chemical parameters are usually analysed directly or measured with selective ion electrodes, either in the field or in the laboratory.

13.4 Turbidity

Turbidity is usually caused by suspended clay or silt particles, dispersed organics and micro-organisms. A lower water temperature increases 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 usually occur during dredging operations and the consequences should be borne in mind during the planning of the initial site preparation and maintenance dredging. Guidance is given in BS 6349-5:1991.

Dredging operations can also cause the release of harmful substances that are locked into fine sediment particles and can remain attached when dredging operations put the material into suspension. The possibility of degradation of the structure caused by redeposited harmful substances should be considered. Detailed consideration should also be given to the area of spoil disposal, because of the possibility of dispersion with possible redeposition back in the dredged area.

13.5 Marine life

Many forms of marine organisms including algae, molluscs, bacteria, crustacea and others attach themselves to a maritime structure. The organisms can cause blockage of intake and discharge pipes, impose or increase mechanical stresses (see **46.2.2**), accelerate degradation (see **59.2.2**), retard flow or simply impede inspections for maintenance or certification purposes [8]. Methods of controlling marine growth include the use of anti-fouling paints, scraping with the hand or mechanical removal by water- or air-jetting. As well as surface-attaching species, structures with timber elements can 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 **66.1**e).

13.6 Pollution

The effects of water-borne pollution on the structure should be considered. Some trade effluents, if insufficiently diluted, can accelerate the deterioration of concrete and steel. The effect of oil spillages is usually benign with respect to structural condition, but the surface coating makes inspection difficult. Pollution can act as nutrients or deterrents to bacteria, significantly affecting microbial induced corrosion.

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 has to adjust in order to achieve a new dynamic equilibrium. Such is the delicate balance of nature that what can 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. Waves and currents are the agents responsible for entraining, transporting and depositing sediment. Of these, waves are the most powerful and important agent. Bathymetry imposes constraints on currents and waves and therefore on erosion and deposition. The size distribution of the indigenous sediment normally 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 cases of beaches, by carrying out a preliminary inspection. A comparison between historic information and modern details can then be made, and changes of depth and apparent movement of features such as offshore sandbars 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 are indicative of the direction of the drift and intensity of sediment transport.

Care should be taken to avoid being misled by seasonal or short-term effects.

14.2 Sediment transport in currents

14.2.1 *General*

When the current flows over a bed of erodible sediment, a certain minimum 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.

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When the current exceeds the threshold value, the bed begins to erode, the displaced particles being carried over the bottom by sliding, rolling of 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 normally considered that suspension becomes appreciable when $v_s/v_o \approx 1$ where v_s is the average fall velocity of the particles in still water and the friction velocity $v_o = \sqrt{(\tau_0/\rho)}$ where τ_0 is the shear stress exerted by the water on the bed and ρ is the water density.

Beds subject to erosion usually form small-scale ripples and larger-scale dunes. These dunes are usually short-crested and migrate slowly in the direction of the current. Although they display a certain average wave length and height, they vary in size. They have flat upstream slopes and steep downstream slopes.

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.

Devices for measuring suspended load are described in **14.2.2.2** to **14.2.2.4**. For the sampler described in **14.2.2.2** and **14.2.2.3**, the grain size distribution is determined by standard sieving, settling column, Coulter counter or laser diffraction 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 *Silt meter*

The meter incorporates a light source (usually infrared) and a photo detector. 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. Because the bed features usually make it impractical 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.

14.2.2.5 *Acoustic devices*

Considerable progress has been made in recent years in the development of acoustic Doppler devices for measuring sediment concentration profiles. The use of multi-frequency systems, together with powerful computer processing, is leading to the capability of determining in-situ concentration and particle size characteristics of the complete profile.

14.2.3 *Measuring bed transport*

For fine sand and silt, bed transport is usually small in comparison with suspended transport, but can become significant with increasing grain size. There are essentially two methods for studying the movement of bed load: those that use laboratory-derived bed load formulae (see **14.2.4**) and those that rely on field measurements (see **14.2.5**). Direction and rate of bed transport can be estimated by employing fluorescent 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 have 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 [9].

14.2.5 *Field tracer studies*

14.2.5.1 *General*

Much research has been carried out in the past in order to develop various methods of tracing sediment movement in the field. Techniques used were to employ coloured, radioactive or natural heavy mineral tracers. As in the case of water movement tracing (see **11.3.1**) radioactive materials are no longer permitted.

Field tracer studies for determination of bed movement are now rare. Coloured tracers are used for very small-scale studies, usually on beaches.

Artificial pebbles, which are radio-tagged, are also used in small-scale studies.

Problems of depositing the labelled material arise because 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.

Unless weather and wave conditions are similar to those of the long-term average, non-typical results can be produced. The direction of sediment movement is usually much easier to determine than its magnitude, which can be considerably in error, depending on circumstances. It should be noted that the techniques described could also be employed to study suspended sediment transport of non-cohesive grains.

14.2.5.2 *Coloured tracers*

Coloured tracers involve the coating of non-cohesive sediments with either a distinguishing coloured paint or a fluorescent substance. Indigenous sediment can 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 can be substituted, provided that it responds to the hydraulic processes in exactly the same way as the indigenous sediment.

The sampling routine depends on the environment being studied. 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 ultraviolet 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 *Radio tagging*

Radio tagging is carried out by embedding a radio microtransmitter in an artificial pebble, which is moulded using a reinforced epoxy resin. A radio detector mounted on a mobile tracked vehicle is used to locate and map the individual pebbles.

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 that 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 timescale of geological processes.

14.3 Sediment transport with waves

14.3.1 ***Movement due to waves***

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 [10, 11].

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

14.3.2 ***Littoral drift***

In the longshore direction, the tangential momentum of waves breaking at an angle to the shoreline can induce currents. 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 sediment to the breaker height and specialist texts should be consulted [12]. 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. It requires careful study whenever construction that might disturb the dynamic equilibrium of the local drift regime is contemplated (see **14.1** and **14.4** to **14.6**).

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14.3.3 ***Rip currents***

There often 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. These are caused by the interaction of wave set up and edge waves (see **25.3**). 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 these techniques give an estimate of the rate and direction of sediment transport in an area, in many engineering problems it is necessary to produce an estimate of the amount of accretion or scour. It should be noted that predictions of this type are only approximations.

14.4.2 ***Trial trench method***

The most obvious method of estimating accretion or scour rates is by trial dredge.

14.5 Bed-form migration

The direction of sediment transport and a qualitative appreciation of the rate of transport can be gained by observing the migration of current-produced bed forms. Bed-form migration appears to be controlled by the time–velocity asymmetry of bottom tide-generated currents and the sediment grain size.

The method of study depends 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 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 might 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 moveable bed 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 first 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 because, 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. The second disadvantage is the inability (because of cost) to run many variations in layout.

On the other hand a mathematical model might be an oversimplification or even misinterpretation of reality. Knowledge of the mechanisms of sediment transport and therefore 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 that 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 take place and therefore 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 may be found in subsequent parts of this code of practice. However, in all projects, there should be consultation between the port operator, ship operators and ship handlers (the pilots and tugmasters). Consultation should be carried out at the concept stage before the layout of a port or a berth has been fixed. At the concept stage the use of simulation studies (see **18.2**) is strongly advised in order to ensure 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;

e) mooring patterns, practices, systems and load measurements;

f) berth occupancy, ship queuing times, port downtime and the effects of sea and weather conditions;

g) 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;

h) regulations affecting any of the previous operations a) to g).

In considering the safety aspects of a facility the initial construction techniques and the requirements for future maintenance should also be included. The Construction (Design and Management) Regulations, 1994 require designers to consider the health and safety implications of any design decision in order to minimize risks during construction and maintenance activities. Designers have specific duties to consider risk minimization and inform clients of the requirements of the Regulations. Designers should refer to the Regulations prior to commencing the design process.

Because safety is a prime concern in the operation of facilities, liaison during the early design stages is vital to ensure adequate transfer of information and appreciation of operational procedures.

16 Design working life

The design working life of a structure can be taken as the specified period for which a structure is to be used for its intended purpose with planned maintenance (see ENV 1991-1).

It should be noted that this requires knowledge of ambient conditions, loadings and rates of deterioration of the elements of the structure throughout its life.

Normally a design working life of the order of 50 years or more is expected of maritime structures such as quay walls, jetties and docks but for flood protection works it is not uncommon for a 100-year life to be required.

For structures associated with industrial installations or the exploitation for a non-renewable natural resource the design working life might only need to be 15 years (see ENV 1991-1).

It should be noted that the design life is not necessarily the same as the return period of the design condition, because such an equality implies a 63 % probability of the design condition occurring during the design working life (see **21.4**).

The design working 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. Greater overall economy is usually 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 that should be considered include type, size and shape, ship handling requirements, cargo or passenger handling requirements and vessel servicing requirements.

Characteristic dimensions and hull forms of many ships vary considerably according to function, age and operational region. This is particularly noticeable for passenger ships and naval craft.

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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 can be obtained from the registration particulars [13].

Ship handling considerations are discussed in clauses **18** and **19**.

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 (GRT) and deadweight tonnage (DWT). 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 can be used to obtain full load displacement from GRT or DWT. 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 previous 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, Figures 1 and 2 can 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 viewpoint ideal accessibility to a harbour implies:

a) a straight, wide approach channel, the direction of which coincides with the direction of currents, winds 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, for the two following reasons. Firstly, the dominant currents rarely coincide with the direction of the highest seas and, secondly, aligning the channel with the highest seas tends to maximize the wave penetration into the harbour. Ports located in estuaries, where the hydraulic conditions are determined mainly by the tides, normally offer better protection for seagoing navigation and many of the larger sea ports are situated in such locations. Access problems can still arise, however, because vessels can 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 can be incurred to remove siltation both in the artificially 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 larger vessels nearer to deep water at sea.

18.2 Studies

Where access channels are to be aligned or alternative routes found, the opinions of experienced masters and pilots are 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 can be controlled by one or two operators, one of whom might 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 timescale are not entirely overcome, however, although the bigger the model, the smaller the problem.

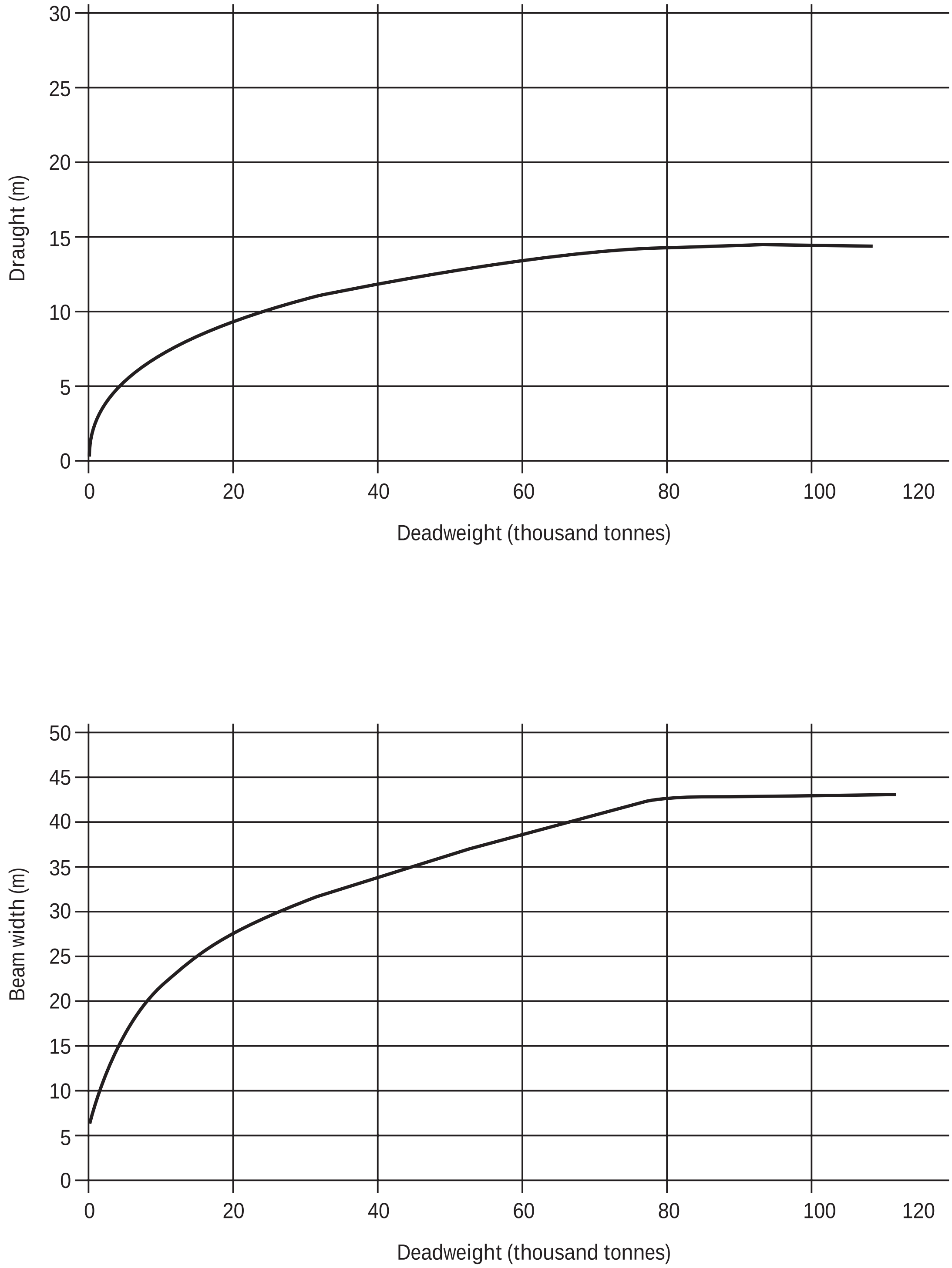


Figure 1a) — Typical ship dimensions — Container ship — Approximate dimensions based on draught against DWT, and beam width against DWT

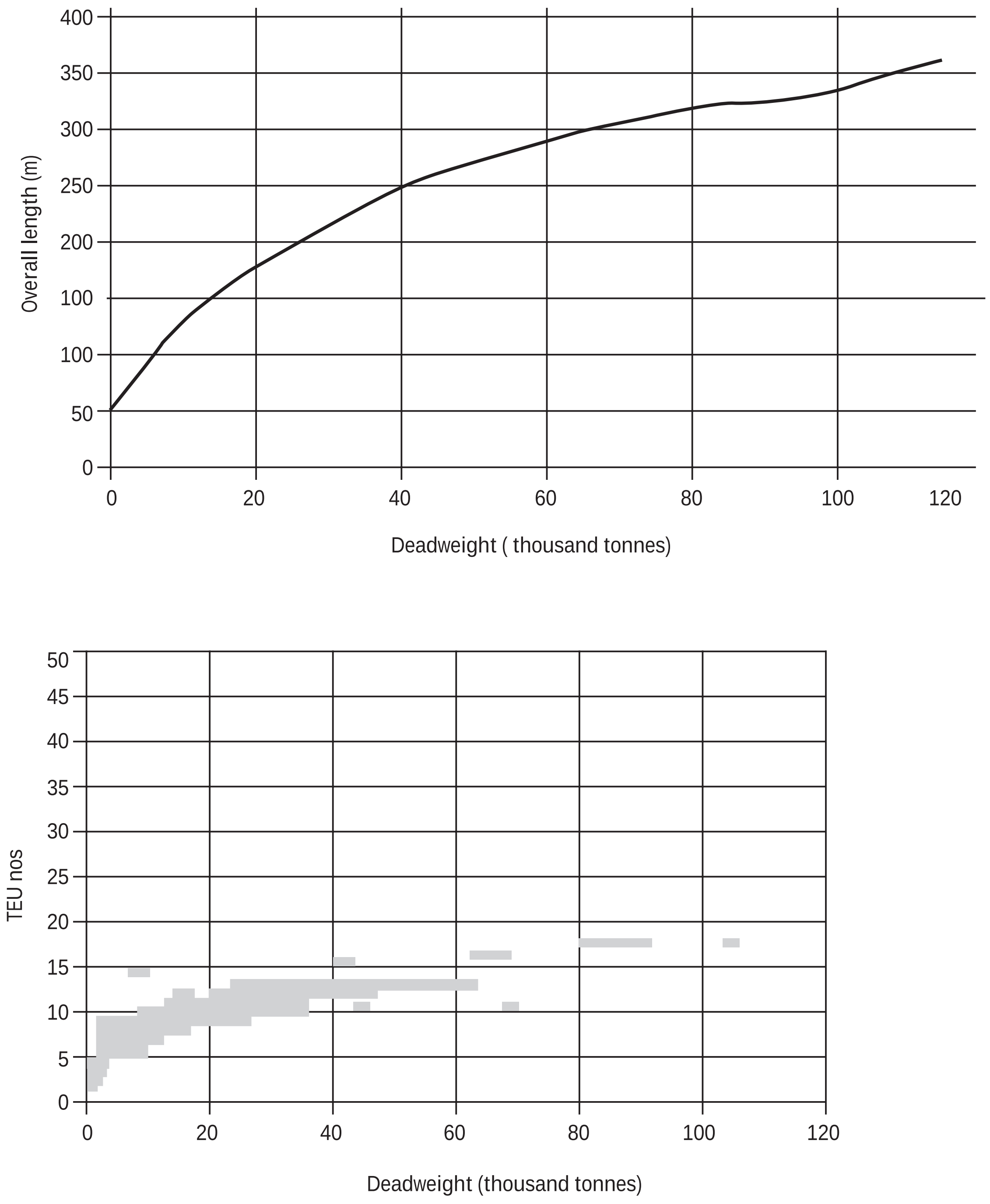
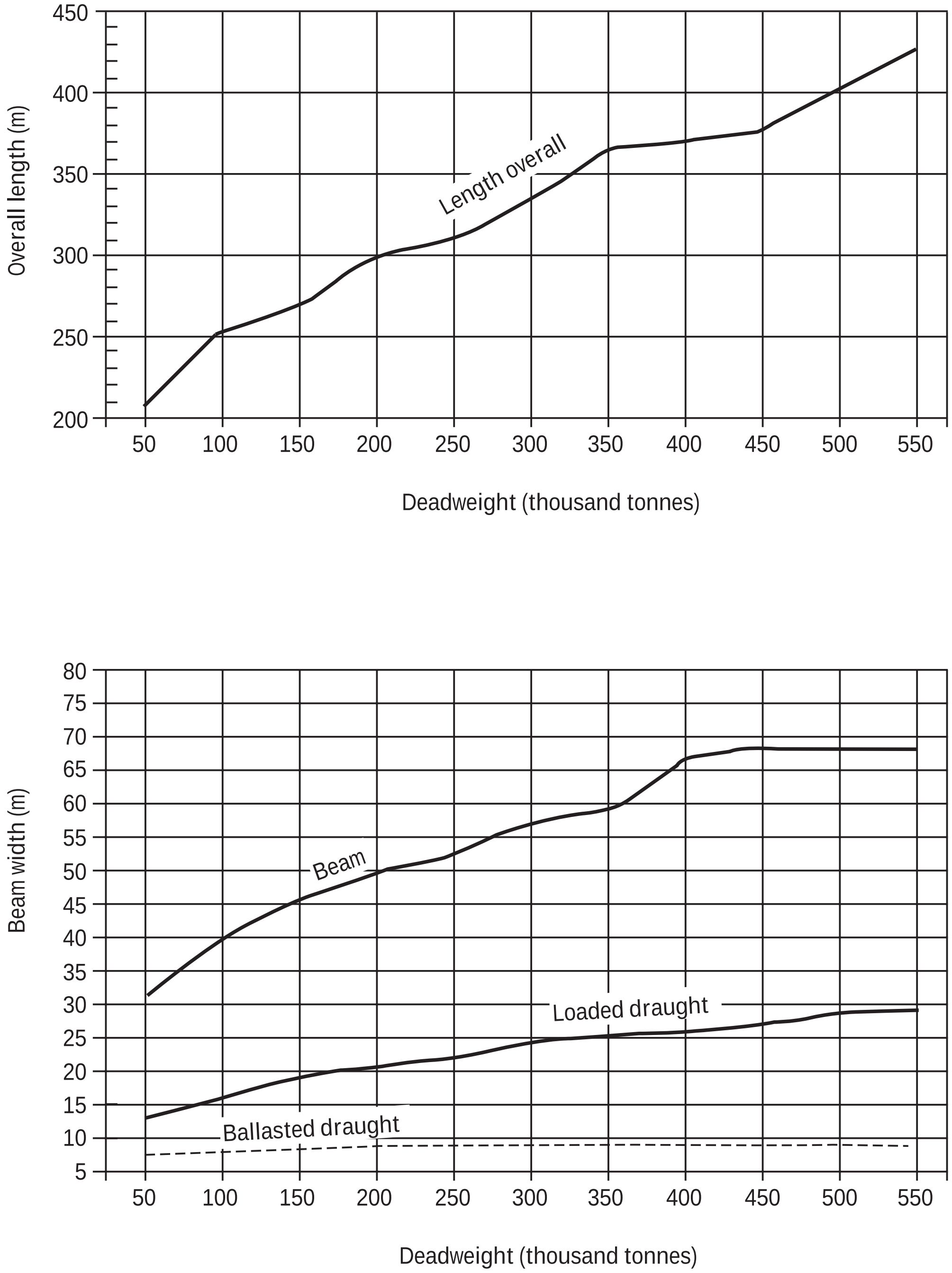


Figure 1b) — Typical ship dimensions — Container ship — Approximate dimensions based on draught against DWT, and TEU against DWT



NOTE TEU = Twenty Tonnes Equivalent Units

Figure 2 — Typical ship dimensions — Bulk carrier

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Computer simulation is now widely used to reproduce ship handling characteristics and the hydrodynamic effects of seabed proximity, currents, wind and wave loadings and towage forces. These simulations have the capability of operating in real time or fast time modes with the ability for the simulated ships to be operator driven or, in the case of channel design studies, can be run using an automated steering to represent the actions of the pilot/helmsman. This particular facility has value when numerous channel transits are to be made for the purposes of a statistical analysis of grounding risk. Typically, these simulator programs operate with a bird's eye view, which is the more usual mode for assessing ship manoeuvring requirements in restricted areas, although a ship's bridge view can be used for assessing the optimum location for buoys or other aids to navigation.

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. This is done 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 timescale problems mentioned previously. 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.

Reduced water depth causes the ship under way to respond more slowly to changes in engine revolutions and helm than when the ship is in deep water. Stopping distances and bend design would need to make allowances for these effects, which become noticeable when the water depth is less than about 1.5 times the ship draught.

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, might 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. 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.

For preliminary design purposes it is usual to allow 10 % of the ship's draught as underkeel clearance when the ship is at harbour manoeuvring speeds. This takes account of squat, and draught/sounding uncertainties and gives a margin for safety.

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 channels [14].

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 caused by sailing in the proximity of a channel bank [14].

The vertical movements under waves and swell depend on the wave height and direction and the ratio of the wave length 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 [15].

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**).

For preliminary design purposes the 10 % of draft, commonly used as an allowance for squat, is increased to 30 % in order to allow for squat plus ship motions due to waves [14]. For more accurate estimations, guidance should be obtained from a specialist. Computer programmes are available [15] to assist the designer.

Where the bottom of the channel consists of mud or silt, it is usual to define a nautical depth as being 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. This is because research has shown that mud layers of a lower density do not significantly impede the passage of a ship. It should be noted that, depending on its operating frequency, an echo sounder identifies the seabed as the level of muds of a significantly lower density, whereas a sounding lead sinks until supported by muds of a density greater 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 normally have relatively long time constants and can 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**). The changing conditions can be reproduced in real time and the response of the vessel and pilot to these observed, as well as assessing the burden that is placed on the skill of the navigator.

As a general guideline, it is suggested that the ratio of channel width to the beam of the vessel should be between 4 and 6, for one-way movements, depending on local conditions.

For somewhat smaller vessels passing one another, the minimum width of channel appears to be about six to eight times the beam of the larger 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 [14]. It is strongly recommended that two-way traffic of large tankers should be avoided whenever possible.

It can be argued that it is more appropriate to use vessel length to specify the minimum channel width because 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. Because the length to beam ratio of large vessels usually lies between 5.5 and 6.5, the channel width to beam ratio given previously can be interpreted in terms of length.

18.5 Other operational aspects

In addition to the hydrodynamic 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 affect the speed at which a ship passes 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 is 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 pilots, tugs and navigational aids.

At the immediate approach to the harbour entrance, sea conditions can sometimes be rougher than elsewhere, due to the waves becoming steeper on an ebbing tide, offshore wind or reflection of waves from the harbour walls or breakwaters when these are steep sided. Accordingly, a minimum velocity is 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 propeller 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 can be halved where tugs are used regularly and can 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.

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19.2 Berthing

Full discussion of operational practices at a planned berth should whenever possible be held with pilots, ships' masters and port operators, 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. This helps the pilot to keep within the maximum allowable speed and to achieve the optimum angle of approach for which the fenders and jetty layout have been designed.

The employment of tugs is compulsory in some ports, although the choice of how many is the decision of the ship's master, who is 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 can 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 sometimes have equipment to monitor the velocity of approach and a display to inform those on board the ship of the transverse approach velocity. Data from the equipment can be used to give guidance on the possibility of accepting larger ships than the design ship at a suitably reduced approach velocity.

Ships that 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.

Predictions of berthing behaviour can 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 can be made to determine the magnitude of the lateral force due to current and wind and to determine the best design of structures and arrangement of fenders to withstand possible impact 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 BS 6349-4:1994.

19.3 Mooring

When the ship has arrived at the berth and has been secured it is still subject to wind and current forces, which cause movement of waves in the harbour (seiches) (see **29.4**), as well as other long period effects caused by the suction of passing ships (see **18.5**).

Where possible, mooring loads should be established on a line by line basis by means of a suitable computer program. Further valuable background data in this respect can be found by discussion with pilots, port operators and ship masters, who are familiar with the port. Such discussions should help to confirm the computer results in respect of the provision and layout of bollards, mooring hooks and other shore-based mooring equipment in relation to the requirements of ships using the berth.

In some cases mooring loads can 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** and guidance on mooring design is given in BS 6349-4:1994.

Section 4. Sea state

20 General

When designing a maritime structure it is 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 can 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 gives 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 can also be obtained by extrapolating a series of wave measurements made at the site. In general this requires a minimum of one 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 attenuation study unreliable. If inshore wave data are not available then a mathematical model of the area can be used to estimate the inshore wave climate. However, care has to be taken with such models because wave height attenuation due to wave breaking and bottom friction is unlikely to be well represented at small model scales. In addition, in situations where large vessels or structures are to be moored inshore or inside harbours that are in a relatively exposed location, it is necessary to measure long waves with periods of the order of minutes at the site. This is because long period wave motions associated with wave grouping are frequently responsible for the large mooring loads that sometimes lead to large vessels and structures breaking away from their moorings. As these long-period wave motions are 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 processes mentioned before 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 sea walls, breakwaters and harbours, together with methods of determining acceptable sea states for moored vessels. General information on wave characteristics is given in **21**, in which definitions and derivations of terms commonly used to describe wave properties can also be found.

A physical model incorporating a random wave generator can be used, but care is needed.

21 Wave characteristics

21.1 Wave forms

Waves have the ability to propagate energy to distant points, but the water itself does not translate with the wave to any significant extent. This can be seen by observing the motion of a small floating object being carried forward in the direction of wave advance and on the wave crest but moving back in a wave trough. 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 that 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 that increasing orders of complexity can 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. For certain applications in these situations sunusoidal wave theory or solitary wave theory can 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 can be obtained from modern references [12, 16].

21.2 Basic wave properties

21.2.1 *General*

For a given water depth, monochromatic waves can 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 usually denoted by H and, in deep water, by H_0.

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21.2.3 ***Wave period***

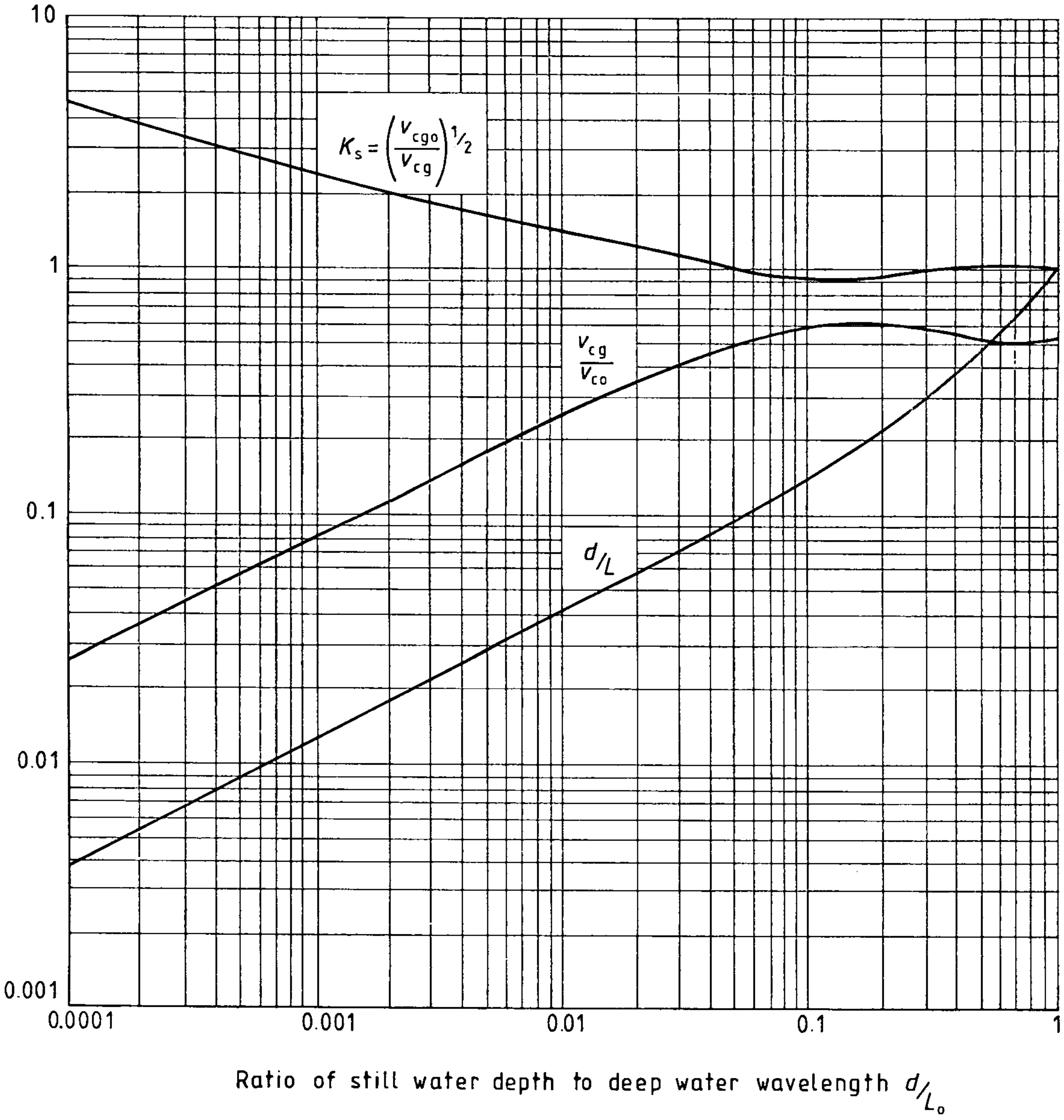
The wave period is the time for two successive wave crests to pass a fixed point and is usually denoted by T. In first order theory the period is assumed to be independent of water depth.

21.2.4 ***Wave frequency***

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

21.2.5 ***Wave length***

The wave length is the distance between consecutive wave crests and is normally denoted by L and, in deep water, by L_0. The variation of wave length in shallow water is shown in terms of the deep water wave length in Figure 3 (see **23.2**). Wave length is related to the wave period and phase velocity (see **21.2.6**). Other information is given in [17].



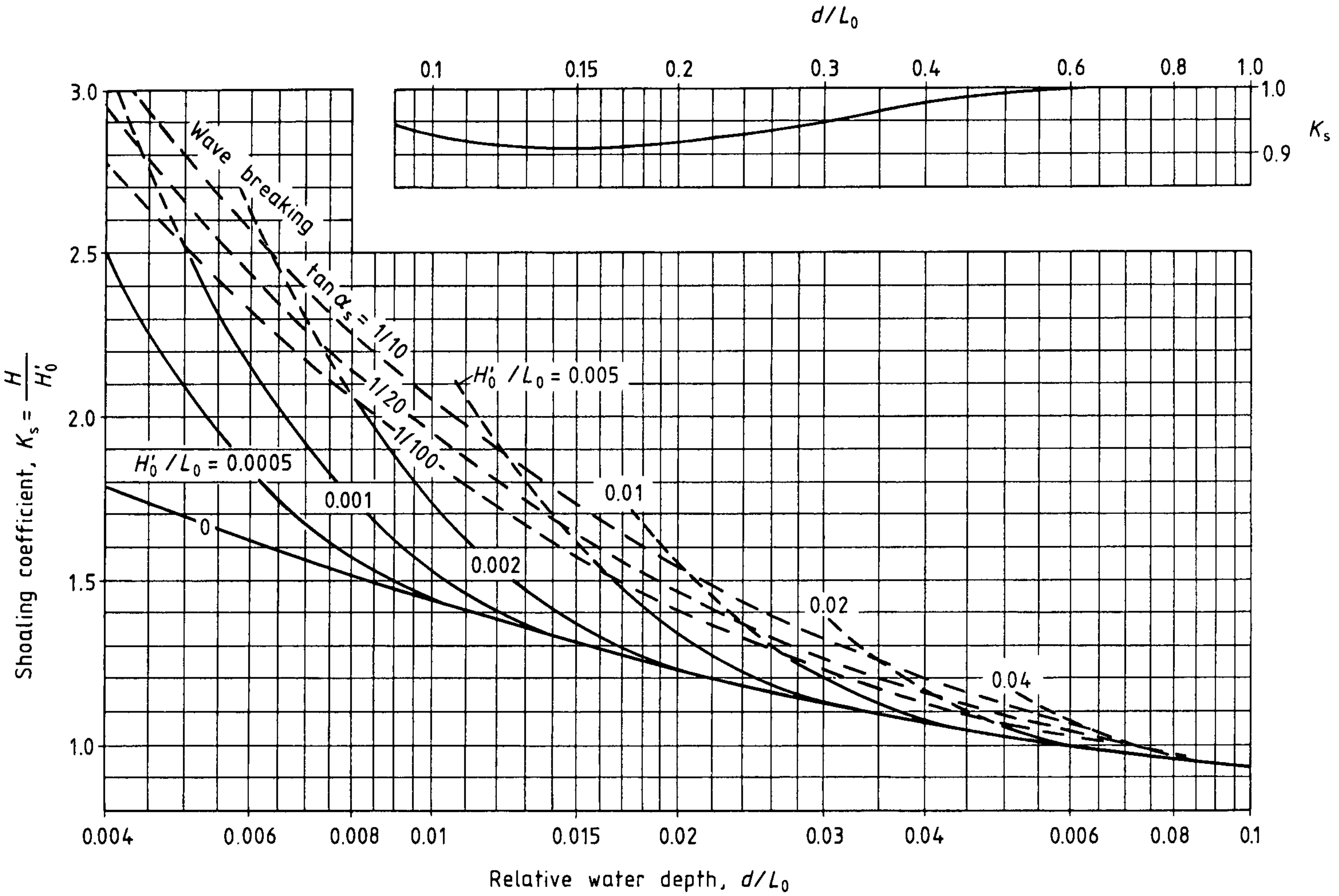
Ratio of still water depth to deep water wave length $\frac{d}{L_0}$

Deep water wave length $L_0 = \frac{gT^2}{2\pi}$

Deep water phase velocity $v_{CO} = \frac{gT}{2\pi}$

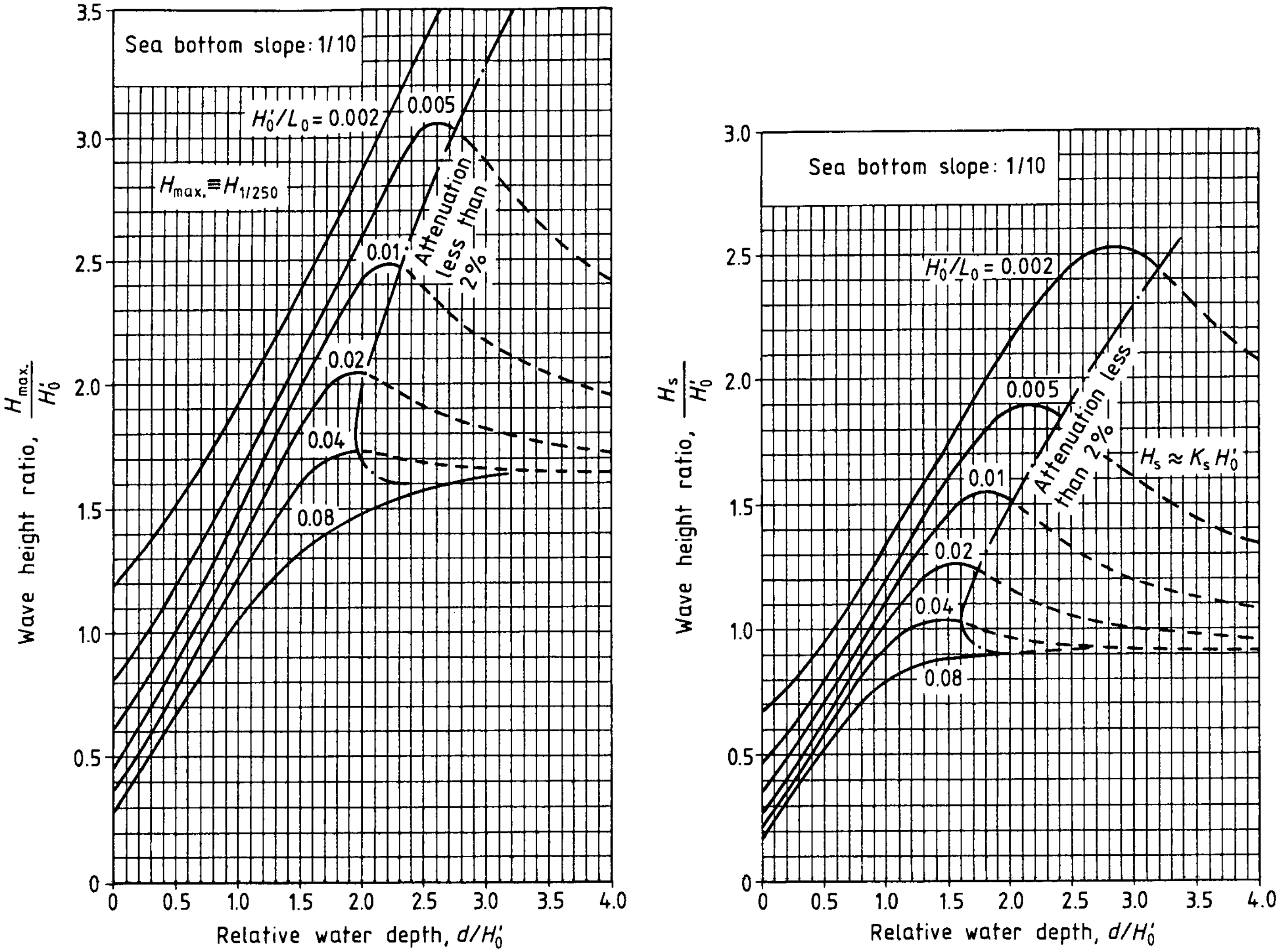
a) Variation of wave length, group velocity and shoaling coefficient with water depth

Figure 3a) — Wave shoaling and estimation of wave height in the surf zone



b) Diagram of non-linear wave shoaling

Figure 3b) — Wave shoaling and estimation of wave height in the surf zone *(continued)*



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

Figure 3c) — Wave shoaling and estimation of wave height in the surf zone *(continued)*

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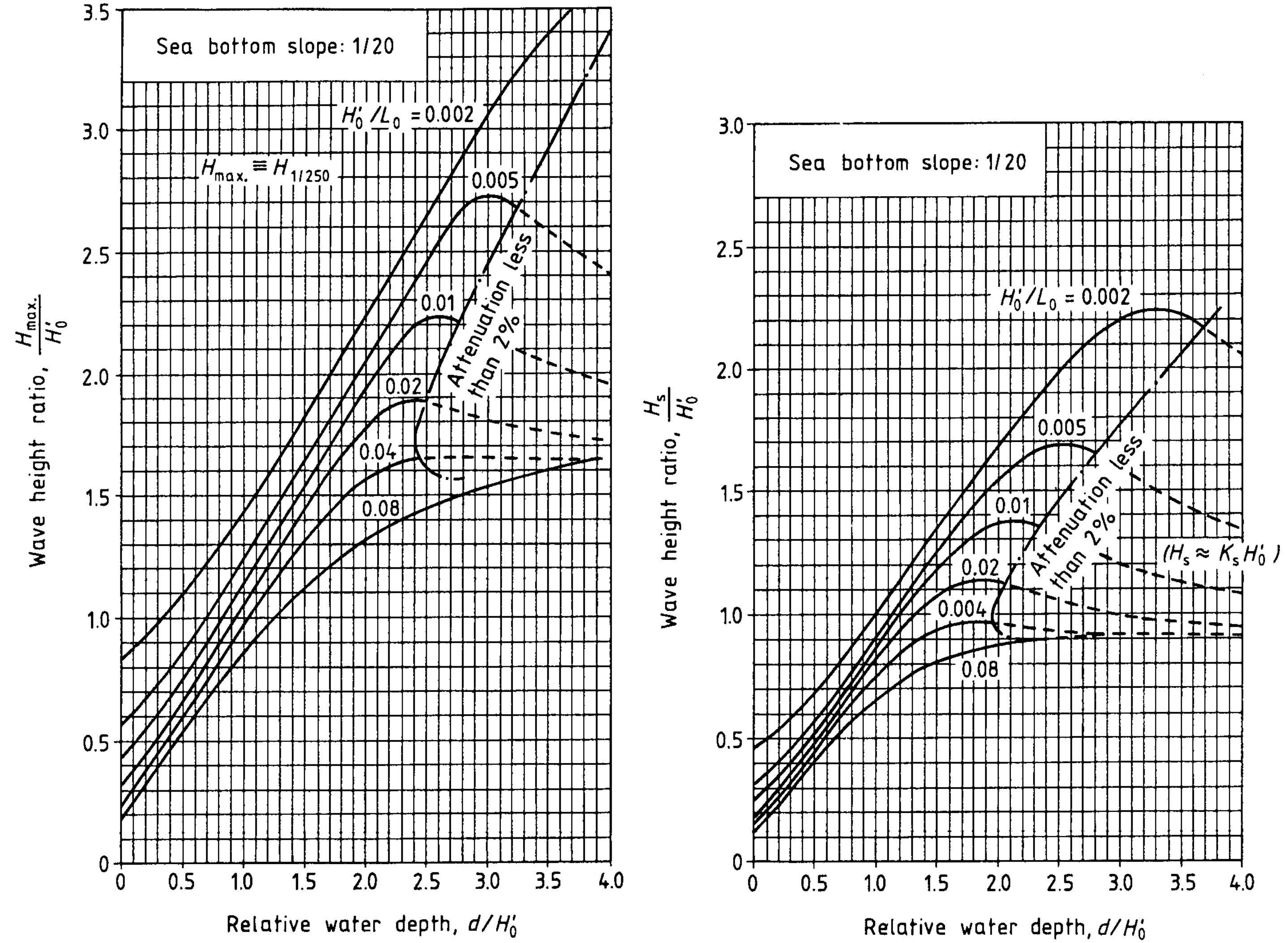
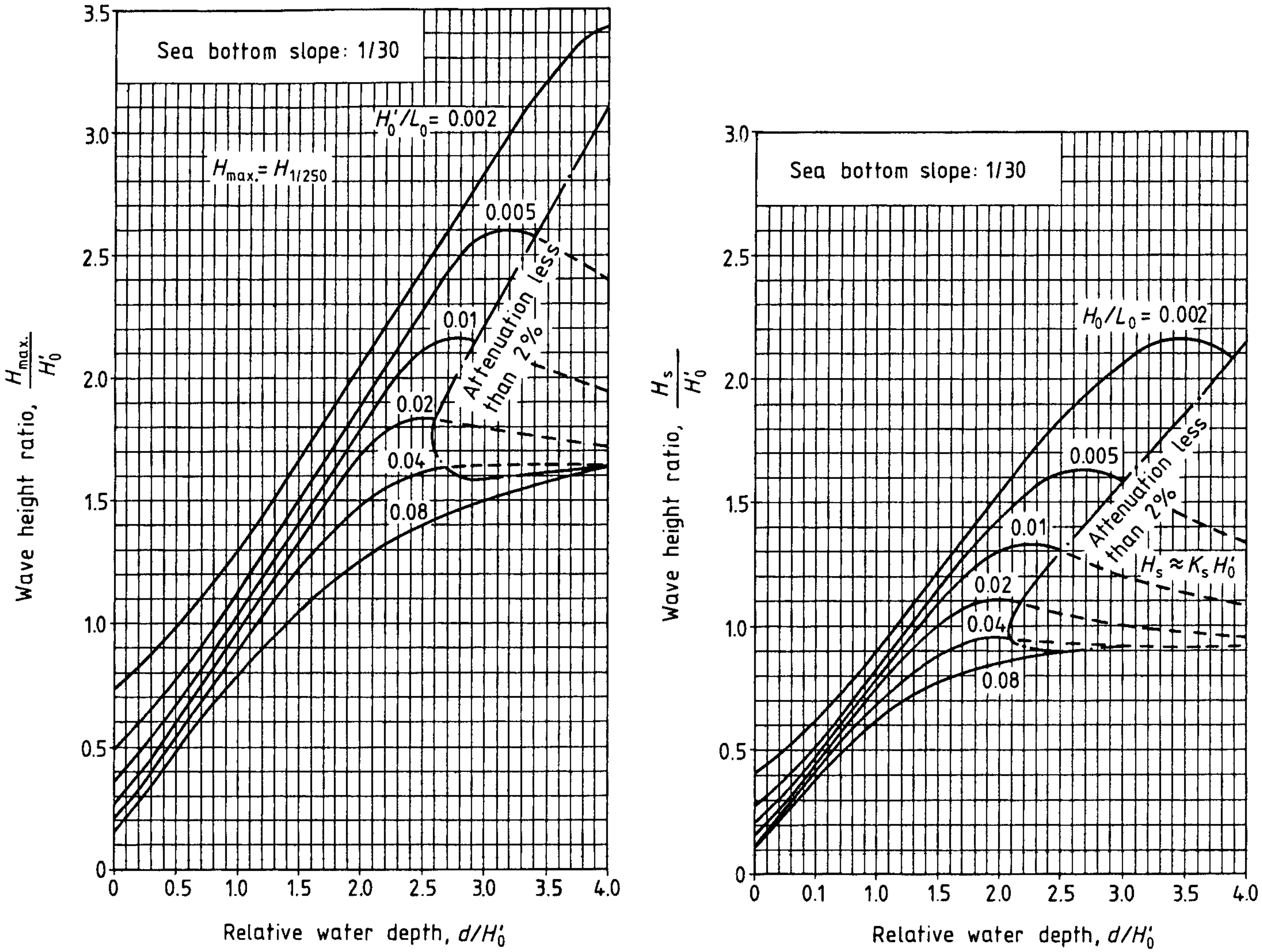


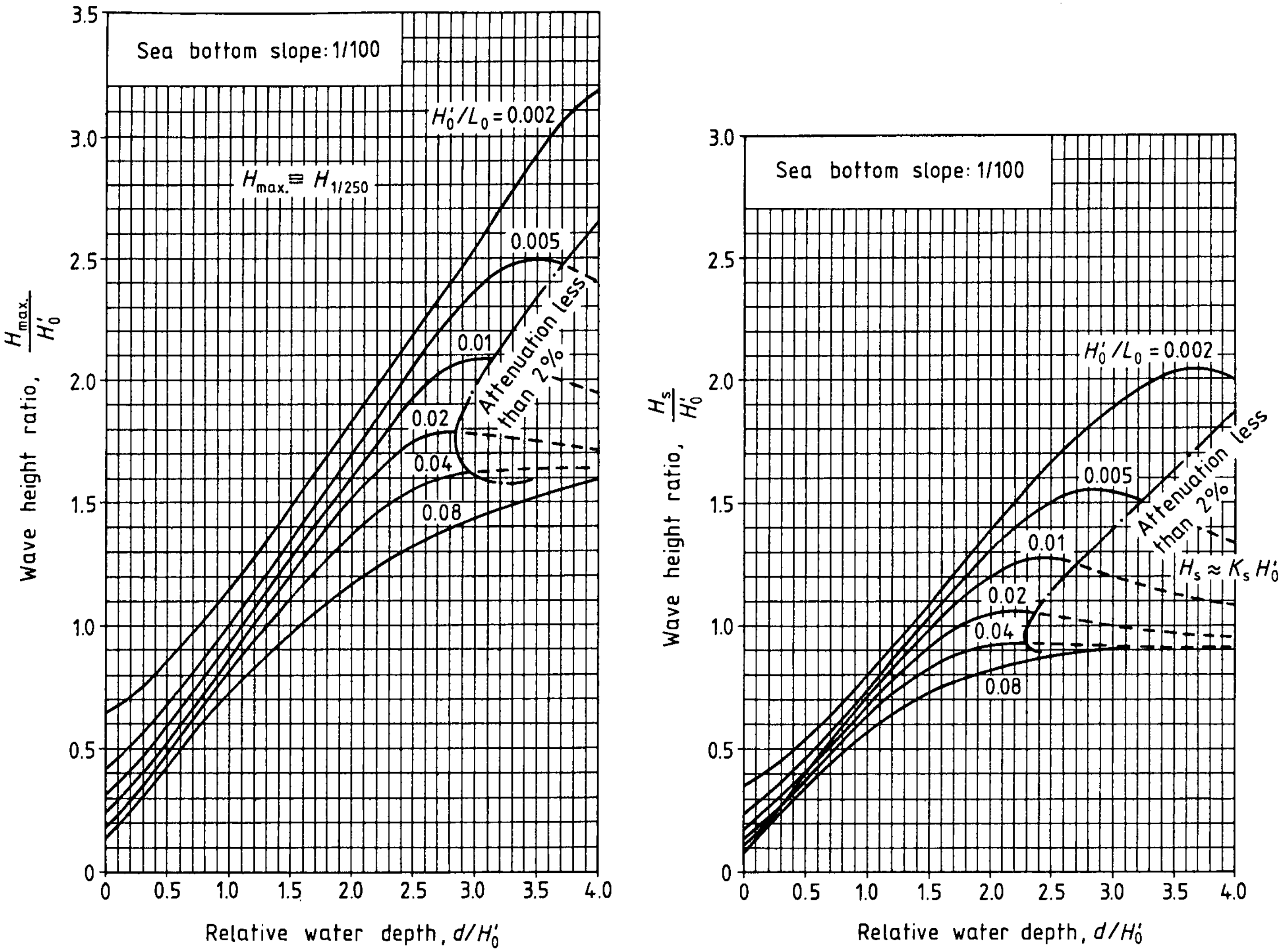
d) Diagrams for the estimation of wave heights in the surf zone (sea bottom slope of 1/20)

Figure 3d) — Wave shoaling and estimation of wave height in the surf zone *(continued)*



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

Figure 3e) — Wave shoaling and estimation of wave height in the surf zone *(continued)*



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

Figure 3f) — Wave shoaling and estimation of wave height in the surf zone *(concluded)*

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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 usually denoted by v_c and, in deep water, by v_{c0}. Phase velocity is related to the wave period and wave length by the following expression:

$v_c = L/T$.

First order theory gives the phase velocity as:

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

where

- d is the still water depth;
- g is the acceleration due to gravity (9.81 m/s^2).

This gives the wave length as:

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

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***

Because 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. Further guidance on this aspect can be found in **39.4.4**. The orbital velocity at the surface is usually much smaller than the phase velocity, v_c, but when it just exceeds v_c, the water particles at the wave crest catch up the preceding wave trough and lead to wave breaking.

21.2.8 ***Wave gradient***

The wave gradient is the wave height divided by the wave length. Comparison of the expressions for orbital velocity and phase velocity indicates 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 gradient of about 1 in 7 in deep water.

21.2.9 ***Group velocity***

A train of waves of single period travelling in still water can be seen to propagate at a velocity less than the phase velocity of the individual waves. In a limited train, therefore, waves 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 usually denoted 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:

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

In deep water, this approximates to:

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

The variation of group velocity in shallow water is shown in terms of the deep water wave length and phase velocity in Figure 3 (see **23.2**).

21.3 Sea state properties

21.3.1 ***General***

Real waves can 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 give rise to a group of larger waves that travel at the group velocity, as described in **21.2.8**. 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 denoted by H_s. It is the average height of the highest one third of the waves, and has been found to approximate to the visual estimate of wave height that would be obtained from an experienced observer.

Other definitions of H_s also exist; the most frequently used is a spectral definition based on integrating the wave energy spectrum to give the zero moment m_0, so that:

$$H_s \approx Hm_0 = 4\sqrt{m_0}$$

21.3.3 ***Significant wave period***

Mean or peak periods T_m and T_p are more commonly used but a significant wave period T_s has often been used in prediction methods based on older American approaches.

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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 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. This value is therefore a measure of the energy of the sea state. It is expressed as a function of wave frequency and direction and is denoted by $I\ (f,\varphi)$.

21.3.6 ***One-dimensional spectral density***

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 $I\ (f,\varphi)$. 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***

This is 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.

21.3.8 ***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 that, on average, separates two occurrences. It should be noted that this does not mean that exactly T_R years will separate two such occurrences. The relationships between design working life, return period and the probability of wave heights exceeding the norm are shown in Figure 4.

For an event with a return period of 100 years, therefore, there is a 1 % probability 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 there is a 63 % probability 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 are necessary in certain situations, but in others they are more expensive than weaker structures, for which the cost of periodical repair is included. By taking costs into account in this way it is 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 10 % probability of occurrence, then the necessary return period from Figure 4 is 200 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 can 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 can 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 or more.

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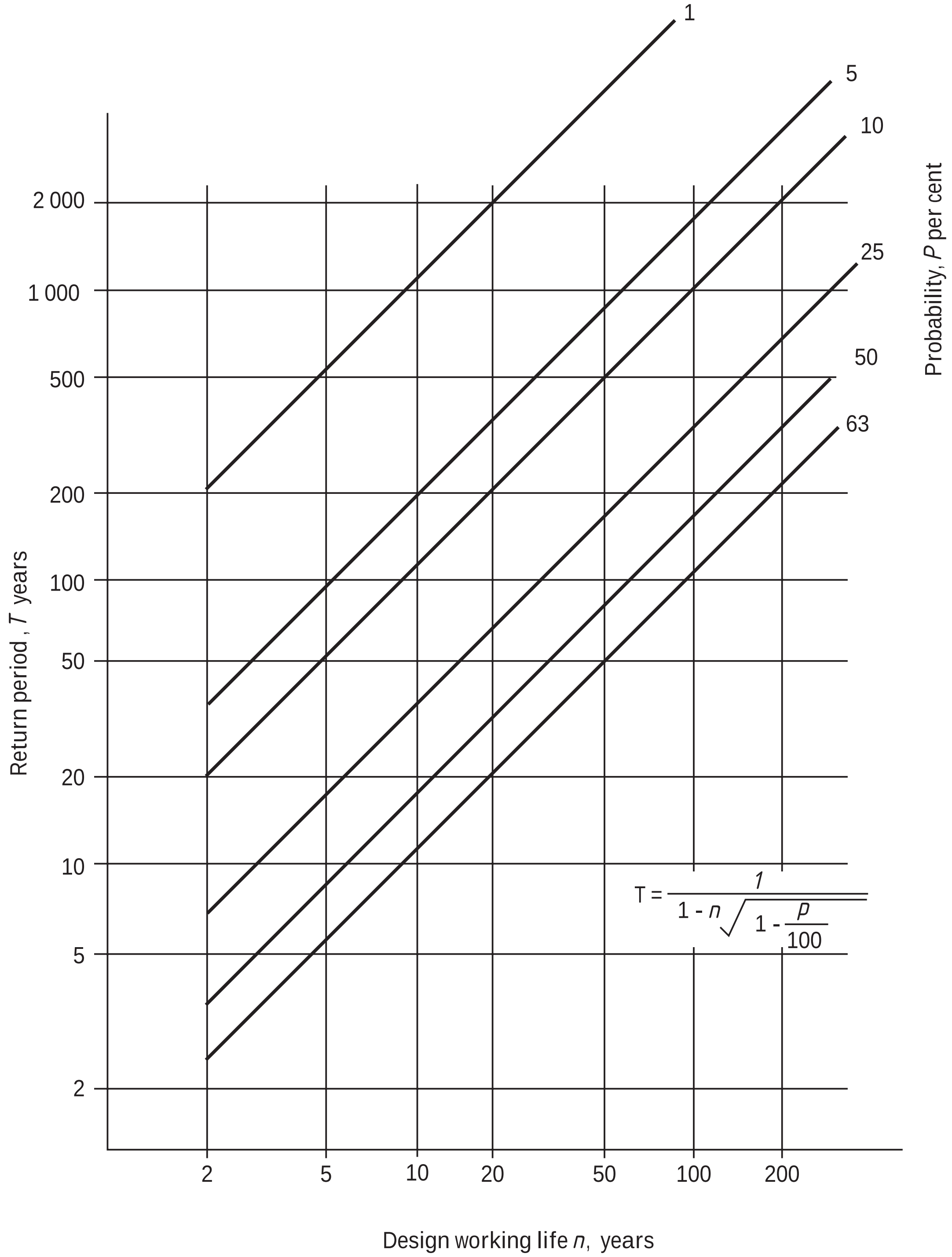

NOTE *T* is the return period of a particular extreme wave condition in years. *P* is the probability of a particular extreme wave condition occurring during design working life *n* years.

Figure 4 — Relationship between design working life, return period and probability of wave heights exceeding the normal average

22 Offshore wave climate

22.1 Wave generation

The following guidance refers to wind-generated waves.

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 that 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 gained of typical one-dimensional wave spectra at the site of interest.

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 can also be affected by swell (see **22.3**). It is therefore good practice to supplement wave forecasts with wave observations made from ships [18] 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, can be used to obtain estimates of wind speed, duration and fetch. In these cases the definition of the wind field, based on calculations that 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 the same as 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 can be reduced where the width of the fetch is much less than its length, but evidence suggests this effect is small. The fetch length should therefore be considered to be the straight line distance from the point at which the wave height is required to the upwind boundary of wave generation. The boundary can 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 databases 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 Figures 5 and 6. These charts can 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 can 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 Figures 5 and 6 represent lines of constant energy. These can 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 000h to 1 600 h and 25 m/s from 1 600 h to 1 800 h, the significant height and period at 1 600 h 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, then moving along this line to the right an amount equal to 2 h, the significant height and period for 1 800 h 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 h would have been fetch limited at 4.4 m and 8.3 s, respectively.

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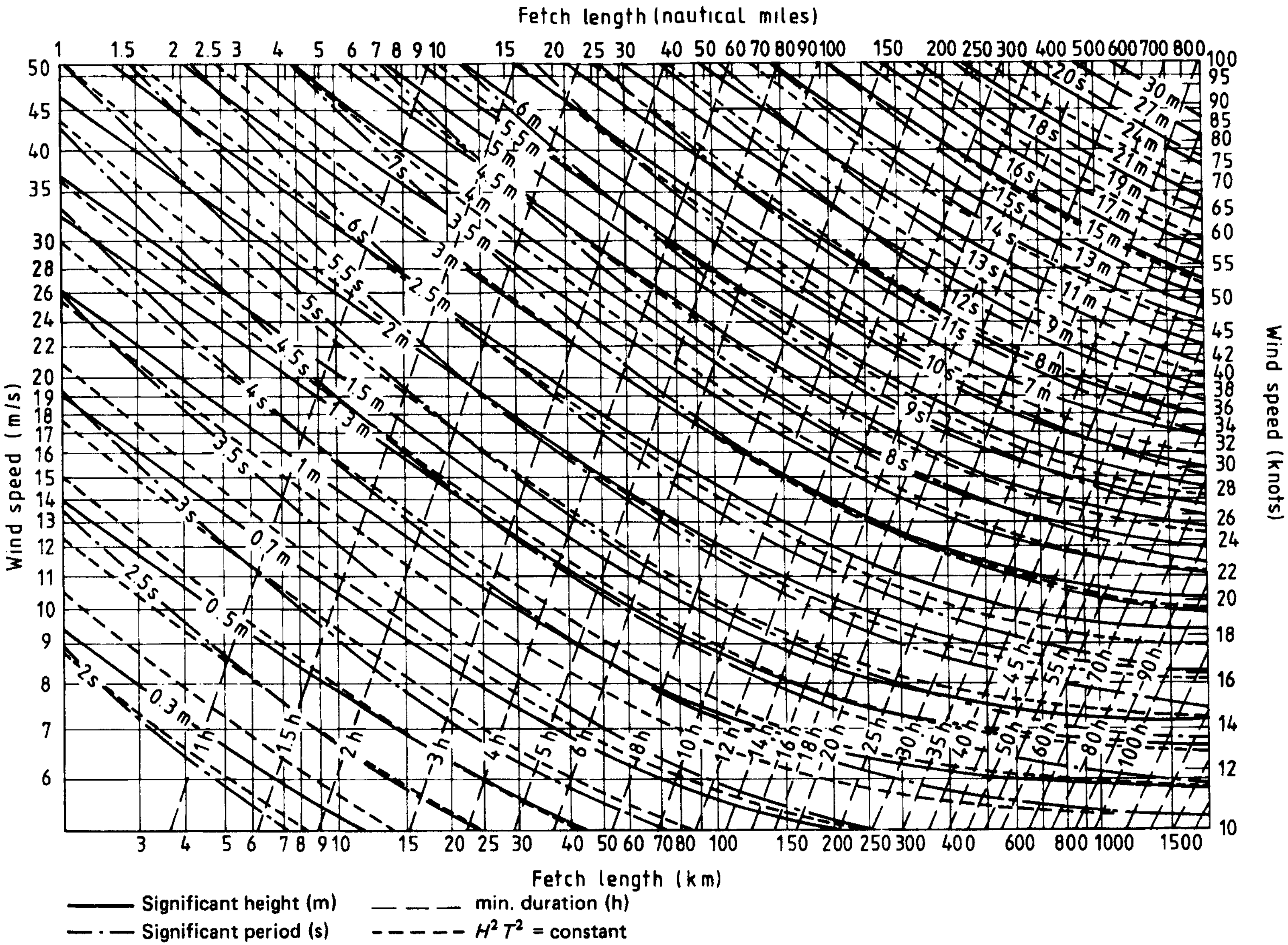

Figure 5 — Significant wave prediction chart — Fetch lengths up to 1 500 km

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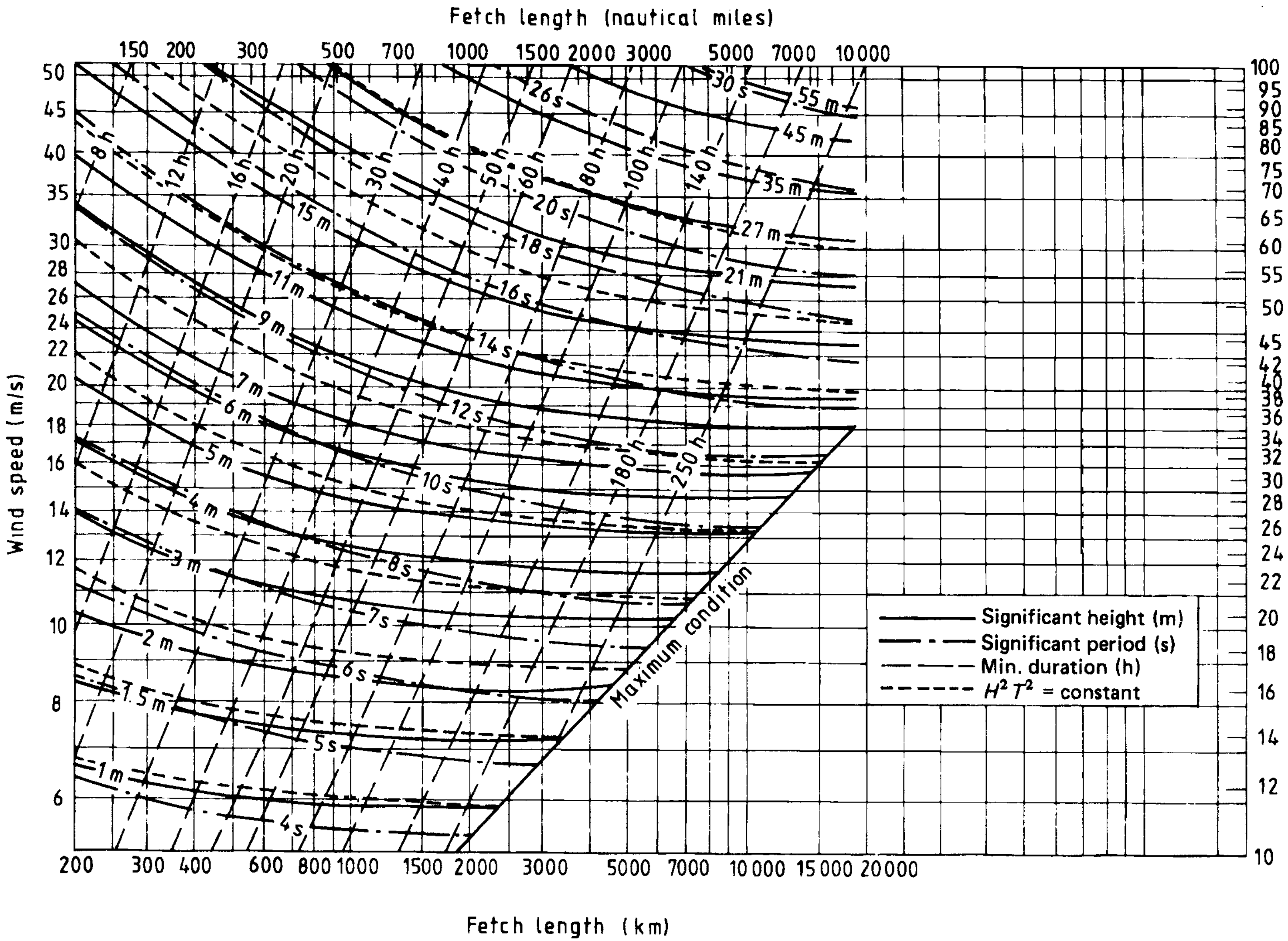

Figure 6 — Significant wave prediction chart — Fetch lengths from 200 km to 20 000 km

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, and these estimates 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 Figures 7 and 8, in which the spectral density, $S(f)$, is plotted against wave frequency, f.

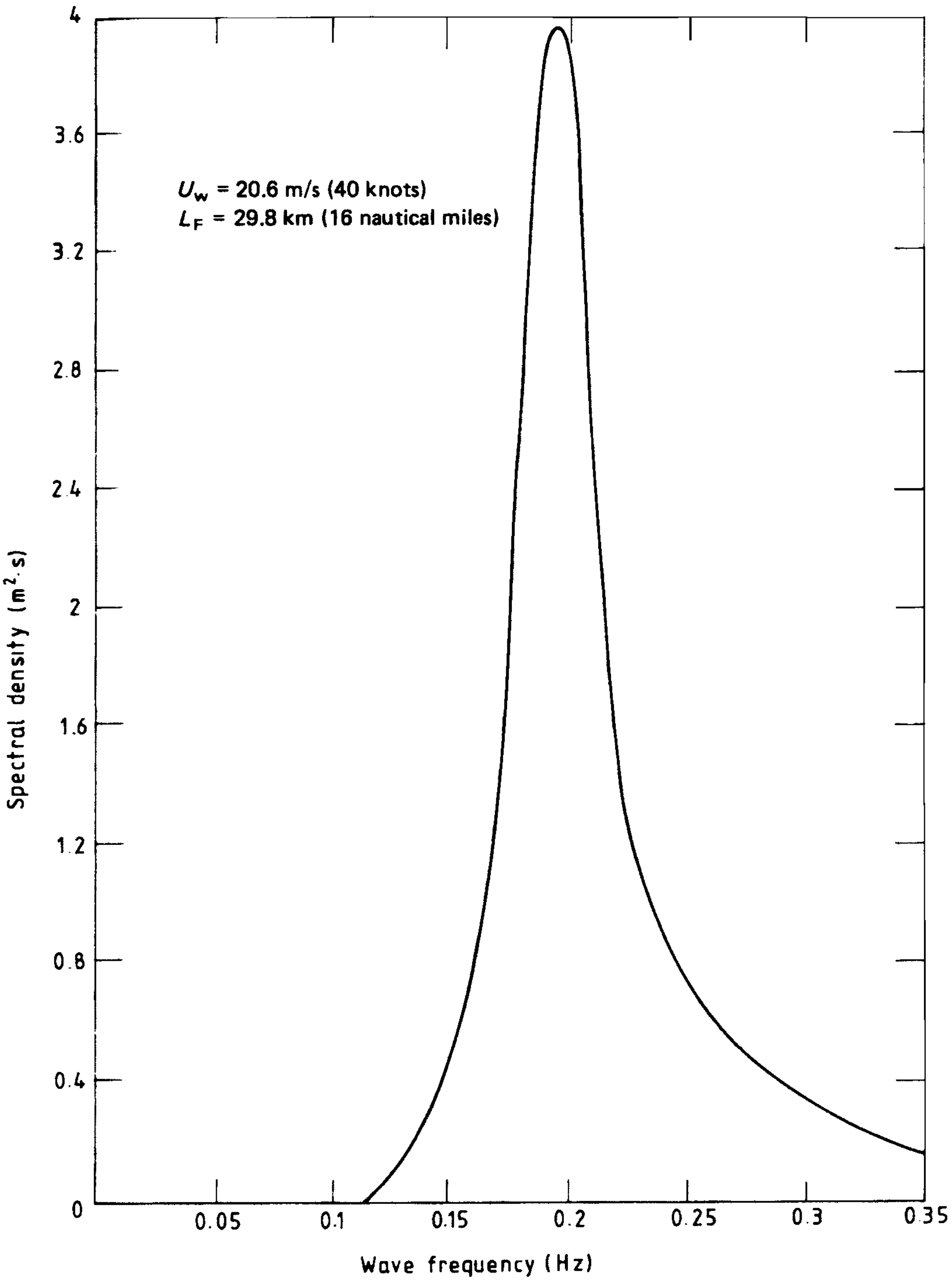

Figure 7 — JONSWAP wave spectrum

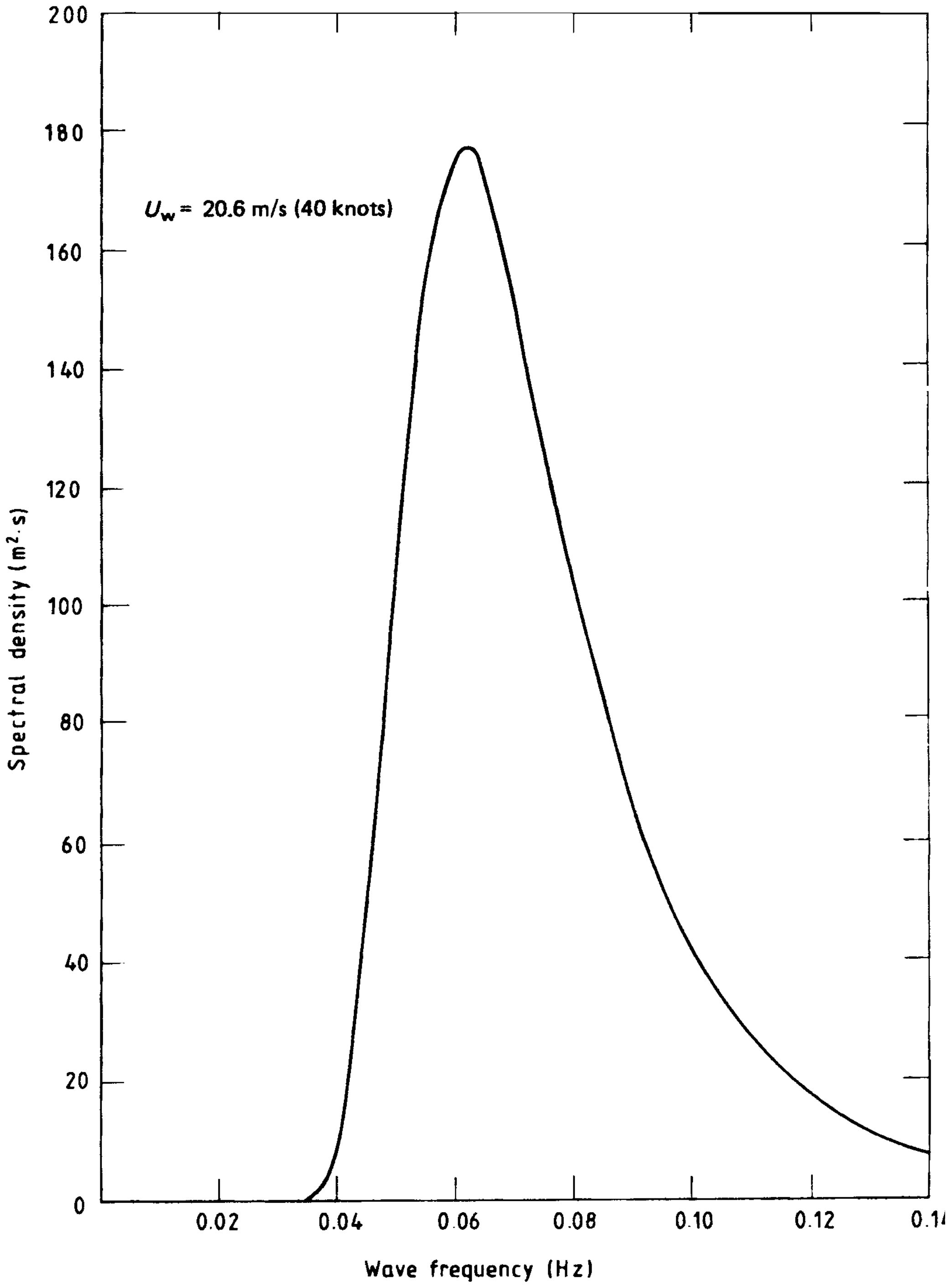

Figure 8 — Pierson–Moskowitz wave spectrum

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, can 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:

$$H_s = 4 \times (\text{area under the spectrum})^{1/2}$$

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:

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

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where

k_j = 0.066 $2/x^{0,2}$

$= 0.033 \left(\frac{(f_m U_w)}{g}\right)^{2/3}$

γ = 3.3;

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

where

ω = 0.07 for $f \leq f_m$

or

ω = 0.09 for $f > f_m$

$x = \frac{gL_F}{U_w{}^2}$

$= \left(\frac{2.84g}{U_w f_m}\right)^{10/3'}$

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 2.84 $g^{0.7}L_F{}^{-0.3}U_w{}^{-0.4}$;

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 those relationships a non-dimensional parameter describing the surface variance was determined from JONSWAP observations. This can 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 previously.

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:

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

where

k_p = 0.008 1;

$f_m = \frac{0.877\ 2g}{2\pi U_{19.5}}$

$U_{19.5}$ 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.

Because 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 Hertz in Figure 7 has moved down to 0.060 4 Hertz in Figure 8. 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 Hertz 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. This 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. The data used to obtain the fetch-limited spectrum were collected for fetches of up to 160 km. The JONSWAP study was therefore 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, thus causing a further migration of the spectral peak to even lower frequencies.

In the absence of any better information it is common practice 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, because the H_s value obtained from the JONSWAP spectrum will exceed that obtained from the fully developed spectrum. This difficulty can be overcome by using Figure 9, where the contours of H_s have been adjusted to give a smooth transition between the two spectra.

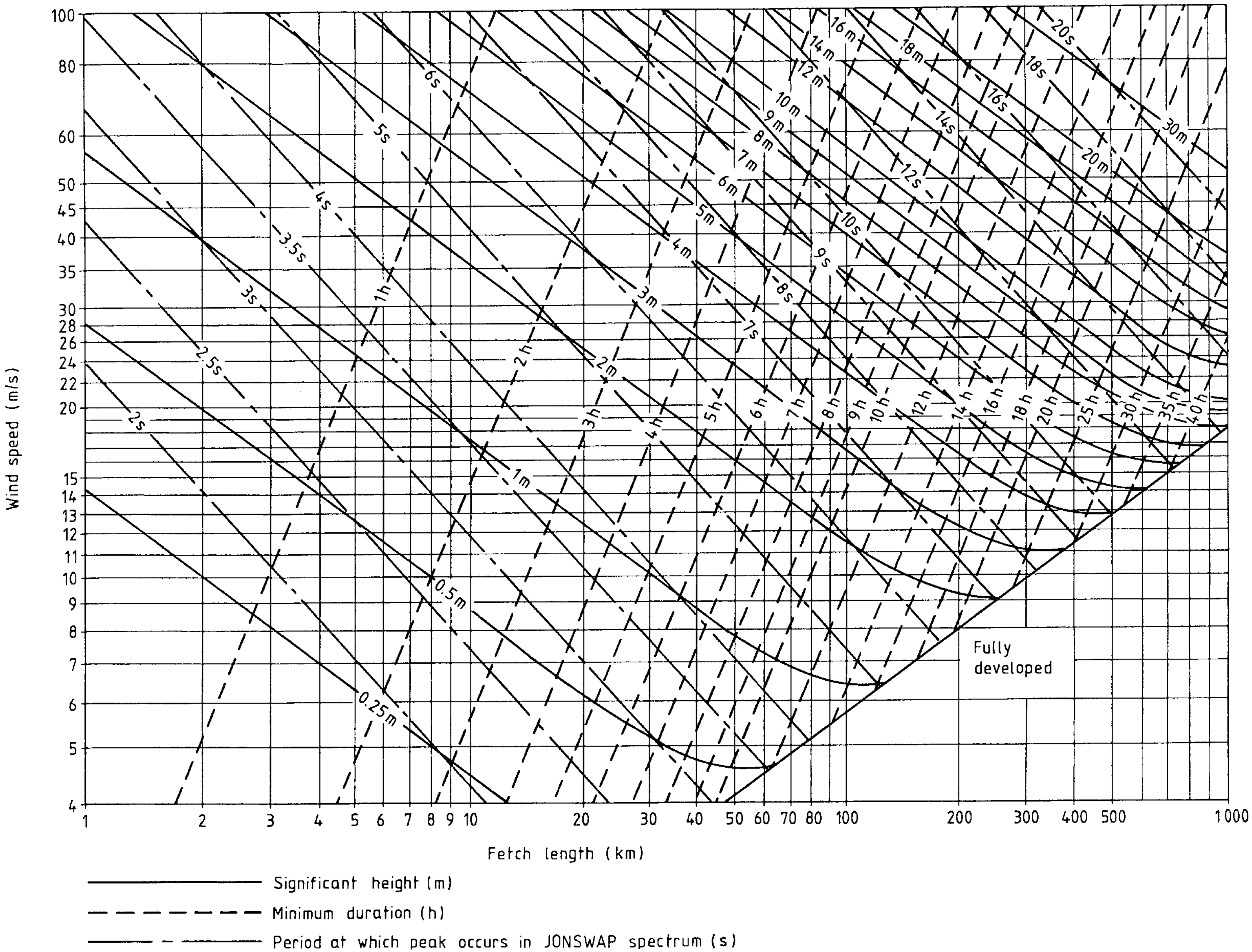

Figure 9 — Significant wave height and peak period for wave spectra

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 [12]. 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 can 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 [18]. Although individual observations taken by eye from a ship can 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 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

When the offshore wave data from one or a number of the previous methods have been obtained, it is necessary to extrapolate these data to obtain extreme conditions appropriate to the design requirements. Extrapolation techniques are described in **4.8**.

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 can be assumed to be constant.

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

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*

Because the phase velocity of a wave decreases in shallower water, a wave front approaching the coast at an angle to the seabed contours is refracted, so that the wave crests tend to align themselves parallel to the bed contours.

The effect is analogous to the refraction of light and the amount of deviation can 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 can 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" can 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 does not take place; whenever waves move into shallow water, however, the shoaling effect, in that wave energy is compressed or stretched in the direction of wave advance in response to variations in the group velocity, is always present.

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 is 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 can 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 $v(b_0/b)$;

where

b_0 is the wave separation in deep water.

Values of the shoaling coefficient, together with wave length and group velocity, can be obtained from Figure 3a).

The shoaling effect upon random waves, including the effects of wave breaking, can be estimated from Figure 3b).

The refraction coefficient can 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 causes energy transfer across rays and thereby invalidates the previous expressions.

23.2.2 ***Computational refraction methods and problems due to caustics***

A number of computer programs are available that 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 gives 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. This 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. Because 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:

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

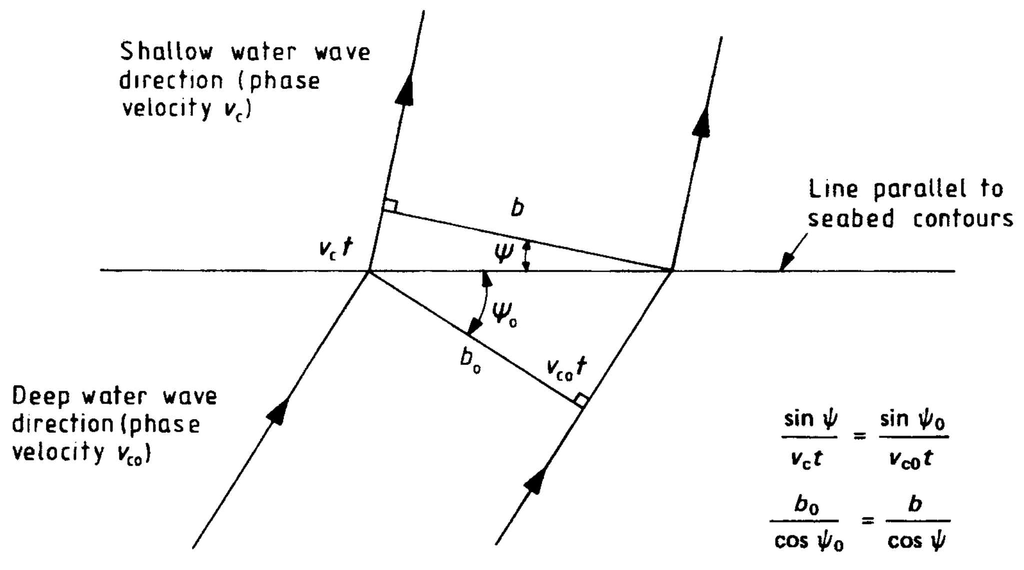

Figure 10 — Schematic diagram of wave refraction

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where

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

$S(f) = \int I(f,\varphi)\, d\varphi$

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 $I(f/\varphi)$ can be expressed in terms of the offshore spectrum I_0 (f,ϕ_0), by:

$$I(f,\psi) = \frac{v_{c0}v_{cg0}}{v_c v_{cg}} \int I((f,\psi_0)$$

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 I_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:

$$\psi_m \int S(f)df = \frac{1}{(\psi_2 - \psi_1)} \int \int_{\psi_1}^{\psi_2} \psi_{I0}(f,\psi)\,df\,d\psi$$

where

ψ_1 and ψ_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:

$$S(f) = \int_{\psi_1}^{\psi_2} I(f,\psi)\,d\psi$$

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 computer programs [19] 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, because the wave crests tend to align themselves with the bottom contours.

23.3 Channel effects

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

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

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 is 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. In the case of extreme conditions, wave breaking can well occur in this region. In addition, because wave energy is distributed over a range of angles, some wave components are always able to propagate across the channel. All these effects 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.

23.4 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 seabed, 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 can 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. This is plotted in Figure 11 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 11 that it is only necessary to take bottom friction into account when information on wave parameters is required in very shallow water such as might 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 can 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 11 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 that 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 can be due to errors in extracting other shallow water effects from the data.

Until satisfactory mathematical models are developed it is necessary 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 can be obtained by taking a bed friction factor of 0.01.

23.5 Wave breaking

In deep water waves break before reaching a limiting wave gradient (see clause **21**), but in shallow water it is necessary to classify 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 gradient (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, is dependent on wave shoaling.

Definitions of the various types of breaking wave are illustrated in Figure 12 [20].

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 give an upper limit to the wave height, can 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 J, the ratio of the offshore wave gradient to the square of the beach slope, tan α_s, 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.

Model tests with irregular waves plunging on a beach slope of 1 in 30 in front of a rubble-mound sea 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 might be one factor operating in random waves that leads to maximum wave heights in excess of the still water depth.

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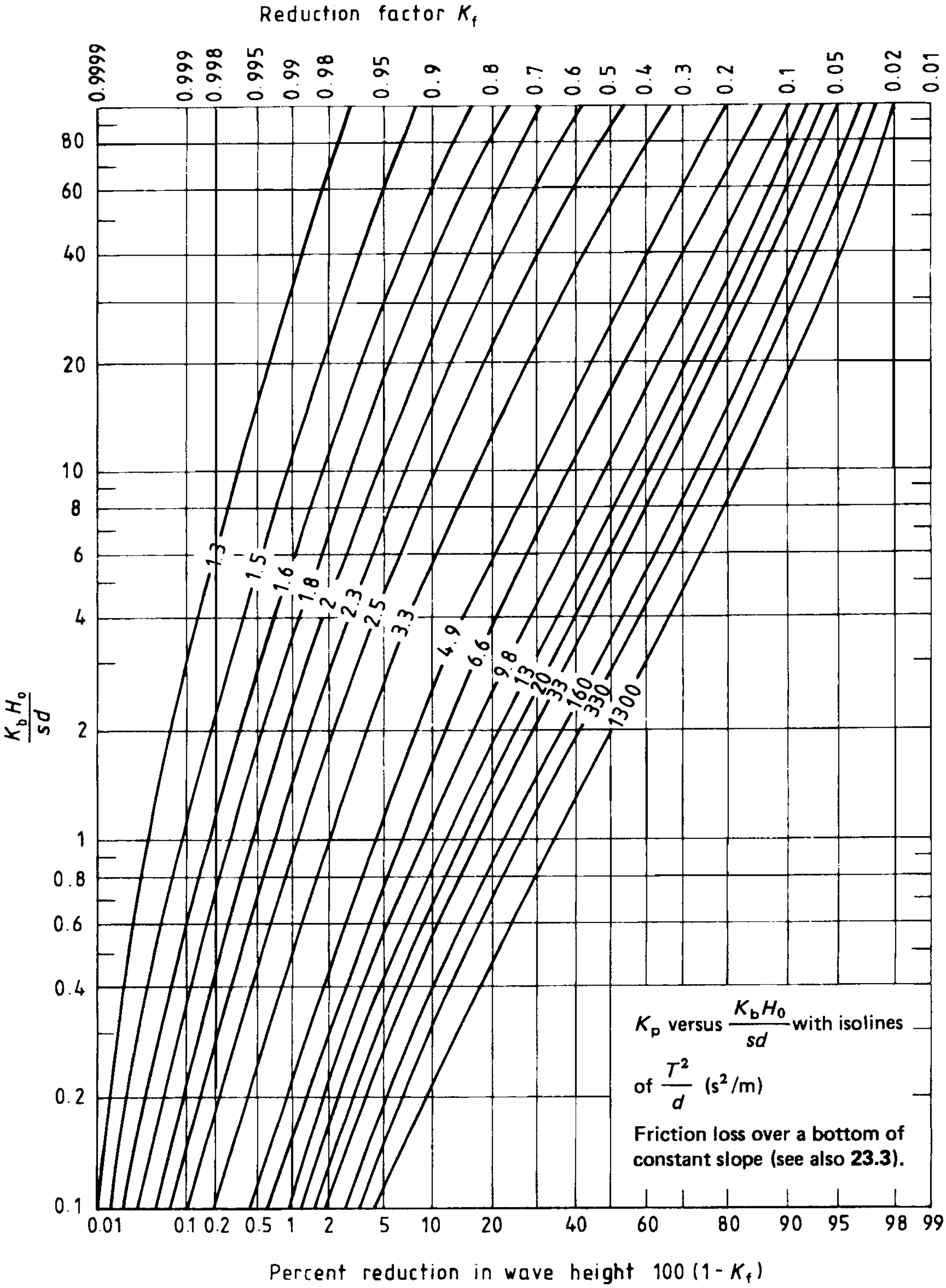

Figure 11 — Wave height reduction factor for bottom friction

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

A method has been developed [21] from the combined results of model tests and prototype observations, which can 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 gradient 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 clause **28**, the maximum wave height should 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 $H_{1/250} = 1.8\,H_s$. Figures 3c) to 3f) 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 3b).

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.

$$T_p = (T_p)_0$$

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

The wave height can be estimated using the following equations:

If $d/L_0 \geq 0.2$

$$H_s = K_s H_0'$$

If $d/L_0 < 0.2$

H_s is the lowest of the following:

$$H_s = \beta_0 H_0' + \beta_1 d \text{ or}$$

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

$$H_s = K_s H_0'$$

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

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

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.

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

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

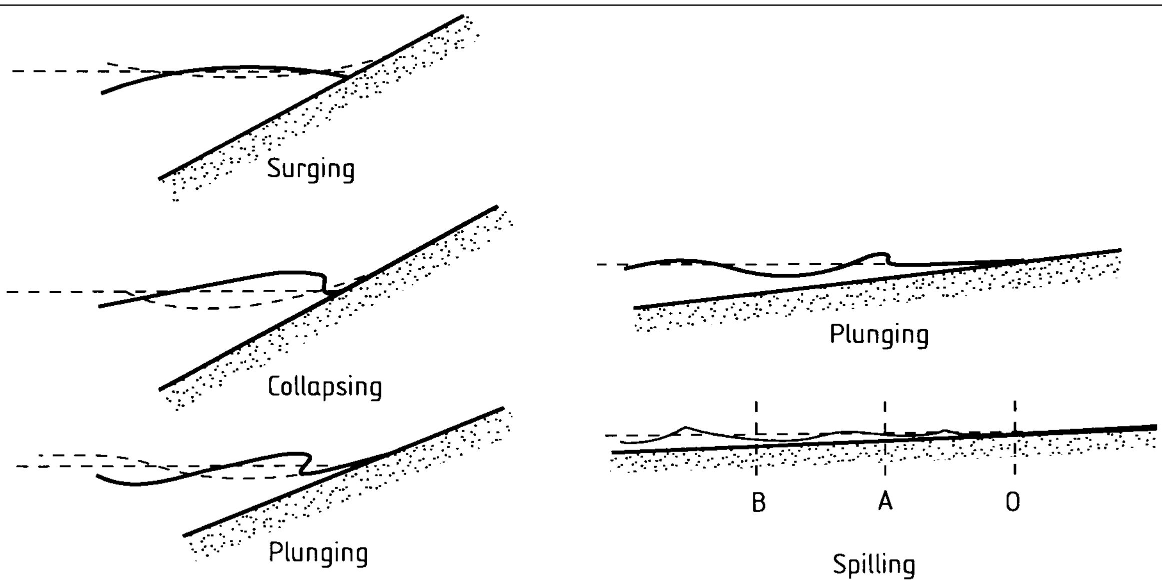

Figure 12 — Breaker types

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

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

differences can exceed 10 % with a similar difference for $H_{1/250}$. There can 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 that are normally 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 at their moorings have natural periods of oscillation that are usually 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, because the damping of such slow movements is small, large mooring loads can be developed.

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 that 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 that 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 that is less than the phase velocity of a long wave of the same period.

Where the water depth to wave length 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:

$$H_{sd} = 0.01 \frac{g H_s^2 T_z^2}{d2}$$

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, has an associated set-down with a significant height of 0.34 m in a depth of 12 m. The average period of set-down is associated with the inverse of the width of the one-dimensional wave spectrum. There is little empirical information to support the previous relationship for the height of set-down so it should only be used as a general guide to its magnitude.

When the set-down reaches the coastline the primary wave system is normally 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 usually called surf beats.

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

24.4 Tsunamis

These waves are usually caused by earthquakes. One of their main features is that, although their height in deep water can 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 usually 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 usually produced by wind set-up. Wind blowing over the sea induces a surface current that 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 can 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 [12] 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.

Because 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 interdependent it may be assumed, as a worst case, 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. This assumption gives an over-conservative answer. If the available data suggest that the two are independent, however, advantage is to be gained from this independence, provided care is taken in specifying the design conditions. This is because 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 that can 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, it should be assumed that they are dependent on one another. This can, though, produce 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 collection by volunteers observing vessels along major shipping routes [18], but a marked scarcity of localized detailed information, particularly in coastal waters.

The British Oceanographic Data Centre, at the Proudman Oceanographic Laboratory, Bidston, maintains an international inventory of such data.

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

26.2 Site measurements

In the absence of suitable existing wave data, site measurements should be taken. Such site measurements, which can 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 can normally be taken as the significant wave height for that period. Instrumental records are always preferable, because it is likely that the maximum waves will be missed or inaccurately estimated even with frequent observation periods. In addition, 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 normally more reliable and therefore likely to produce a higher data return.

Where instrumental 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 one 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 8 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. These long waves are usually correlated with high storm or swell wave activity, so it is often sufficient to arrange for the data logging system to switch from the periodic recording mode to the continuous recording mode. This should be done 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 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 one year.

To determine the general offshore wave climate, the wave recording instrument should be sited adjacent to the study area in water that is deep relative to the wave lengths. If possible the instrument should be positioned so that signals taken from the waves at right angles can be easily transferred to the points of interest inshore. The recorder should be sited away from local features that could cause reflection or diffraction, and from uncharacteristically strong currents, which can 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 the advantage that they do not require a fixed structure for support, and need only a suitable mooring system. They are suitable for waves with periods of from 3 s to 20 s.

A 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 disadvantages are that they require a fixed structure for support and regular maintenance.

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*

Pressure fluctuations measured by a pressure transducer 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 can limit their use.

Inverted echo sounders mounted below the surface are also capable of measuring waves as they pass overhead. The measurements are unreliable for storm waves, though, because, where the surface becomes aerated, this causes 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 can be used to measure waves. 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. Satellite based altimeters, if calibrated against data from a buoy-mounted wave recorder, can provide wave height records that are as accurate as those from the buoy itself. Satellite altimeters provide data only when they pass over the area of interest and, because the time of return is several days, it takes a few years to collect enough data to enable wave height predictions to be extrapolated with confidence.

Because of their wide "footprint" they are most suited for use in open oceans rather than coastal waters, where wave conditions can vary considerably over distances of less than 1 km.

Satellite altimeters do not provide data on wave direction or period.

26.3.4 ***Directional systems***

Wave direction can be measured by several means, using off-the-shelf equipment, but the systems in current use are expensive. 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 scatter, 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.

It is prudent to check wave direction records by visual observation, local knowledge and 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 can 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 has been replaced by electronic recording and direct computer analysis. It is wise to retain the raw data, whichever recording system is used, in order 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, can be processed manually [22] [23].

The analysis gives an estimate of H_S and T_Z for each record, which can then be extrapolated to give significant wave heights for the required return period and, when required, can be further extrapolated to give design maximum wave heights for various probabilities. Alternatively, the average maximum wave height occurring between records can 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 can also be analysed using a curve-following machine, which converts the chart data into digital data that can be processed on a computer to obtain the one-dimensional wave spectrum. Where the information is directly recorded 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 can be divided into the following two types:

a) an 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) an 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 can comprise a set of significant wave heights or it can comprise a set of maximum wave heights already obtained from the significant heights by individual extrapolations of the type described in b). It is important, however, 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 gradient (see **27.4**).

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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 can be introduced.

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, because their reliability can be affected by long-term changes in the climatic pattern.

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 database 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**. There is doubt about the validity of the results, though, because the original values applying at every 3 h or 4 h interval in the year cannot be considered as statistically independent.

This difficulty can be overcome if the original data set is reduced to include only the maximum wave heights associated with the peak of a storm, because storms can 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.

27.2.2 *Wave heights*

The method consists of plotting the initial wave heights against their cumulative probabilities of occurrence. It uses an appropriate probability function, with the object of achieving a straight-line graph that can 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) can be denoted by:

$$\frac{n}{n_x + 1}$$

Therefore the probability, p_n, that H_n is equalled or exceeded is given by:

$$p_n = 1 - \frac{n}{n_x + 1}$$

Values of p_n can be calculated directly by the previous 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. Sometimes one distribution will fit the lower wave heights well and 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 can be obtained, with the aid of tables, from:

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

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 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 was obtained. Provided the original set is statistically independent and representative of typical conditions, then extrapolation to increasingly lower probability values, and therefore longer return periods, can be made. By definition, the return period is that period during which the event occurs 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. By substituting n_x for n in the expression for p_n shown previously, this gives a probability of:

$$\frac{1}{n_x + 1}$$

Therefore, the wave height with a given return period T_R will have a probability of occurrence of:

$$\frac{T_o}{T_R\,(n_x + 1)}$$

on the probability plot, which can 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, because any shorter duration is unlikely to yield a representative set.

Although extrapolations of wave heights measured at inshore sites can be made using these techniques, 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. This is because the wave climate can be limited by breaking criteria (see **23.4**).

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.

The extrapolations made to obtain the design conditions normally 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 can 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 can 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 **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, is simply a function of the average wave parameters and duration of the design conditions. The most probable value of the maximum wave height is the average maximum, and this can be estimated from the following expression:

$$\overline{H}_{max} = \frac{H_s}{\sqrt{2}}\{(\log_e N)^{½} + 0.288\,6\,(\log_e N)^{-½}\}$$

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:

$$H_{max} = \frac{H_s}{\sqrt{2}}\left\{\log_e N - \log_e \log_e\left(\frac{1}{1-p}\right)\right\}^{½}$$

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 13.

In the case of inshore sites the previous methods of obtaining maximum wave heights can be applied, but within the limitation that the inshore significant wave height cannot exceed the height at which breaking occurs (**23.4**).

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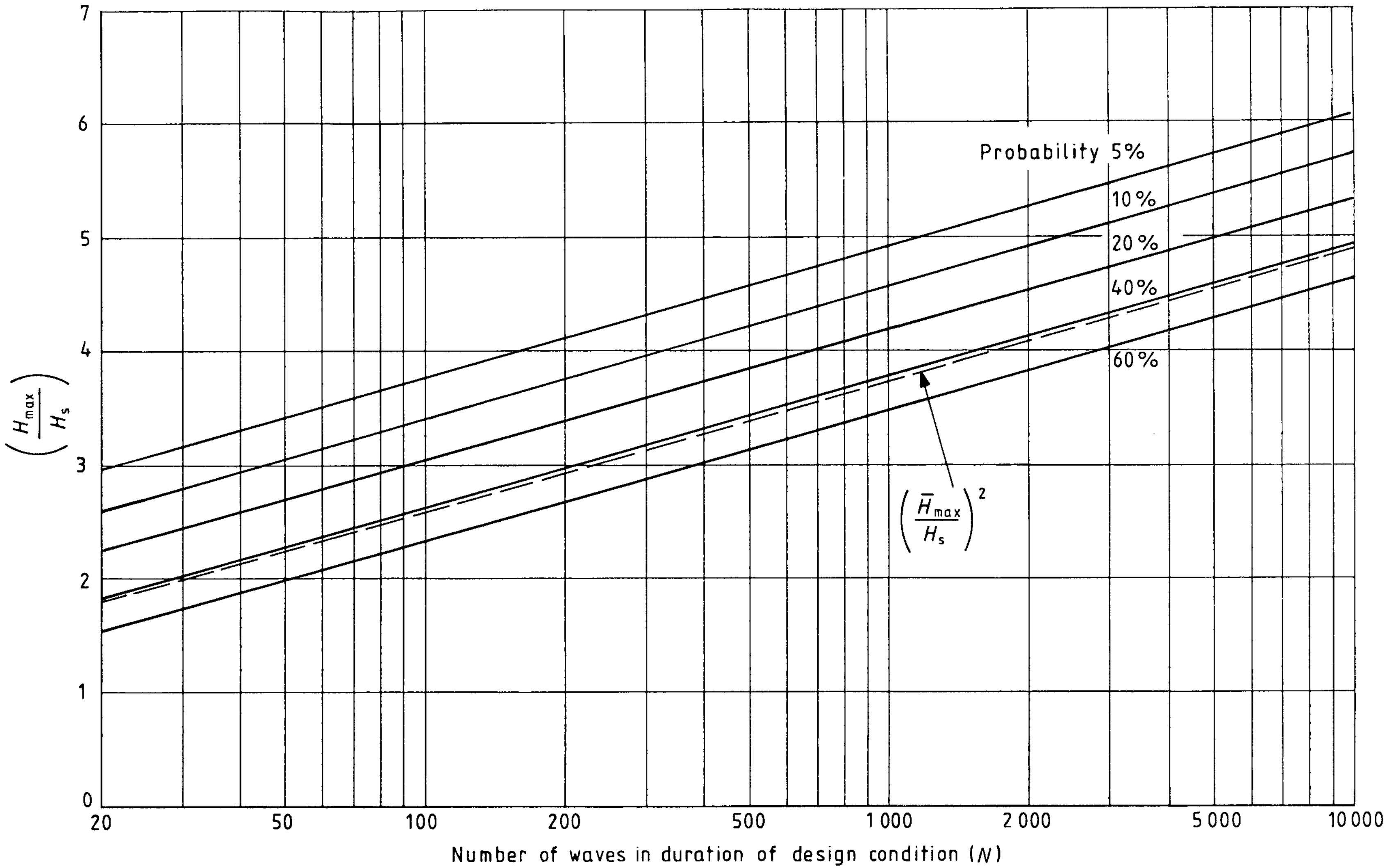

Figure 13 — Variation of $(H_{max}/H_s)^2$ with N

27.4 Extrapolation of wave periods

Whether the wave information used for the extrapolation of wave height was obtained from forecasts from wind using Figures 5 or 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_m, 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:

$T_m < T_s < T_p$

It is usual to plot a scatter diagram of wave height against wave period on which curves of constant wave gradient can also be plotted. In general, this diagram suggests a prevalent value of wave gradient that can be assigned to the design condition. However, if there is considerable scatter in the values of wave gradient it might 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 gradient, i.e. longer wave periods, usually 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 gradient, $2\pi H_s/(gT_z^2)$, in terms of significant wave height and the deep water wave length associated with the zero-crossing wave period, is typically within the range 0.04 to 0.06. For a fully developed sea state it can be taken as 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 usually significantly longer than the zero-crossing wave period, T_m. For a fully developed sea state $T_p/T_m \approx 1.4$.

28 Effects of breakwaters and sea walls on sea states

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.

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 can create 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. This 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.

Examples of various types of breakwater are given in BS 6349-7:1991. In some cases breakwaters consist of vertical faced structures 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 [24].

Figure 14 shows the run-up on a graded riprap slope and Figure 15 shows the run-up on a rubble-mound slope (see **57.3.3** for an explanation of “riprap”).

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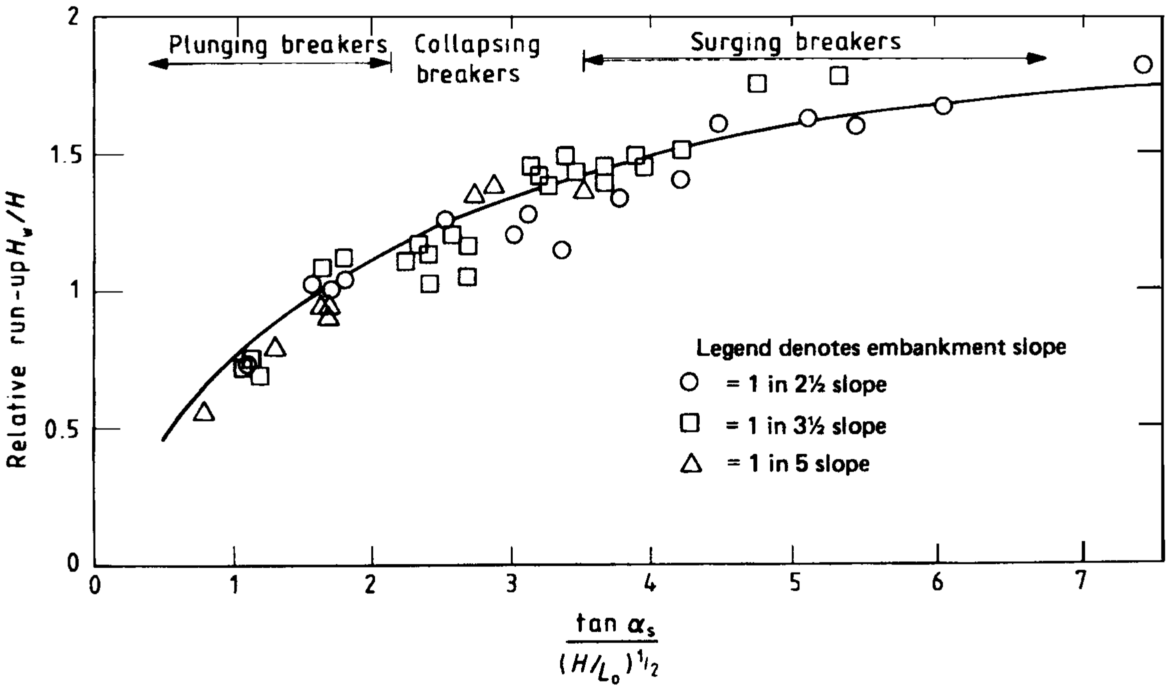

Figure 14 — Run-up on graded riprap (see **57.3.3**) **slope**

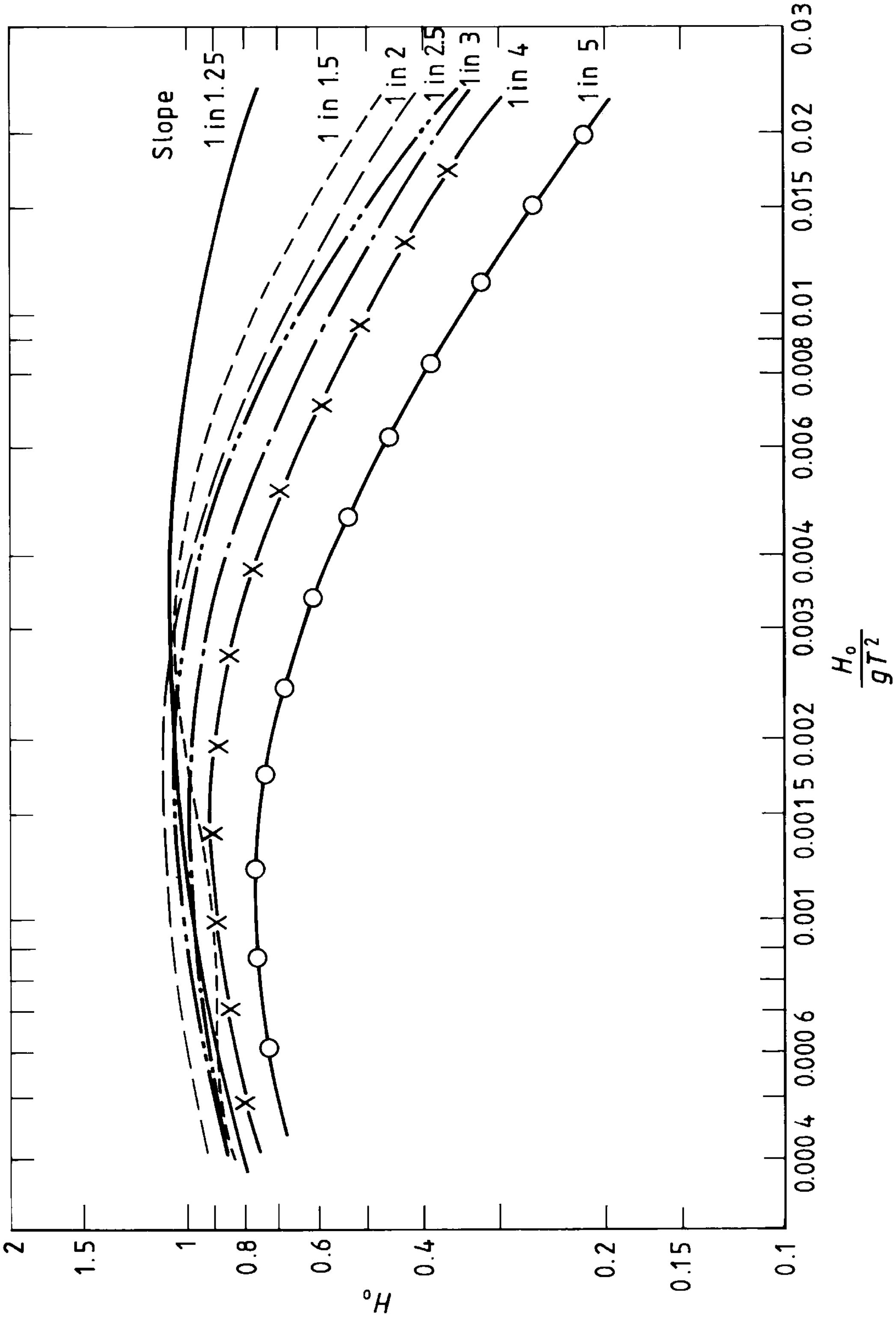

Figure 15 — Run-up on rubble-mound slope

29 Harbour response

29.1 General

A general requirement in the design of harbours is the ability to estimate the degree of shelter that results from any given layout of breakwaters. The wave pattern within a harbour with a breakwater arm 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.

With this simple type of breakwater arrangement the wave inside the harbour cannot usually be reduced to less than one third of the external wave height.

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 can undergo further reflection and for certain wave lengths these multiple reflections can reinforce one another, giving rise to an amplification of the incident wave height. This effect, known as harbour resonance or seiche, 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 dissipate the wave energy (see clause **28**). Natural beaches dissipate storm and swell wave energy well, so a careful evaluation should be made before making any harbour modification that would decrease or eliminate such areas.

In **29.2** 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 effects of wave diffraction for a flat seabed can be assessed in two ways, namely:

a) on the assumption that waves are of single frequency, for which methods of analysis are given in the literature (e.g. [12]);

b) on the basis of random waves, which is more realistic.

In nature, most seas are composed of waves of many frequencies and directions. Single frequency diffraction diagrams give a misleading impression of the shelter provided by a breakwater, if they are applied to an equivalent wave of a period equal to that of the significant period of a random sea. In general, when the incident sea is more random than normal, more energy is diffracted into the shadow area. Because the total admitted to the harbour is limited by the width of the entrance, there is a corresponding reduction of wave height elsewhere.

29.2.2 *Diffraction of a random sea*

In deep water a random sea contains components travelling in directions other than the principal direction. It is normally assumed that within the generating area components can travel in any direction but the directional spread of wave energy is still a subject for discussion. Measurements made in the North Sea during the Joint North Sea Wave Project, which led to the JONSWAP wave spectrum, supported the hypothesis that the amount of wave energy travelling in any direction is proportional to the square of the cosine of the angle between the direction of the component and the principal direction. In areas outside the generating area the distribution becomes progressively narrower with increasing distance from the source of the waves. There is also evidence that the distribution becomes narrower as waves advance into shallower water.

Figures 16 and 17 can be used to make a preliminary estimate of the likely effects of wave diffraction. The figures are based upon a distribution of wave energy that is cosine squared and a Pierson–Moskowitz spectrum in the water depth at the breakwater. They do not differ significantly from similar figures calculated for a JONSWAP spectrum and can be used with such a distribution of wave energy. Figures 16a) and 16b) give diffraction coefficients for breakwater gaps of one and two wave lengths respectively, and four angles of incidence at distances from the gap, expressed as multiples of the gap width. The wave length to be used should be that corresponding to the peak period of the wave spectrum, i.e. the wave period at which the energy density is greatest. Figures 17c) and 17d) give similar information for an island breakwater with a length of one or two wave lengths respectively.

For further discussion of the variation of directional spreading and its effects upon diffraction around breakwaters see [22].

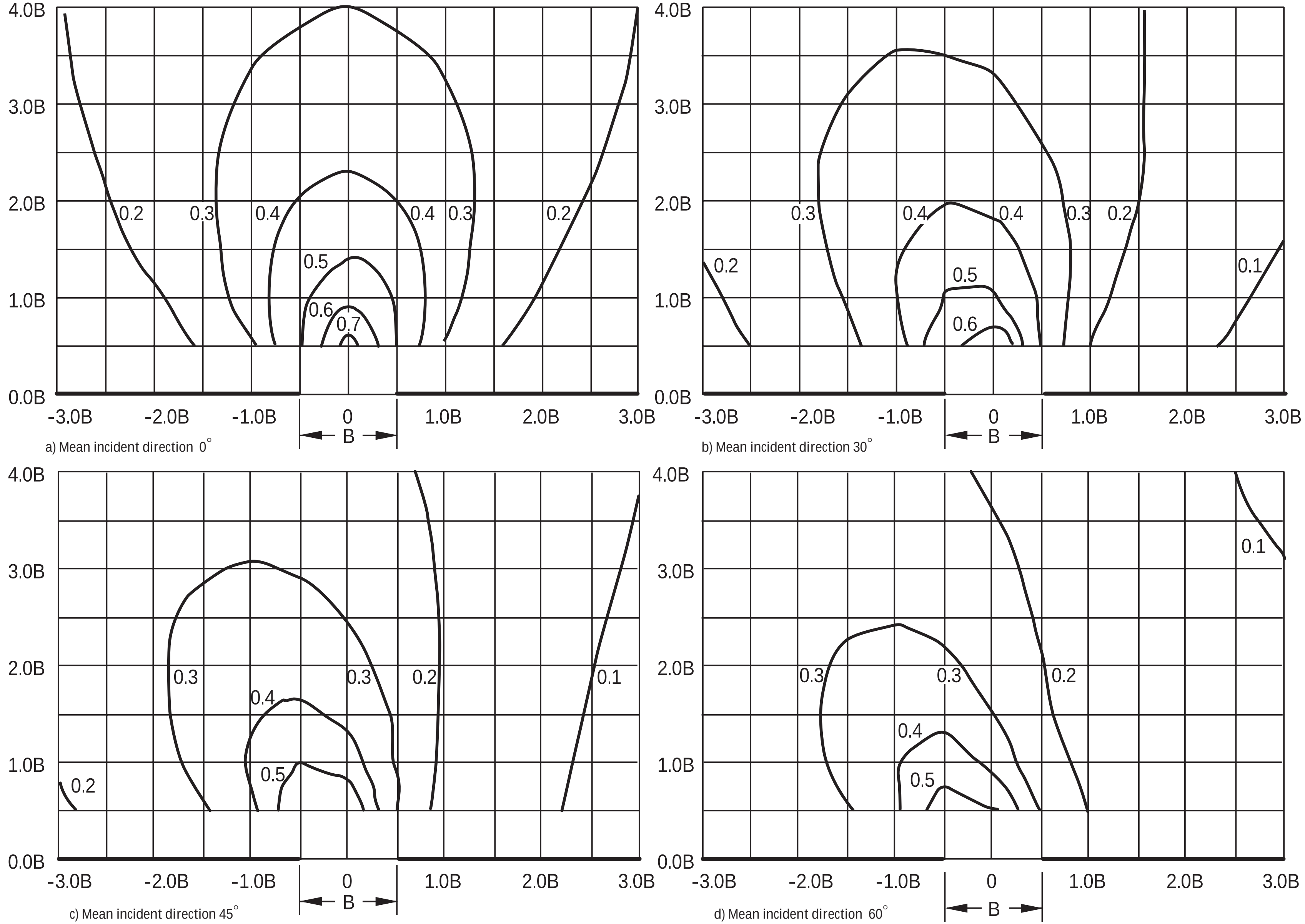

Figure 16a) — Diffraction coefficients for breakwater gap of length B = one wave length, Pierson–Moskowitz spectrum, Cos^2 directional spread. (Acknowledgement: HR Wallingford)

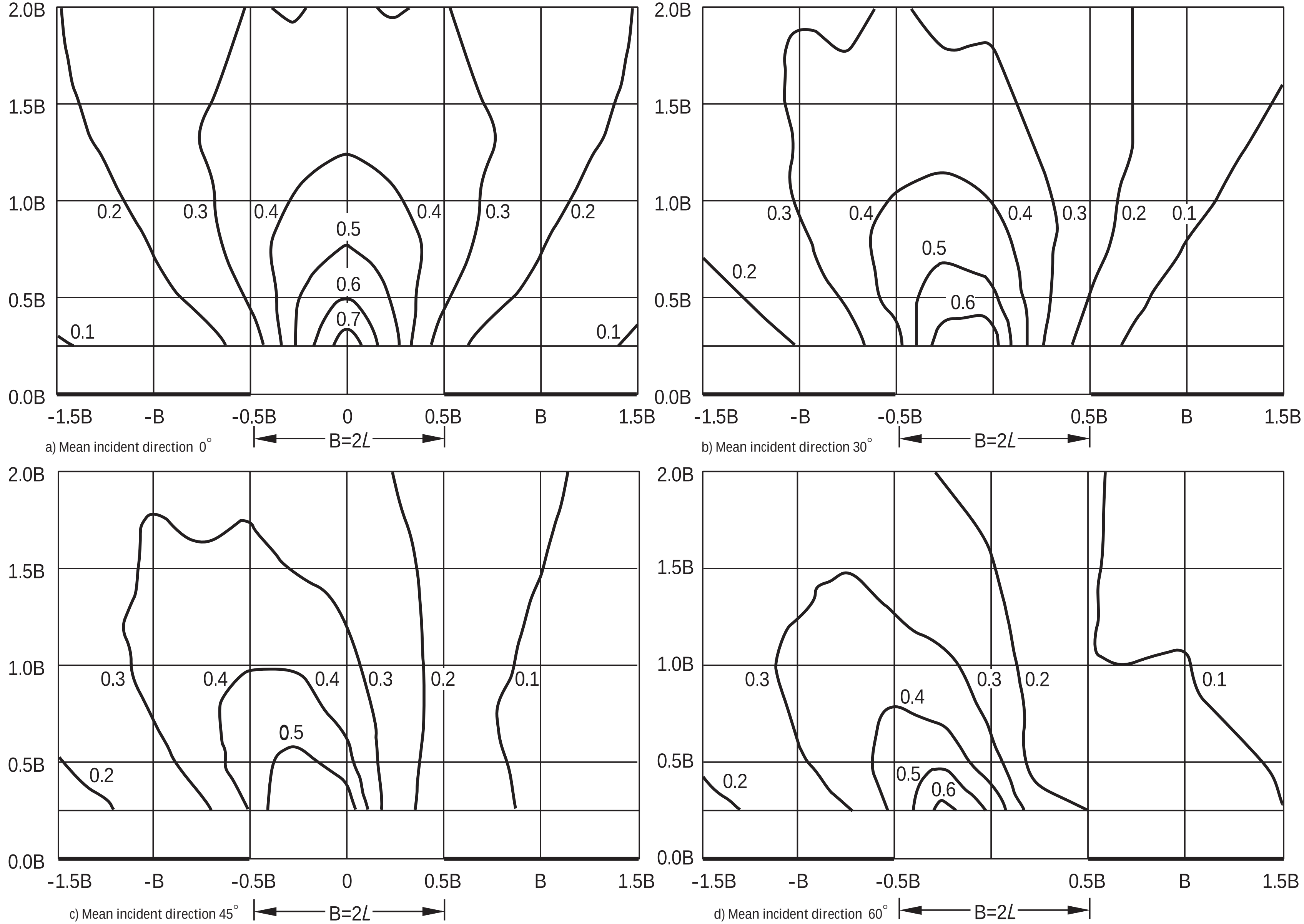

Figure 16b) — Diffraction coefficients for breakwater gap of length B = two wave lengths, Pierson–Moskowitz spectrum, Cos^2 directional spread. (Acknowledgement: HR Wallingford)

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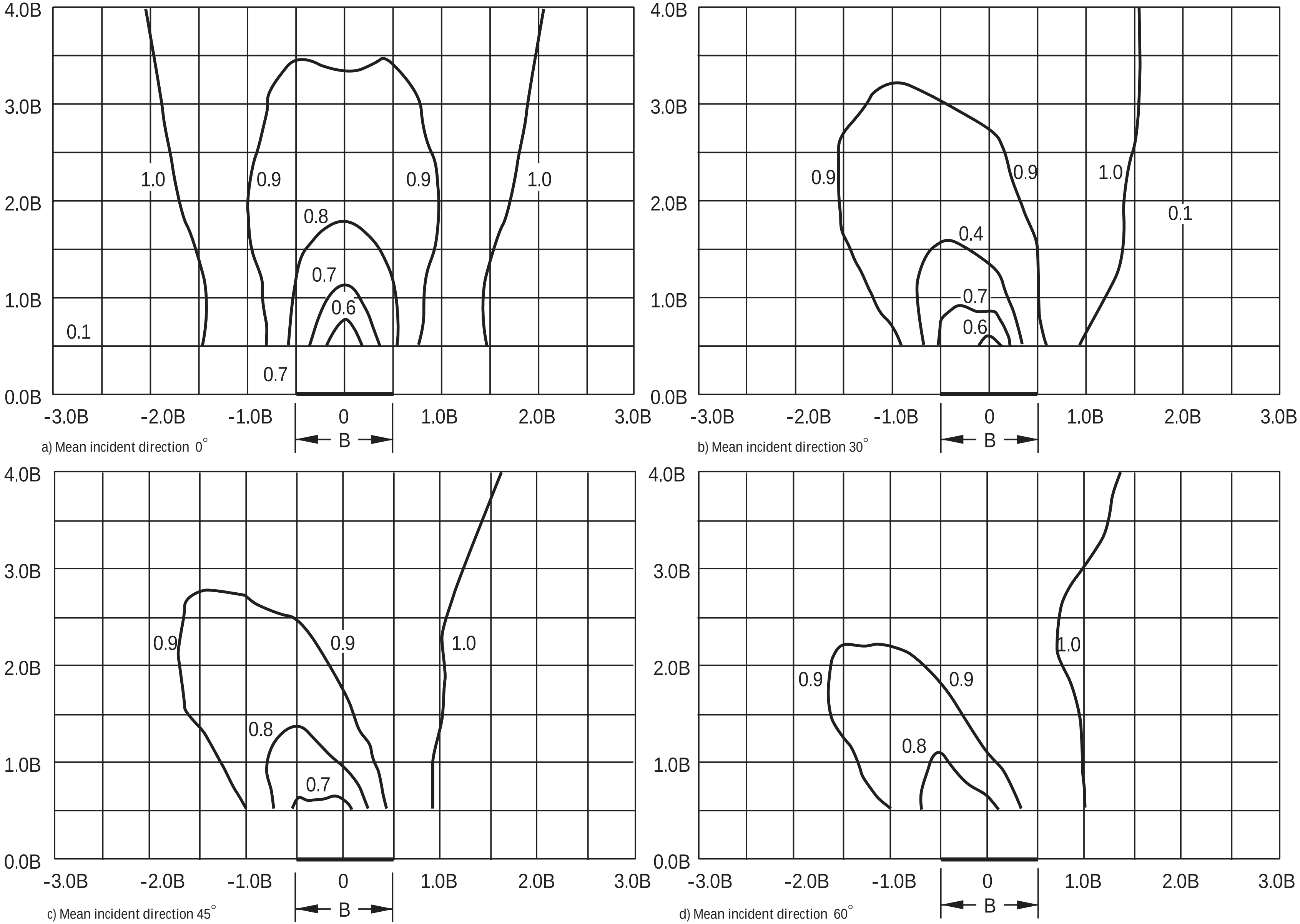

Figure 17a) — Diffraction coefficients for island breakwater of length B = one wave length, Pierson–Moskowitz spectrum, Cos^2 directional spread. (Acknowledgement: HR Wallingford)

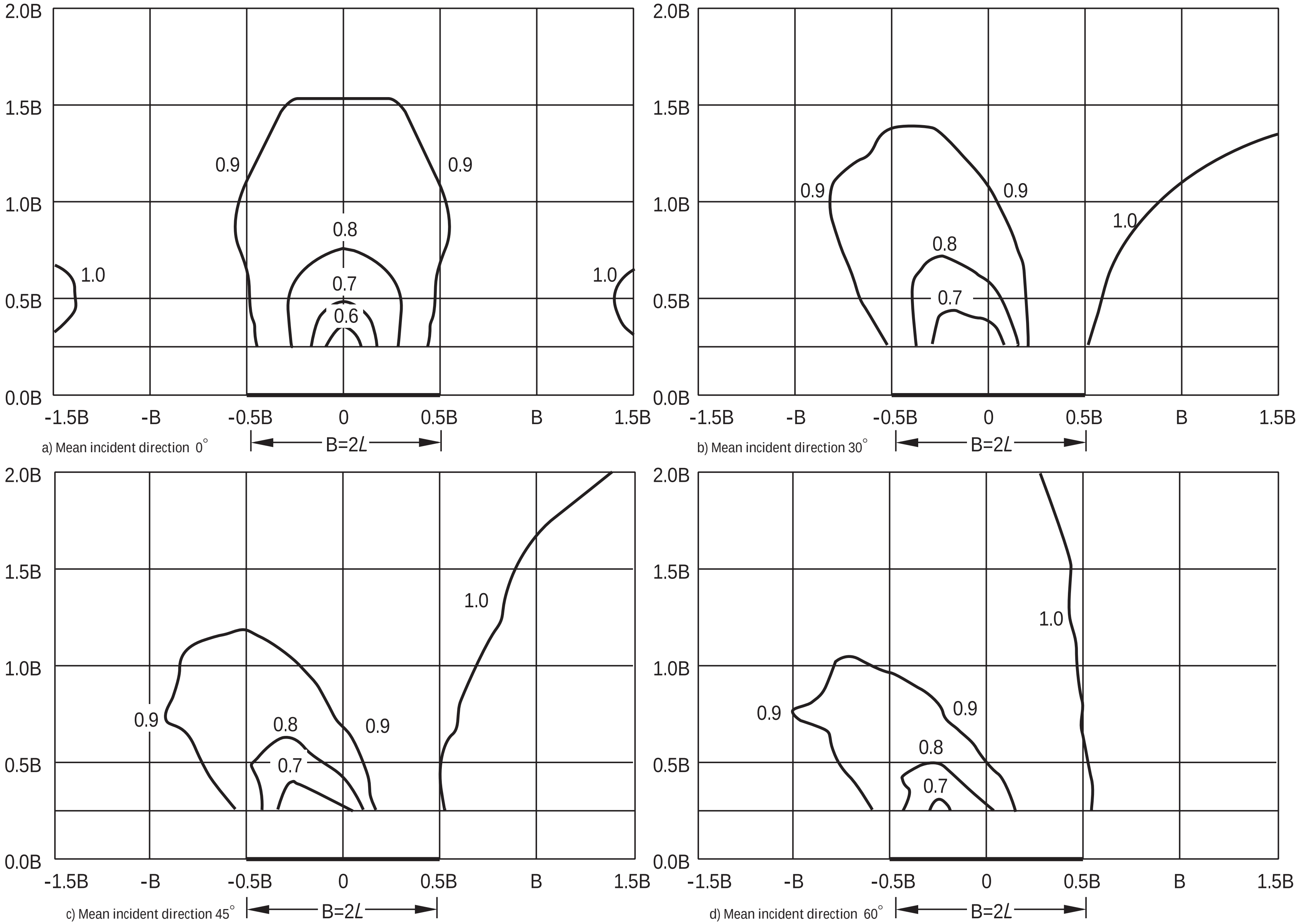

Figure 17b) Diffraction coefficients for island breakwater of length B = two wave lengths, Pierson–Moskowitz spectrum, Cos^2 directional spread. (Acknowledgement: HR Wallingford)

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29.2.3 ***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 difference coefficient shown in Figure 16a) 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, increases as the wave height increases. The diffraction pattern, shown in Figure 16a), leads to a wave set-up that decreases along the harbour boundary in the direction of the shelter of the breakwater. This then generates a flow of water towards the sheltered side, inducing a return current, which travels 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 are modified.

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

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

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;
- $I_D(\theta)$ is the intensity factor [Figures 16a) and 16b)] 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 wave length 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 previously are approximate and so 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 wave lengths reinforce and amplify the incident wave pattern.

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. This is sometimes called the sloshing mode. A range of increasingly shorter period resonances can occur but they are difficult to describe for typical harbours because of their complex shapes.

It should be noted that resonant wave lengths are fixed by the dimensions of the harbour, but the resonant wave period varies with the state of the tide.

For typical large ship harbours, the longest resonant wave lengths 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 **24**). Resonant modes at storm or swell wave periods are normally of less importance in large harbours but they can occur in smaller harbours such as those used by fishing boats and pleasure craft.

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Resonance problems are significantly worsened when the inner walls of a harbour are highly reflective. Care should be taken when developing a harbour area, if this involves replacing beaches or rubble slopes with vertical walls.

29.5 Physical models

At present, the most reliable method of evaluating harbour designs is by the use of physical models using irregular waves.

The physical model has the advantage over mathematical models because it is able to determine harbour response in the presence of the combined effects of wave diffraction, refraction and reflection. In addition, because it is a direct attempt to reproduce reality on a smaller scale, it automatically describes various secondary physical effects, which can only be added to the indirect mathematical 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 carried out in laboratories with the appropriate specialized facilities.

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 model study is needed. This is because the necessary information for such an investigation might require the installation of a wave recorder to obtain at least one year's wave data from the site (see clause **26**) and might 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 normally 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, normally 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 number 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.

Because 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 in the ratio of $1:N_s$ the following scale factors apply:

a) acceleration 1:1;

b) length $1:N_s$;

c) time $1:N_s^{1/2}$;

d) velocity $1:N_s^{1/2}$;

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, although, with the vertical scale larger than the horizontal, the main effects of long-wave behaviour will be well represented, provided that the horizontal currents, which are the dominant feature of very long waves of a minute or longer period, are correctly modelled. 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 Mathematical models

29.6.1 ***General***

The present generation of mathematical 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 that 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 that 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. They will also give an accurate prediction of the shape of the mode that the program represents, by indicating those areas that have vertical movement of the water surface and those that 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 that 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 wave lengths, such as those associated with the pumping or Helmholtz mode in which large oscillatory flows are produced at the harbour entrance. Another dissipation mechanism that 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 available that 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 that cover the harbour area. Within this area the water depth has to vary sufficiently slowly over distances equal to the deep water wave length. The further assumption is usually made that the seabed 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 small vessels

30.1 General

Guidance is given in clause **30** on the maximum wave heights that can be tolerated for boats moored inside marinas and fishing harbours and for barges moored alongside larger vessels. More detailed guidance can be found in [26]. Because the opinion of harbour masters can be expected to vary, the quoted figures should be taken only as general guidelines. Wave heights larger than those given in Table 1 might be tolerable in situations where careful attention is paid to moorings and where boats are not moored in beam seas. They might also be tolerable if boats are moored, or if collisions between the boats and piers or other boats are prevented between individual finger piers.

Table 1 — Acceptable wave heights in marinas and fishing harbours

	H_S (m)
Marinas	
— Moored at jetties or quay walls	0.15
— Swinging moorings	0.60
Fishing harbours	
— Moored at jetties or quay walls (boats up to 30 m length)	0.40

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

Table 1 gives guidance on the maximum wave height that is normally considered to be acceptable in marinas. One of the major limitations 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 is 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. It is frequently necessary to provide inner harbours or basins where pleasure craft can be accommodated safely.

30.3 Fishing harbours

Because fishing craft are normally larger and more strongly built than pleasure craft the acceptable wave height shown in Table 1 can be greater than for pleasure craft. As in the case of pleasure craft, inner harbours or basins are frequently provided for accommodating fishing boats safely.

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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, depend on the nature of the cargo, the method of cargo handling and the sort of risk considered acceptable at the particular site. It is unlikely that the maximum acceptable wave height will exceed 2m, making the maximum acceptable significant wave height about 1 m, provided that such wave conditions are 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 clause **31** on the response to wave actions of larger vessels moored inside harbours or at fixed 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, because 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 (added mass). Oscillation also produces waves that propagate away from the body carrying energy with them that 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. For vertical motions, these can be within the range of swell and storm waves. 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 that act on a vessel to cause oscillation can be divided into two types. The first type is linear wave forces of the same period as the waves and can be obtained by integrating the fluctuating water pressure over the submerged area of the hull. Because the vessel usually alters 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. Non-linear moorings are also capable of exciting the natural periods of horizontal oscillation. The strongest non-linearity in moorings arises because the fenders are usually 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 that is nearest to the natural period of this motion. This type of vessel response can be avoided by making the fenders as soft as the mooring lines. It does not necessarily follow, though, 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 is non-linear and occurs as a consequence of the irregular nature of the sea surface. Because 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. Because 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 that is scattering the waves, a force is produced at the waterline due to scatter of the momentum carried by the waves. Because this momentum is larger in a group of high waves than in a group of small waves, the force produced has the periodicity of wave groups. The fluctuating water pressures produced by set-down beneath wave groups (see **24.3**) 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 because 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 mathematical 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 that can be made to reproduce any non-linear characteristic. Strain gauges are normally used to measure mooring loads. A variety of systems can be used to measure all six degrees of freedom of vessel movement (see Figure 18) but in the process of measurement they should not impose forces on the model vessel that could significantly affect its response.

It is most important that an irregular sea be represented so that non-linear wave forces are 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 are accurately generated in the model. In some models it might be necessary to add an appropriate long period movement to the wave generator, in order to produce a wave climate in the model that agrees with the one measured at the site. This can be explained by the interaction of the 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 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. This is because a physical model study could require a year's wave data from the site to determine the long wave climate, followed by 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 that can have a considerable influence on the wave climate. Currents flowing against the waves tend to make the waves steeper 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 be represented in physical models by the use of a bank of fans.

After a model study the final conclusion often takes the form of an estimate of the number of days in the year when a given berth is untenable with regard to the loading and unloading of cargo. In severe cases, an estimate of the number of days when ships cannot moor at the berth is also given. 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 an overall stiffness of the moorings to cover both conditions. This system would allow movement at the swell wave period, but also be 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 that 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 is amplified if a natural period of oscillation of the basin coincides with a wave period.

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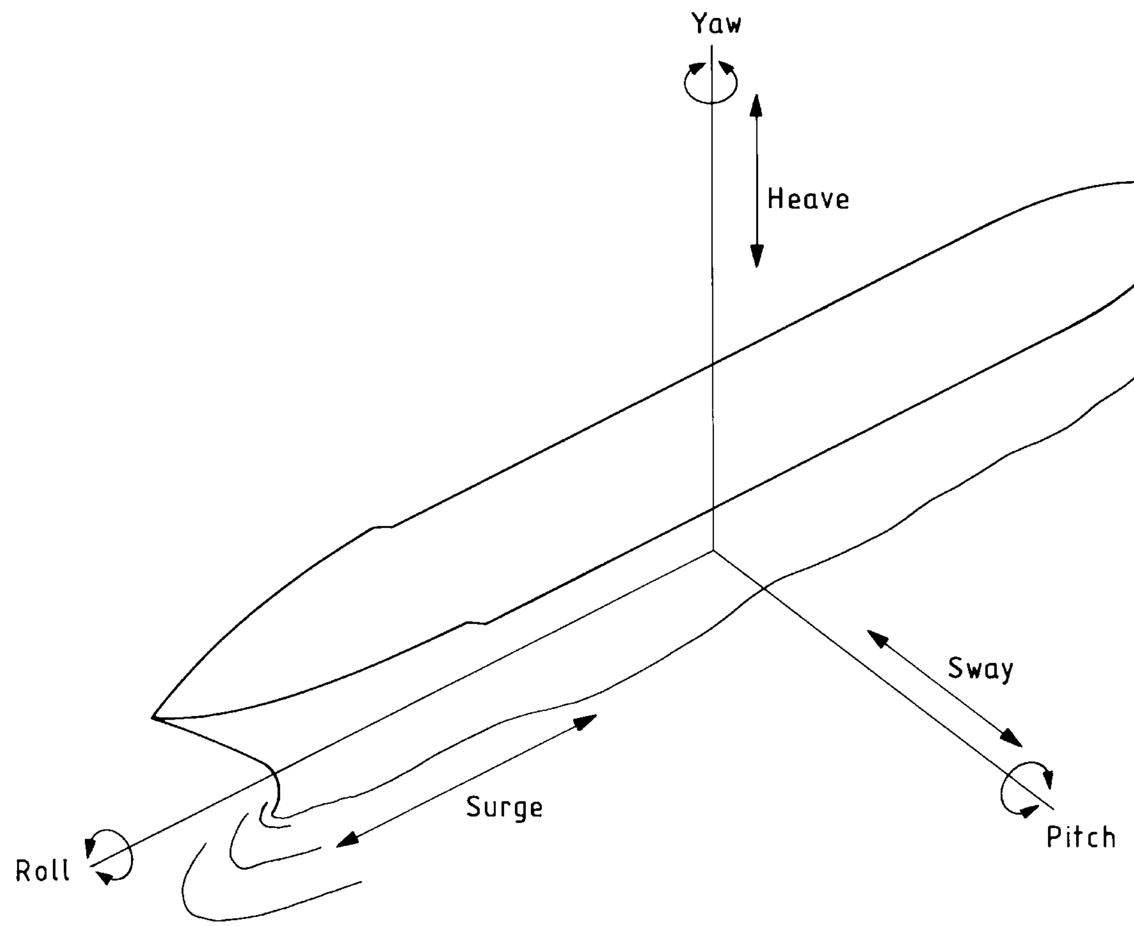

Figure 18 — The six degrees of freedom of vessel movement

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 18) 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 has to be taken into account. Analytical methods now exist that 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. These depend both on the primary wave system with diffraction effects and on 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 mathematical methods offer the best way of dealing with the general problem.

31.3.4 ***Mathematical 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 can 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 can then be calculated. This is done by modifying the amount of wave energy present in the wave spectrum for that period by a factor representing the pitch response of the vessel to a uniform wave of the 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.

The mathematical method described so far can only deal with linear mooring systems and is inadequate for most situations. One method of solving the problem mathematically 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.

Mathematical models are available to deal with the complex calculations needed. These are much less costly and offer a quicker solution than physical models, although the reliability of the answers is uncertain. The technique is best suited to situations where data from an existing operation is available to verify the model, as for use in preliminary studies of layout and location.

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 as follows, based on these limiting criteria:

a) safety limits, which if exceeded, could result in damage to the ship, other vessels and the port installations;

b) limits imposed by cargo handling, which, if exceeded, might result in the inefficient loading and unloading of cargo or it being halted altogether;

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

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 clearly depends on the softness or stiffness of the mooring system. A soft system allows more movement of a vessel before mooring loads reach their safety limit, whereas a stiff system does not.

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.

Another approach is based on limiting the dynamic impact of a moored ship against the quay. In this case the limiting condition for damage to ship and/or quay is the kinetic energy of the ship. This has been studied by the Nordic countries [26] for moored fishing vessels up to 3 000 grt and the recommended criteria are given in Table 2. The same reference [26] states that the safety criteria apply to ships up to 8 000 grt, such as coasters, freighters, ferries and ro-ro vessels as well as fishing vessels.

Table 2 — Guidance on maximum velocity criteria for safe mooring conditions

Ship size DWT	**Surge** m/s	**Sway** m/s	**Heave** m/s	**Yaw** degrees/s	**Pitch** degrees/s	**Roll** degrees/s
1 000	0.6	0.6	—	2.0	—	2.0
2 000	0.4	0.4	—	1.5	—	1.5
8 000	0.3	0.3	—	1.0	—	1.0
NOTE. These criteria are applicable to fishing vessels, coasters, freighters, ferries and ro-ro vessels.						

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31.4.3 ***Limits imposed by cargo handling***

The amount of ship movement allowed under this criterion depends on the type of vessel.

Table 3, which is taken from [26], gives the recommended criteria for safe working of cargoes for a wide range of vessel types.

It should be noted that the larger the vessel the less it will respond in surge, sway and yaw to the primary wave system. For example, the horizontal movement of large container ships is relatively slow, probably at periods longer than 20 s. The operators of container cranes might be able to cope better with these slow movements than with oscillations at swell or storm-wave periods.

In the case of tankers and gas carriers, 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.

The degree of acceptable movement depends greatly on the environmental conditions at the berth and requires the optimization of berthing forces versus ship movements. Discussion with the operator and with the designer of the cargo transfer equipment at an early stage is therefore essential.

Although the loading arm is capable of accepting up to ±4 m of surge and 4 m of sway or yaw off the jetty, the emergency shut-down system on an LNG berth, for example, could start working when the relative movement is 0.5 m or less.

In the case of cargo operations that impose no obvious limitations in the link between the vessel and shore, the amount of acceptable movement should be defined by experience.

31.4.4 ***Limits based on experience***

It is always prudent to obtain opinions from the harbour authority and, if possible, from ship operators, as to the amount of movement of moored ships that they consider to be acceptable.

31.5 Downtime

If the data are available the designer should assess the annual downtime for cargo handling that is caused by excessive motions of the moored ship. Acceptable figures for downtime vary according to the type of cargo handled. In the oil and gas industry a limit of 10 % on an annual basis is the norm.

Table 3 — Guidance on maximum motion criteria for safe working conditions

Ship type	Cargo handling equipment	Type of motion					
		Surge[a] m	Sway[a] m	Heave[a] m	Yaw[a] degrees	Pitch[a] degrees	Roll[a] degrees
Fishing vessels	Elevator crane	0.15	0.15				
	Lift-on-lift-off	1.0	1.0	0.4	3	3	3
	Suction pump	2.0	1.0				
Freighters, coasters	Ship's gear	1.0	1.2	0.6	1	1	2
	Quay cranes	1.0	1.2	0.8	2	1	3
Ferries, ro-ro	Side ramp[b]	0.6	0.6	0.6	1	1	2
	Bow/stern ramp	0.8	0.6	0.8	1	1	4
	Linkspan	0.4	0.6	0.8	3	2	4
	Rail ramp	0.1	0.1	0.4	—	1	1
General cargo	—	2.0	1.5	1.0	3	2	5
Container vessels	100 % efficiency	1.0	0.6	0.8	1	1	3
	50 % efficiency	2.0	1.2	1.2	1.5	2	6
Bulk carriers	Cranes	2.0	1.0	1.0	2	2	6
	Elevator/bucket-wheel	1.0	0.5	1.0	2	2	2
	Conveyor belt	5.0	2.5		3		
Oil tankers	Loading arms	0.5 − 2.0[c]	0.5 − 2.0[c]				
Gas tankers	Loading arms	0.5	0.5[c]				

[a] Motions refer to peak-peak values (except for sway: zero-peak).

[b] Ramps equipped with rollers.

[c] Refer to **31.4.3** for comment.

Section 5. Loads, movements and vibrations

32 General

32.1 Basic loads

In addition to dead loads and soil pressures, the other forces that can 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, 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 can 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 provisions should be made.

The loading design criteria adopted should be clearly stated and recorded. If it is proposed to change the operational use, or to introduce new heavy equipment or storage systems, a check should 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 can 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 is magnified relative to that predicted by quasi-static analysis.

Typical frequencies of cyclic loads in the maritime environment are shown in Table 4 as a preliminary guide.

Dynamic effects are not usually 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 can 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 are 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 are 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 **31**).

Table 4 — Typical frequencies of environmental forces

Environmental force	Typical frequency Hz	Period 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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32.4 Fatigue

Structural members subjected to fluctuating loads can suffer from fatigue failure. For maritime structures, problems due to fatigue are most likely to arise in steel members subjected to wave loading. (See **39.2** for fatigue analysis.)

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 groundwater conditions;

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

34 Winds

For maritime structures the 3 s gust speed is only used for the design of individual members.

In the case of ships it is recommended that the 1 min mean wind speed should be used for the design of moorings, because of the time needed for full line loads to develop, taking into account the inertia of the vessel. Design wind speed can also be limited by operational practices (see section **2**). The value of the 1 min wind speed can be estimated from the following relationship:

1 min mean speed = 0.85 × 3 s gust

In cases where wind loading is critical, values of aerodynamic force coefficients might 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 BS 6399 should be used.

In the recent past, loading from floating sea ice has not been a problem around the British Isles and needs 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 that do warrant general consideration are superstructure icing and expansion of small pockets of trapped ice. Guidance on superstructure icing is given in CP3:ChV-2:1972, Appendix F.

NOTE Specialist advice on particular aspects of ice and icing effects can 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 5.

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 normally have a return period of not less than 50 years for permanent works.

Extreme water levels, which can be caused by a combination of astronomical tides, positive or negative 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) lines of action of mooring and berthing forces, forces from other floating objects and wave forces.

Table 5 — Effective temperature range for maritime structure decks in British coastal waters

Type of deck	Maximum temperature °C	Minimum temperature °C
Steel deck on steel beams	49	−16
Concrete deck on steel beams	41	−14
Concrete deck on concrete beams	35	−9

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 usually preferable to represent the buoyancy and gravitational loads as separate systems.

Reduced safety factors are appropriate 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 assessing the relationship between astronomical data and surge tide levels, reference can be made to the Proudman Oceanographic Laboratory, Bidston, Birkenhead, England.

38 Currents

38.1 General

For design purposes the current speed should be the maximum value expected at the site during the design life of the structure. It should be established primarily in relation to the purpose of the structure and the accepted probability of occurrence (see **21.4**), but should normally 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 can 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.

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, can be calculated from the expression:

$$F_D = \frac{1}{2}(C_D \rho V^2 A_n)$$

where

F_D is the steady drag force in kilonewtons (kN);

C_D is the dimensionless time-averaged drag force coefficient;

ρ is the water density in tonnes per cubic metre (t/m^3);

V is the incident current velocity in metres per second (m/s);

A_n is the area normal to flow in square metres (m^2).

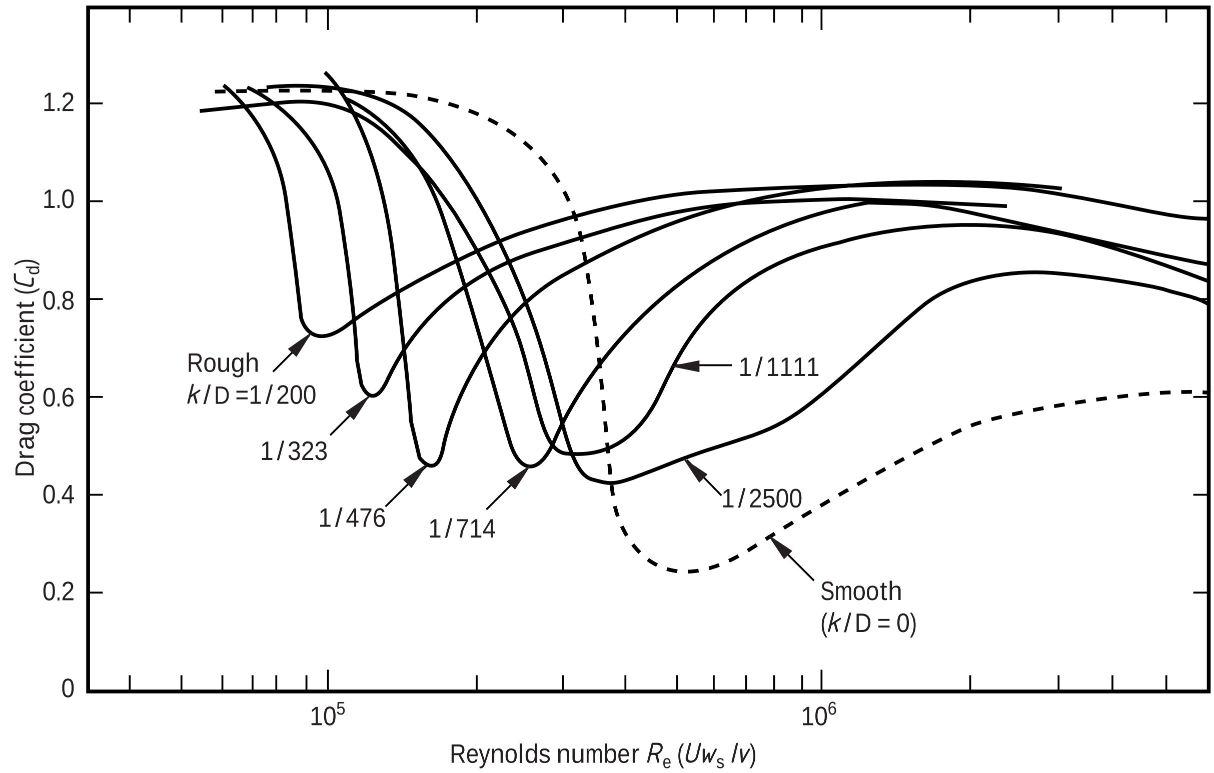

Figure 19 — Drag coefficient values for circular cylinders

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The values used for C_D and A_n in the previous expression should be determined taking due account of the effect of marine growth on cross-sectional dimensions. Values of C_D are discussed as follows 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 structural member is gently tapered, the total force and the line of action can be determined by integration. Where the structure is fully submerged and end effects can be significant, or where it is floating or of significantly non-uniform shape, it might be necessary to measure the drag force 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 result used to calculate the drag force from the formula given previously. Inertial forces might 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 previously are given in Figure 19 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 usually independent of Reynolds number, but depend on the angle of incidence. Values are given in Table 6 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. When the cylinder is in any mode in which it is free to oscillate, the amplitude of the fluctuating force increases as its frequency approaches the natural frequency of the cylinder or that of the whole structure. This is done by a feedback 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 might 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 previous expression the values for f_N and W_s should be derived taking due account of the effect of marine growth. Because the critical condition for flow-induced oscillation usually occurs during construction, however, this is likely to be negligible.

Guidance on the calculation of the natural frequencies of structural members is given in **47.2.2**.

The most common type of structure has 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 occurring in this structure are given in Figure 20. The curves are conservative in that they assume water surface is at the top of the pile. For piles that are similar, but which have a different fixity and/or different motion conditions, the critical velocities can be obtained by applying the modification factors given in Table 7 to the values obtained from Figure 20.

Table 6 — Modification factors for critical flow velocity

Motion	Pinned to fixed bottom	Cantilever	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 critical. This is because vortex shedding is a resonant phenomenon, in that 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 on the basis of either:

a) a critical flow speed that is higher than the design current speed; or

b) a mass and damping that are sufficient to prevent significant motion.

The first criterion is satisfied if the current speed is less than $1.2 f_N W_s$. The second criterion is 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:

$$\frac{2\overline{m}\Delta}{\rho W_s^2}$$

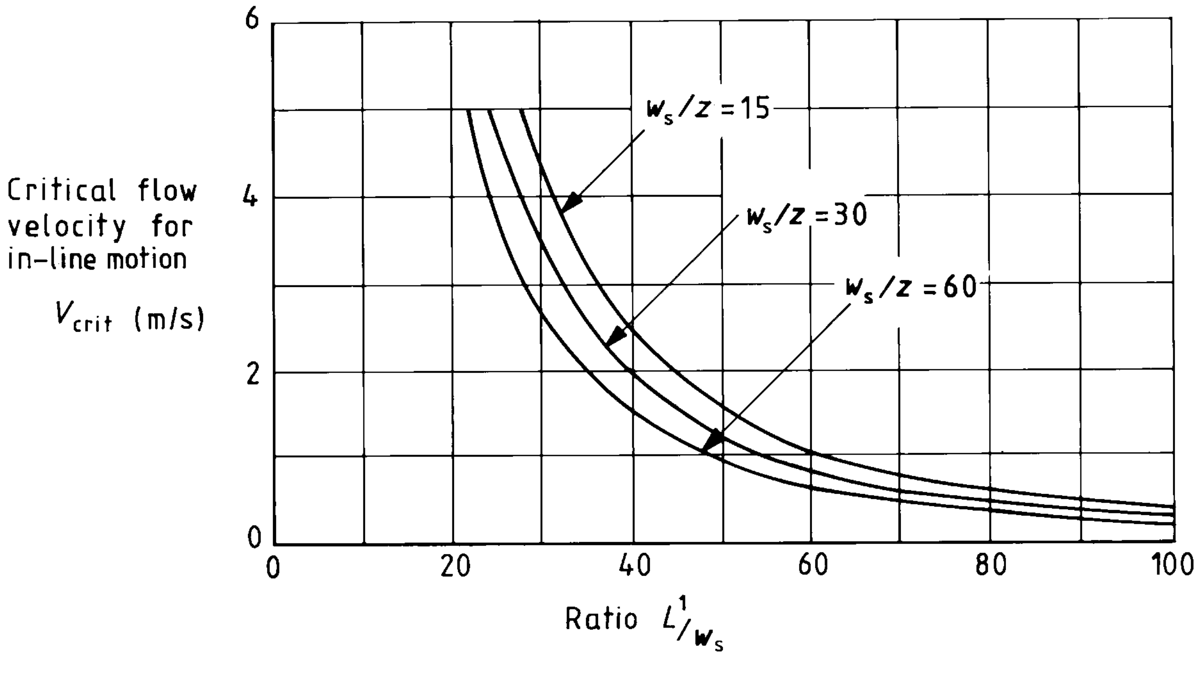
where

- Δ is the logarithmic decrement of structural damping, which can 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:

$$\overline{m} = \frac{\int_0^{L'} m_L (y(x))^2 d_x}{\int_0^{l'} (y(x))^2 d_x}$$

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.



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 20 — Critical flow velocity for circular piles for in-line oscillations

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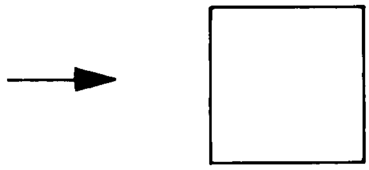
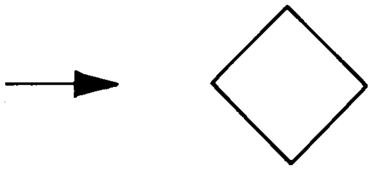
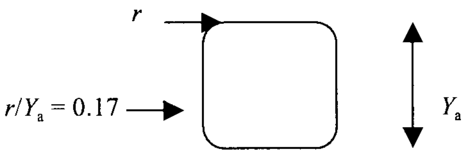
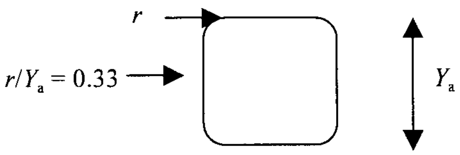
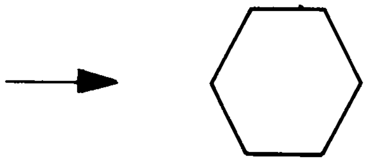
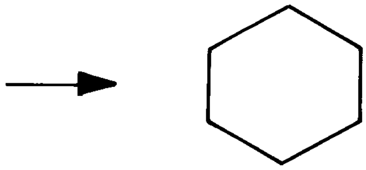
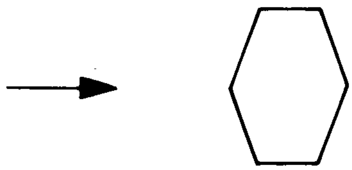
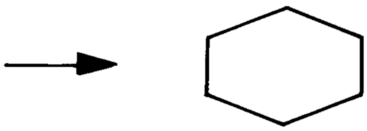
Table 7 — Drag and inertia coefficients for common structural forms

Cross-section type	Attitude to flow or wave direction	Drag coefficient C_D	Inertia coefficient C_I
Circle	Any	See Figure 19	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	*
Dodecagon	Any	1.1	*
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.

38.3.2 ***Non-circular sections***

Non-circular cross-sections are 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 [16]. Model tests might be necessary 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 can then be determined by using the Palmgren–Miner equation.

$$\text{Factor of safety} = \sum_{I=1}^{n_T} (n_i/N_i)$$

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 purposes 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 that 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 can 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, because of the difficulties of accurately modelling or calculating the shock pressures that 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 wave length, 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 can normally be assumed to be valid, but in shallow water, where the depth to deep water wave length ratio, d/L_0, is less than 0.1, as the wave form starts to deviate significantly from sinusoidal, it might be necessary to use either solitary wave theory or cnoidal wave theory for greater accuracy. Details of these theories are given elsewhere [12, 16].

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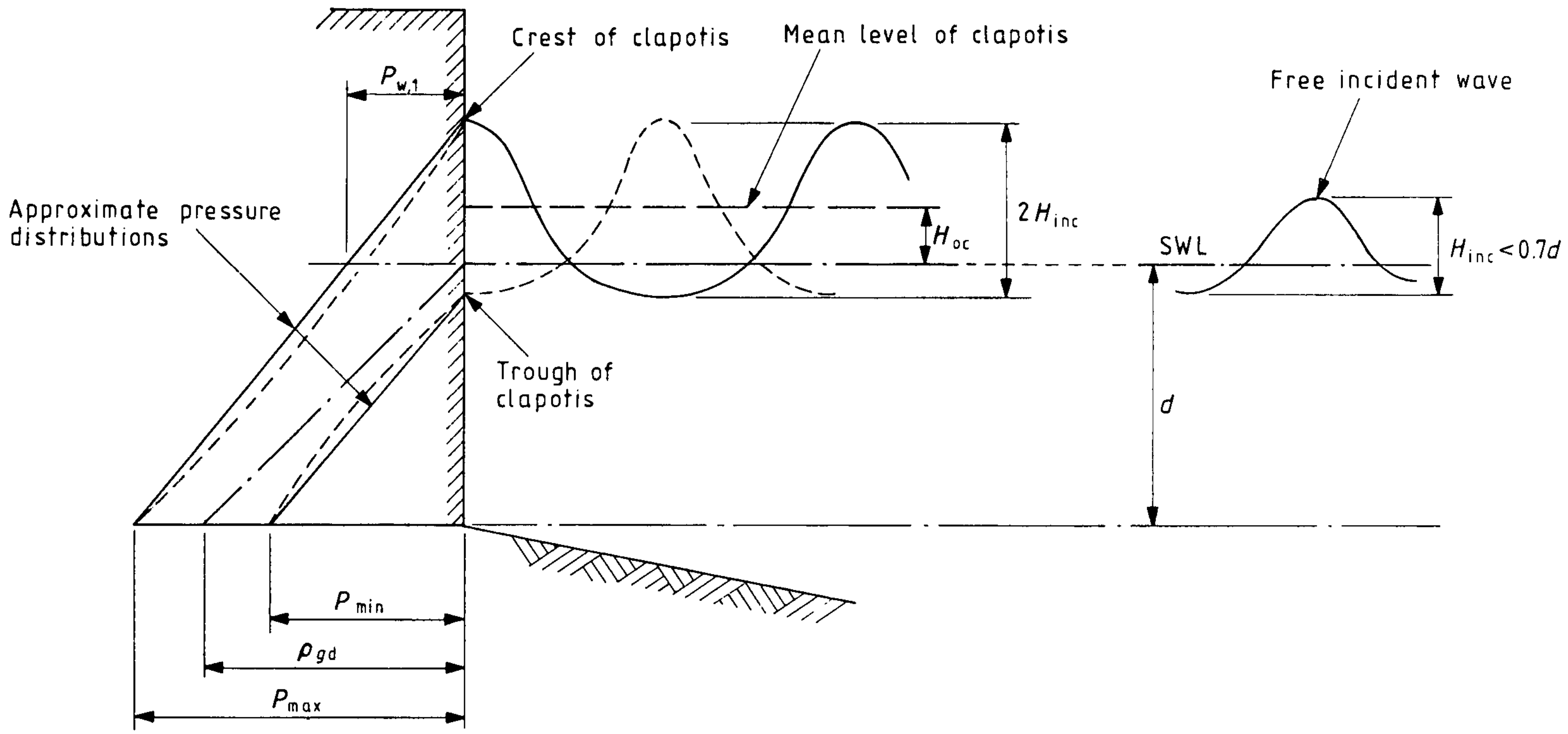

Height of clapotis = 2 times height of free incident wave
Clapotis set-up:

$$H_{0c} = \frac{\pi H_{inc}^{2}}{L} \coth \frac{2\pi d}{L}$$

Maximum pressure on exposed face:

$$P_{max} = \frac{pgd + pg \cdot H_{inc}}{\cosh(2\pi d/L)}$$

Minimum pressure on exposed face:

$$P_{min} = \frac{pgd - pg \cdot H_{inc}}{\cosh(2\pi d/L)}$$

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 21 — Wave pressure distribution at reflective walls for non-breaking waves

39.4.2 ***Reflective conditions***

Waves incident upon an infinitely long vertical surface can 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, as shown in Figure 21. In actual cases, the end result can be a standing wave varying in height along the wall about a mean value of twice the incident wave height. The variation can amount to 20 % for regular waves and be evident for at least two wave lengths along the wall from its end. A similar variation would occur with long crested random waves but the peak variation can be 15 % and the variation would be damped out within one wave length 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 wave length and wave height, waves can break against the wall producing impulsive loading, which can be very large over small surface areas. The average wave pressure on a long structure from breaking waves and from reflected waves can be estimated as follows.

The vertical distribution of wave pressure can be assumed to take the form shown in Figure 22.

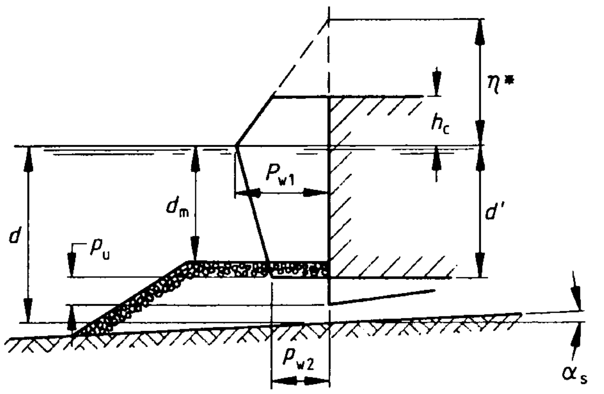

Figure 22 — Estimation of alpha values for calculation of wave pressure — Vertical distribution of wave pressure

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 $HD = 1.8\, H_s$. Within the surf zone HD 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 seabed at the wall face. HD and H_s are obtained from Figures 3b) to 3f) 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 is zero. When the angle exceeds 15°:

β = actual angle of 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:

$$\alpha_2 = \frac{1}{3}\left(1 - \frac{d_m}{d_b}\right)\left(\frac{H_D}{d_m}\right)^2$$

or:

$$\alpha_2 = \frac{2d_m}{H_D}$$

$$\alpha_2 = 1 - \frac{d_1}{d}\left(1 - \frac{1}{\cosh 2\pi\frac{d}{L}}\right)$$

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 can be estimated from Figures 23, 24 and 25.

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:

If:

$$h_c \geq \eta^* F_w = \left(\frac{P_{w1} + P_{w2}}{2}\right)d' + \eta^*\frac{P_{w1}}{2}$$

If:

$$h_c < \eta^* F_w = \left(\frac{P_{w1} + P_{w2}}{2}\right)d' + \frac{P_{w1}h_c}{2}\left(2 - \frac{h_c}{\eta^*}\right)$$

where

h_c is the crest height as shown in Figure 21.

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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 can then be determined according to the theory of Sainflou, as given in Figure 21. The figure of 0.7 might not be correct for steep wave conditions, steeply sloping seabeds and composite structures.

Local wave pressures arising when a wave breaks against a flat surface can be much larger than the average values calculated previously. Analysis of prototype observations has led to the expression $P = \lambda \rho T v_c^2$ being proposed for local pressures. The values of λ depend 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 kilograms per cubic metre (kg/m^3), the period in seconds (s) and the phase velocity in metres per second (m/s), the calculated pressure is in newtons per square metre (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 [16] 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 can 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.

A method for the estimation of drag forces on large bodies, such as a pontoon moored in a tidal stream, is given in BS 6349-6:1989.

As a conservative approximation, the wave force can 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:

$$w_p = \frac{H}{\tanh\frac{2\pi d}{L}}$$

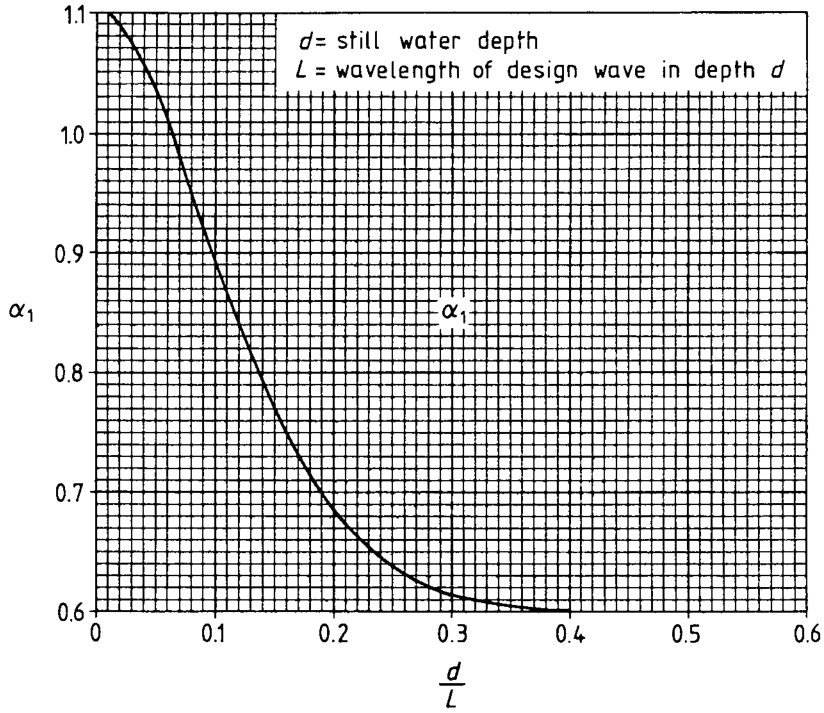

Figure 23 — Estimation of alpha values for calculation of wave pressure — Coefficient of wave pressure at surface dependent upon wave period

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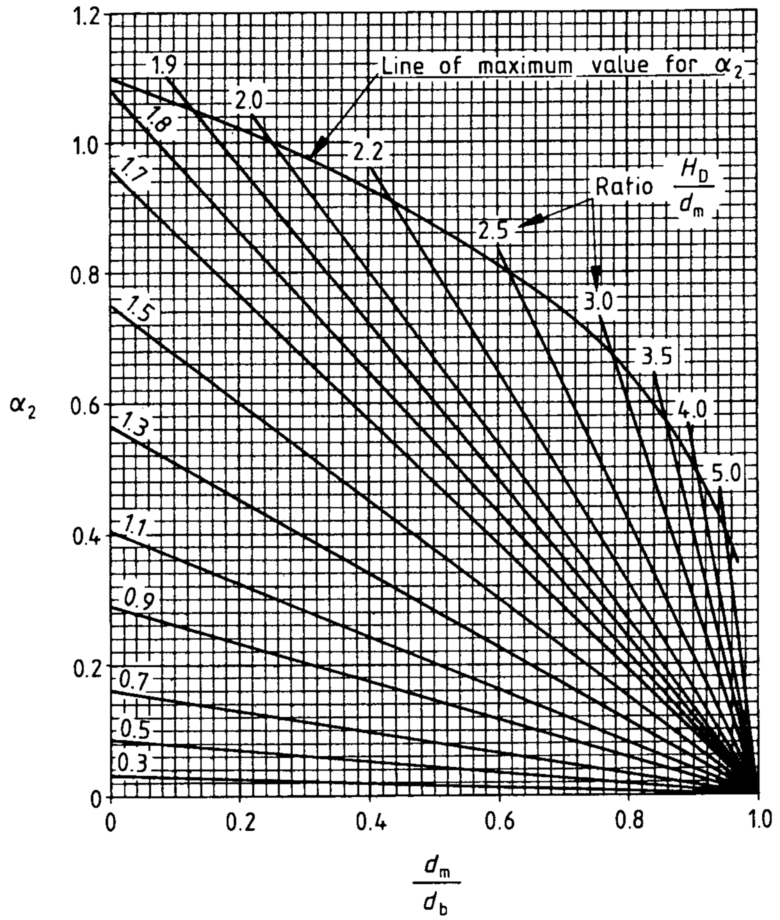

Figure 24 — Estimation of alpha values for calculation of wave pressure — Coefficient of wave pressure at surface dependent upon shoaling

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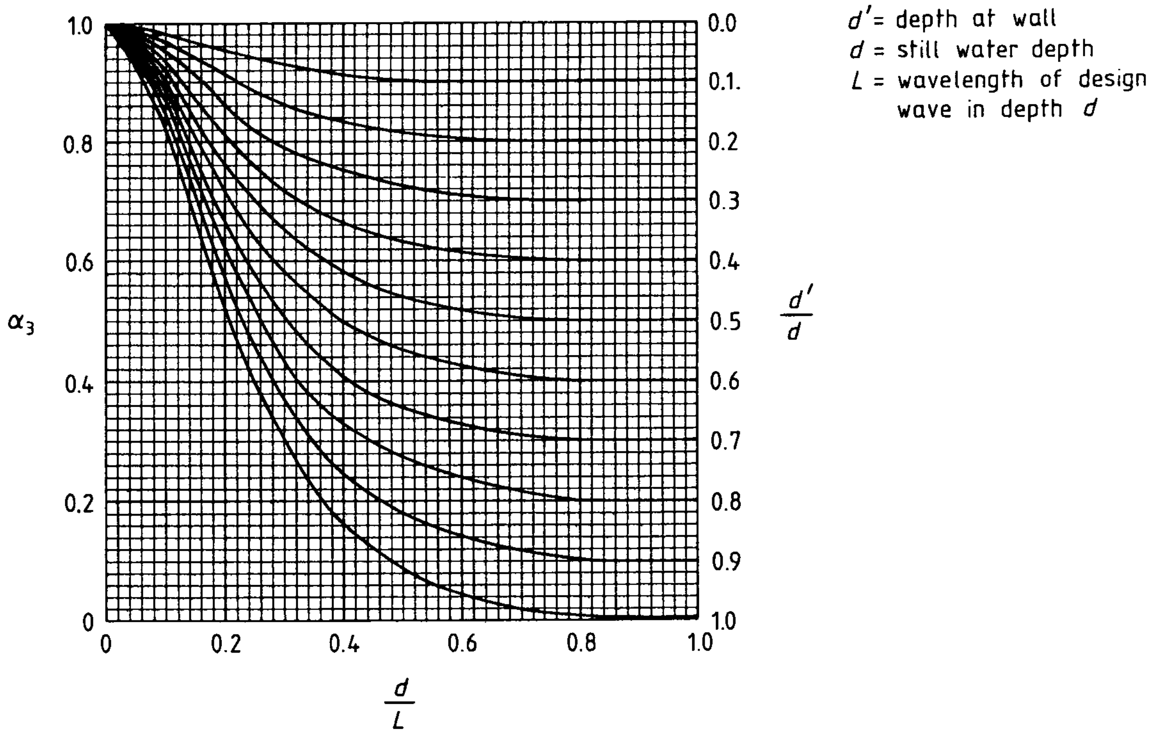

Figure 25 — Estimation of alpha values for calculation of wave pressure — Ratio between wave pressures at surface and at depth *d'*

where

- H is the wave height;
- d is the still water depth at the structure;
- L is the wave length.

The Morison equation can be expressed as follows:

$F_W = F_D + F_1$

where

- F_W is the total wave force normal to the axis of the member in kilonewtons (kN);
- F_D is the drag force component in kilonewtons (kN) given by:

$$F_D = \int_0^{L_s} (½C_D \rho W_s |U| U) dL_s;$$

- F_I is the inertia force component in kilonewtons (kN) given by:

$$F_I = \int_0^{L_s} (C_i \rho A \dot{U})\, dL_s$$

where

- L_s is the submerged length of the member, of which dL_s is an elemental length in metres (m);
- C_D is the drag coefficient;
- C_I is the inertia coefficient;
- $\dot{U}$ is the instantaneous velocity in metres per second (m/s);
- U is the instantaneous acceleration of the water particles, both measured normal to the axis of the member at the elemental length dL_s;
- ρ is the density of the water in tonnes per cubic metre (t/m^3);
- A is the cross-sectional area of the member in square metres (m^2);

Where the member extends through the wave surface the integration limit L_s is governed by the instantaneous water level, η.

Suggested values of C_D are given for circular cylinders in Figure 19 and of C_I and C_D for some standard structural shapes in Table 6.

For closely grouped members, C_I can increase and should be determined from physical model tests. For floating structures, a modified form of Morison's equation should be used [16].

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 $\dot{U}$.

U and $\dot{U}$ are normally 90° out of phase for pure waves, but this changes 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 acceleration are derived from linear theory.

$$\eta = \frac{H}{2} \cos \left\{ 2\pi \left(\frac{x}{L} - \frac{t}{T} \right) \right\}$$

$$u = \frac{\pi H}{T} \frac{\cosh\{2\pi(y + d)/L\}}{\sinh(2\pi d/L)} \cos \left\{ 2\pi \left(\frac{x}{L} - \frac{t}{T} \right) \right\}$$

$$u = \frac{\pi H}{T} \frac{\cosh\{2\pi(y + d)/L\}}{\sinh(2\pi d/L)} \cos \left\{ 2\pi \left(\frac{x}{L} - \frac{t}{T} \right) \right\}$$

$$v = \frac{\pi H}{T} \frac{\sinh\{2\pi(y + d)/L\}}{\sinh(2\pi d/L)} \sin \left\{ 2\pi \left(\frac{x}{L} - \frac{t}{T} \right) \right\}$$

$$\dot{u} = \frac{2\pi^2 H}{T^2} \frac{\cosh\{2\pi(y + d)/L\}}{\sinh(2\pi d/L)} \sin \left\{ 2\pi \left(\frac{x}{L} - \frac{t}{T} \right) \right\}$$

$$\dot{v} = \frac{2\pi^2 H}{T^2} \frac{\sinh\{2\pi(y + d)/L\}}{\sinh(2\pi d/L)} \cos \left\{ 2\pi \left(\frac{x}{L} - \frac{t}{T} \right) \right\}$$

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 wave length;
- 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 can be particularly significant.

The vertical slam force for a cylindrical member can be determined from:

$$F_s = \frac{1}{2} C_s \rho V_\eta^{\,2} / W_s$$

where

- F_s is the vertical slam force in kilonewtons (kN);
- C_s is the slamming coefficient;
- ρ is the water density in tonnes per cubic metre (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 metres per second (m/s);
- l is the length of the cylinder in metres (m);
- W_s is the diameter of the cylinder in metres (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 can be obtained from the expression for η given in **39.4.4**.

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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. In the case of sensitive structures seismic effects might need to be considered.

Many countries subject to earthquakes include specific seismic design considerations 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 level through their foundations, base or pile support. The response of a structure to these accelerations depends upon its type, mass and dimensions and the failure modes to which it might be subject. It is therefore important in seismically active areas to select a type of structure that has as little sensitivity to seismic action as can be contrived.

Fine sandy soils are especially vulnerable to liquefaction.

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 are 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 depends 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 are usually resisted by passive soil pressure developed behind the quay wall with little effect on the structure itself and accordingly might require little consideration except to minimize damage to ships.

The displacement of water as the ship nears the wall has a cushioning effect and helps to reduce the velocity of approach.

For other structures the berthing loads are predominantly a design consideration.

41.2 Operational factors

The transverse velocity of approach of a vessel is a major factor in determining the kinetic energy of berthing. Guidance on this, and other operational factors, is given in **19.2**.

It is always possible that catastrophic impacts can occur from ships drifting out of control. The necessity to provide against such impacts, particularly to vulnerable oil or gas pipelines, should be considered in relation to the consequences of such impacts, both with respect to the risk of loss of life or environmental damage as well as the cost of repairs.

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 impact loading from a moving vessel into reactions that both vessel and structure can safely sustain. Fendering systems should be capable of sustaining both the resulting loads perpendicular to the fender faces and any component parallel to the berthing face, both horizontally and vertically, which can result from ship movements (see also **47.2.3** and **47.3.4**).

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.

41.4 Design of fendering

Reference should be made to BS 6349-4:1994 for the design of fendering.

41.5 Assessment of berthing energy

Guidance on the assessment of berthing energy is given in BS 6349-4:1994.

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 either the 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**.

In harbours and sheltered anchorages mooring loads principally result from turbulent winds and currents. Most of this turbulence is of shorter period than the resonant periods of large moored vessels, so it does not excite significant dynamic response in such vessels. Small vessels, however, can respond significantly differently, but the loads generated are not usually critical in terms of 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 clause **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

In the absence of a special assessment of mooring loads, bollards for vessels up to 20 000 t loaded displacement should be provided along continuous quays at intervals of 15 m to 30 m. The load capacity should be as given in Table 8, which allows for more than one rope to be attached to each bollard.

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. Calculation methods are described in BS 6349-4:1994.

Design wind speeds should be derived as described in clause **34**.

Design current velocities should be derived as described in clause **38**.

Wind and current forces vary considerably both in 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 26.

The method of calculation that 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 can 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 can be evaluated using the expressions given as follows.

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 kilonewtons (kN);
- F_{LW} is the longitudinal wind force in kilonewtons (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 can be taken to vary from 1.309 6 kg/m^3 at 0 °C to 1.170 3 kg/m^3 at 30 °C;
- A_L is the longitudinal projected area of the vessel above the waterline in square metres (m^2);
- V_W is the design wind speed in m/s at a height of 10 m above water level.

Table 8 — Nominal bollard and fairlead loadings for vessels up to 20 000 t displacement

Vessel loaded displacement	Bollard loading kN	Fairlead loading kN
up to 2 000	100	200
up to 10 000	300	500
up to 20 000	600	1 000

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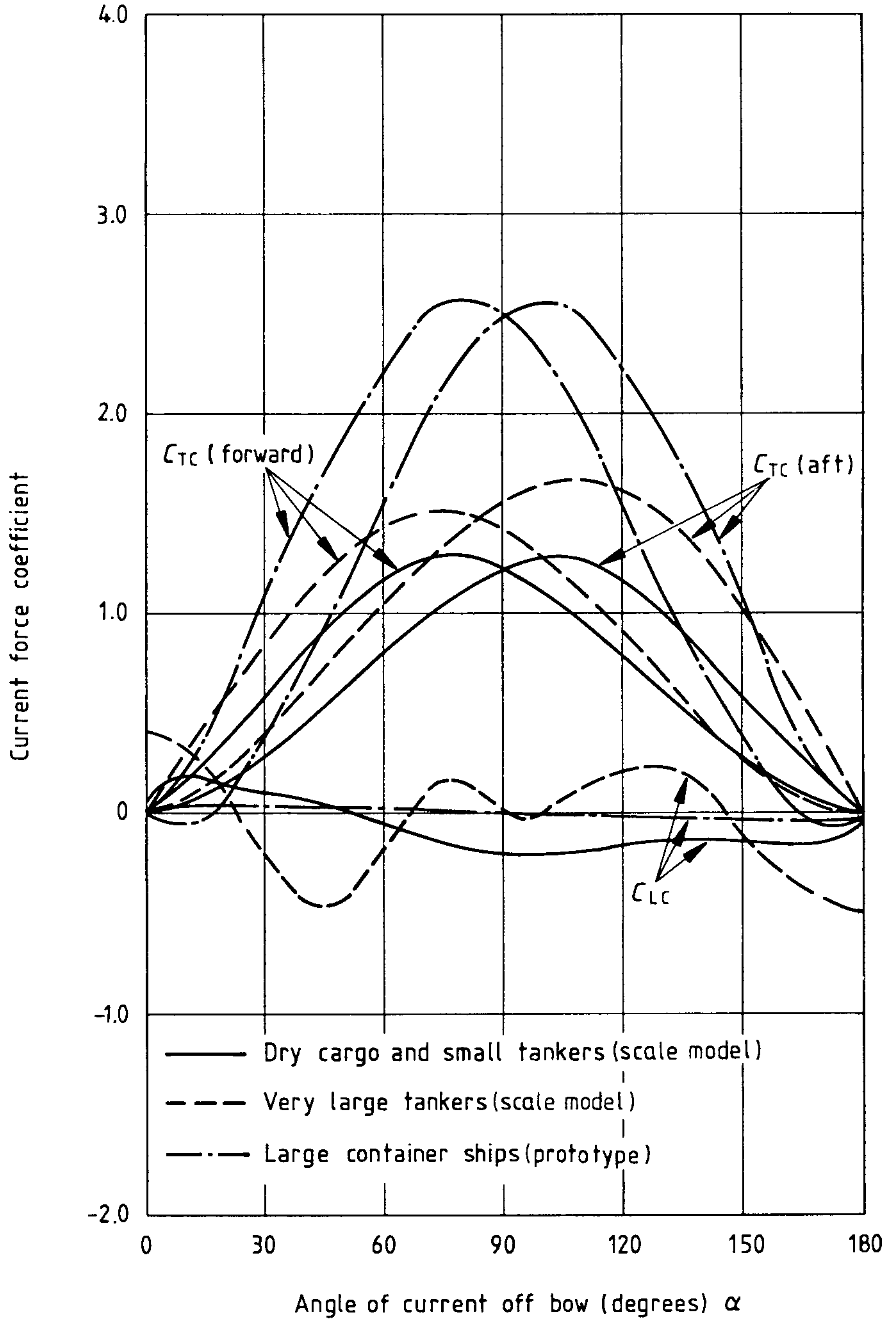

Figure 26 — Current drag force coefficients, all ships, deep water case

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 kilonewtons (kN);

F_{LC} is the longitudinal current force in kilonewtons (kN);

C_{TC} is the transverse current drag force coefficient, forward or aft;

C_{LC} is the longitudinal current drag force coefficient;

C_{CL} is the depth correction factor for longitudinal current forces[2)];

C_{CT} is the depth correction factor for transverse current drag forces[2)];

ρ is the density of water in kg/m^3 and can be taken as 1 000 kg/m^3 for fresh water and 1 025 kg/m^3 for seawater;

L_{BP} is the length between perpendiculars of the vessel in metres (m);

d_m is the mean draught of the vessel in metres (m);

V_c' is the average current velocity, resolved in the direction considered over the mean depth of the vessel, in metres per second (m/s).

Values for wind force coefficients are given in Figures 27, 28 and 29 for various angles of wind approach for various types of vessel, both in the ballasted and loaded condition.

Values for current force drag coefficients are given in Figure 26 and correction factors for shallow water effects in Figures 30 and 31.

For very large crude carriers (VLCCs) 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 32 shows an envelope of force coefficients that 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 [26] [27 [28]. In addition, further information relating to liquid gas carriers can be found in [29].

Typical values for the lengths, draughts and lateral areas of bulk carriers and container vessels are given in Figures 2, 35 and 36. These figures can be taken as guides for the values to be inserted in the expressions given in **42.2** but, for working designs, it is recommended that the characteristics of the actual ships that visit the berth be used. For wall-sided box shaped floating structures, such as pontoons, drag force coefficients have been determined from full size towing tests. Reference should be made to BS 6349-6:1989.

[2)] This is to be included when the depth to draught ratio is less than six.

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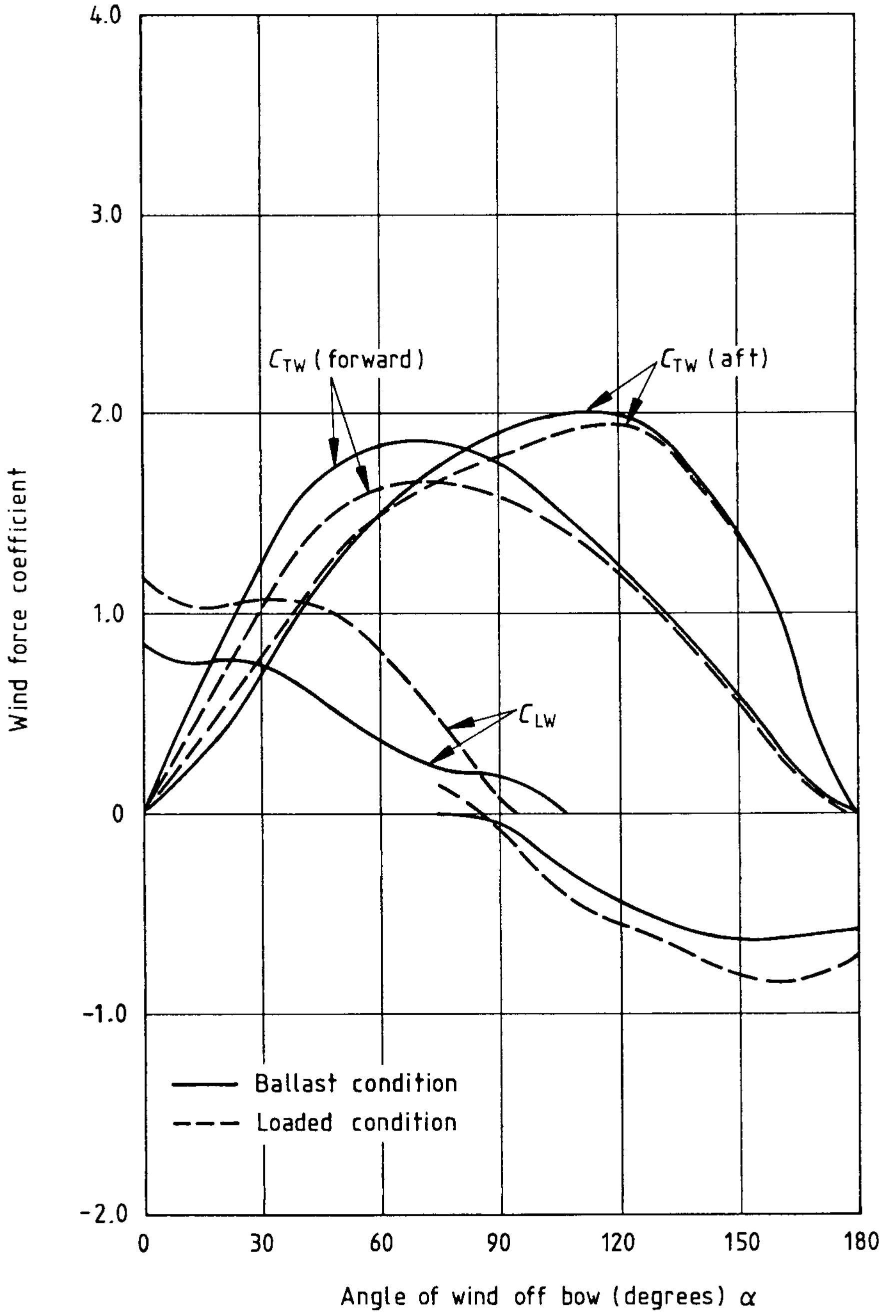

Figure 27 — Envelope of wind force coefficients for dry cargo vessels and small tankers

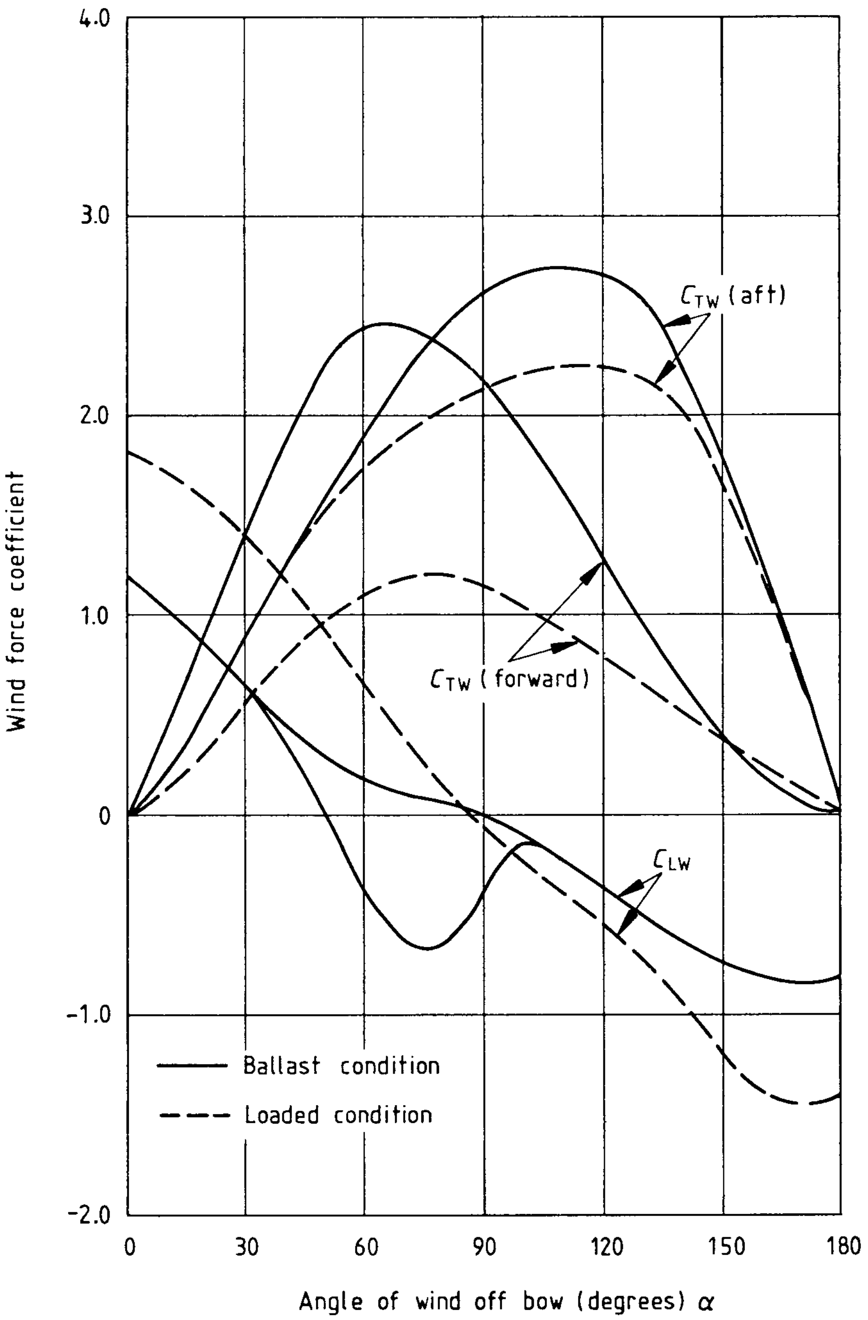

Figure 28 — Wind force coefficients for very large tankers with superstructures aft

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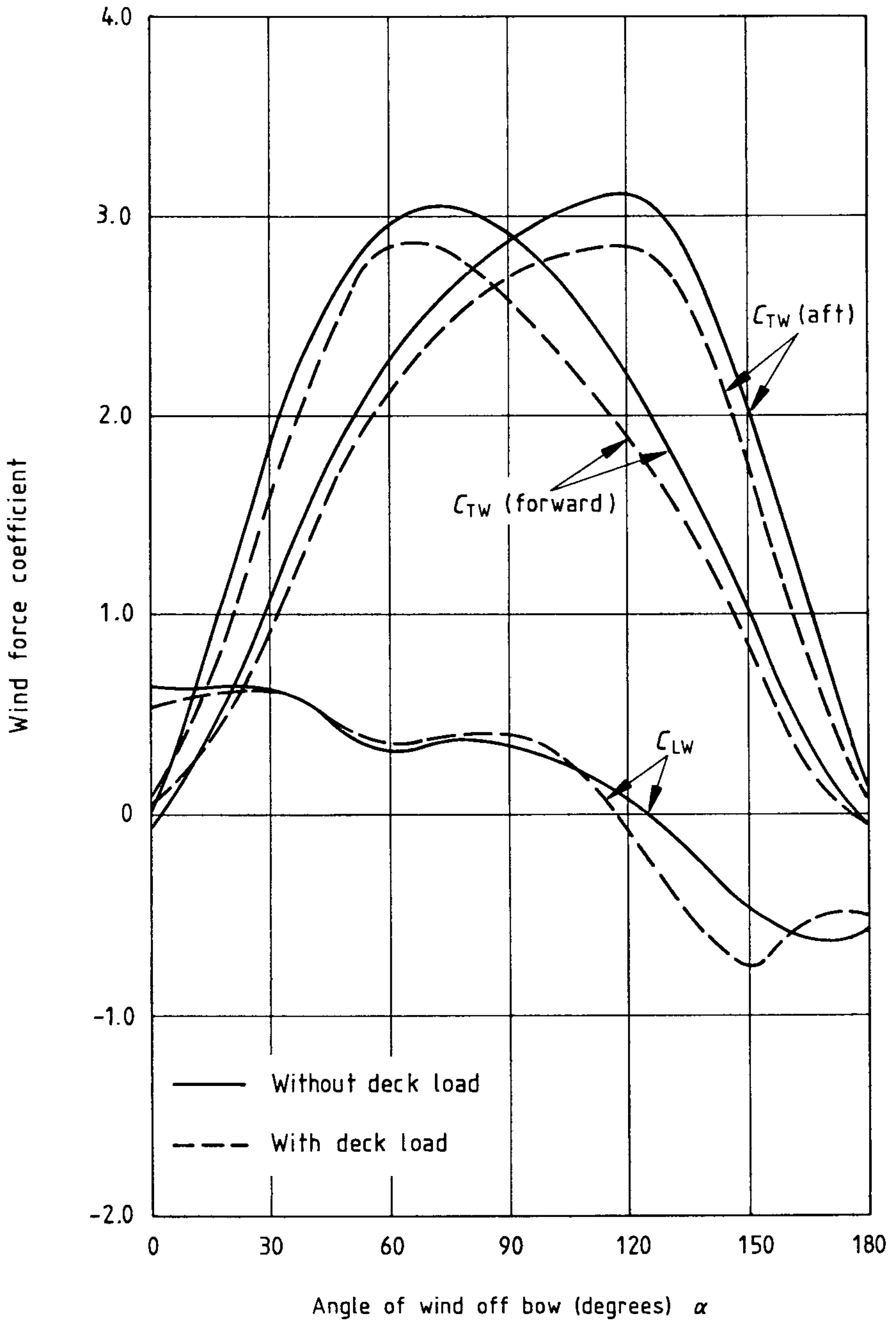

Figure 29 — Wind force coefficients for typical container ship

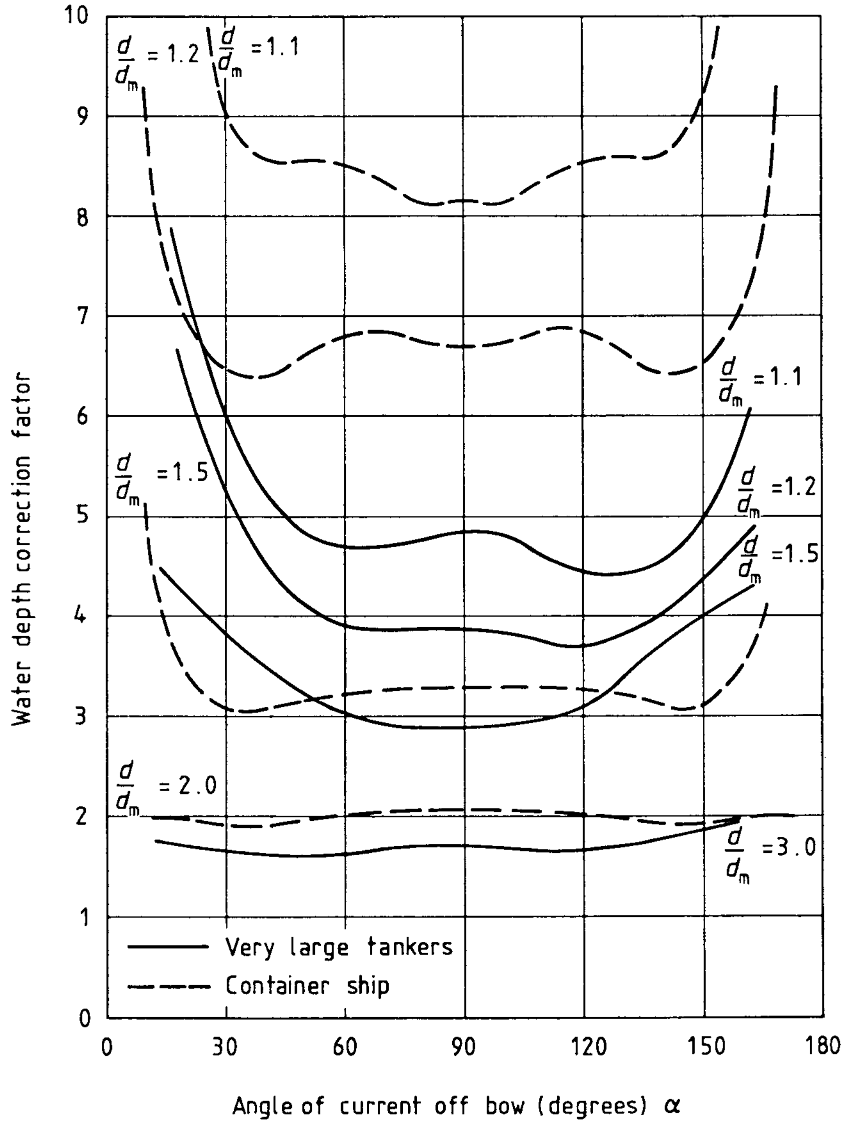

Figure 30 — Water depth correction factors for lateral current forces

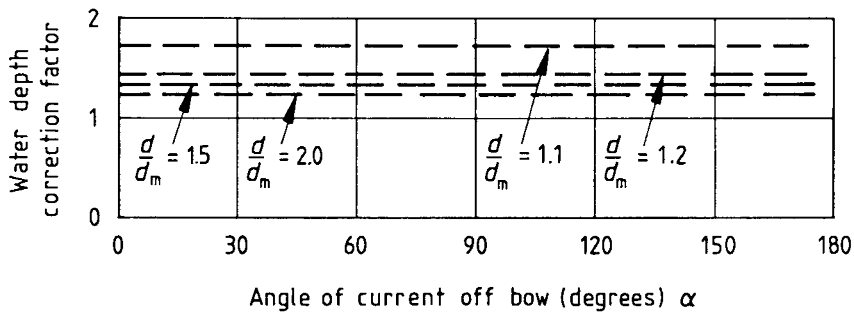

Figure 31 — Water depth correction factor for longitudinal current forces on container ships

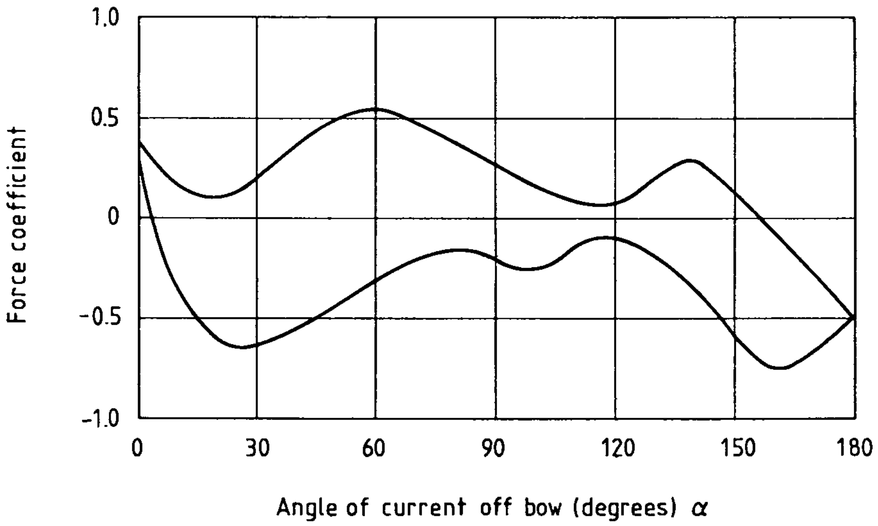

Figure 32 — Envelope of longitudinal force coefficients for VLCCs in shallow water

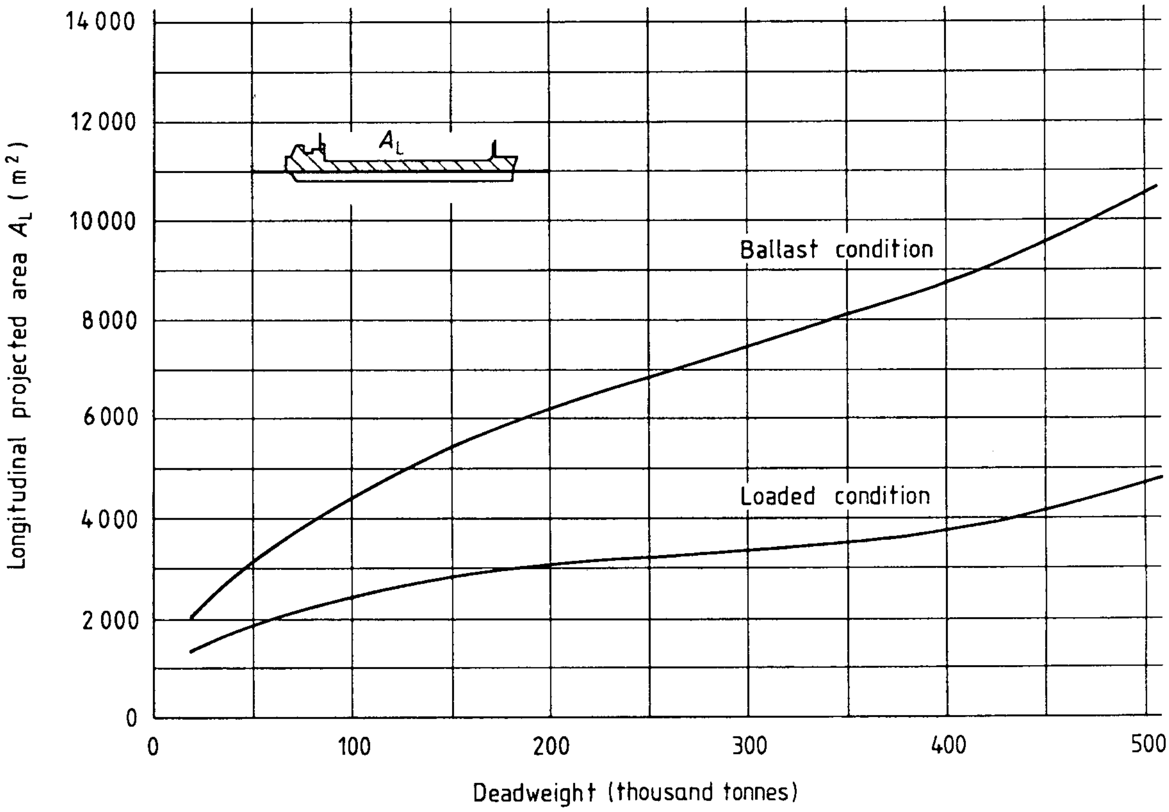

Figure 33 — Typical longitudinal projected areas of tankers

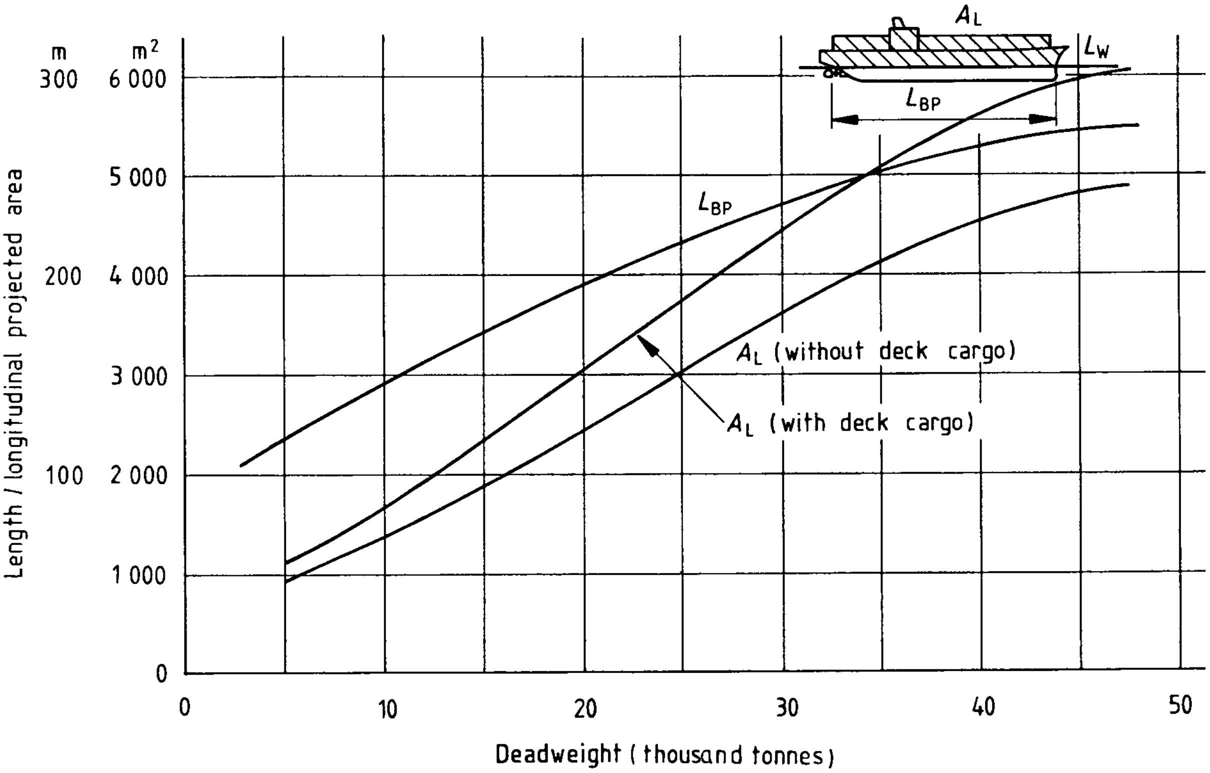

Figure 34 — Container ships: lengths and longitudinal projected areas

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, can 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 BS 6349-3:1988.

44 Cargo storage

44.1 General

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. The testing of pipelines is usually carried out using water, which should be taken into account in the loading calculations. Where the loading might 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.

44.2 Dry bulk stacks

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 that 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 can lead to increased heights. Some typical values of dry bulk densities and angles of repose are given in annex A.

44.2.1 *Other commodities*

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 can 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.

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In the absence of more specific information, the typical values of stacking height given in Table 9 should be adopted. Typical values of effective stacked densities for some common commodities are given in annex A.

If better information is not available the loading from general cargo can be taken as 20 kN/m^2.

Table 9 — Typical stacking heights

Cargo type	Stacking height m
General palletized cargo	5
Timber or timber products	6 to 7
Metal products	3
Fish	2.5
Vegetables and fruit	4

44.3 Containers

Table 10 gives equivalent uniform distributed loads for containers. Consideration should be given to concentrated loads due to:

a) corner castings of containers;

b) handling equipment of wheeled containers;

c) dolly wheels of parked trailers.

Table 10 — Container loads expressed as uniformly distributed loads

Type of load	Load kN/m^2
Empty, stacked 4-high	15
Full, 1 load	20
Full load, stacked 2-high	35
Full load, stacked 4-high	55
NOTE The values for full containers that are stacked 2-high and above include an appropriate factor for those that are less than full. A 20 ft container, when full, is taken as having an average weight of 150 kN. The maximum can be as great as 300 kN.	

44.4 Other loads

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 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 embankments, causeways, or other protective measures.

45 Cargo handling and transport systems

45.1 General

Cargo handling and transport systems operating within ports can 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.

The loading imposed on structures should be considered in both vertical and horizontal directions. When designing the superstructure in works, consideration should be given to the effects of collision impacts.

Operations of cranes are usually halted at high wind speeds and while handling cargo the wind speed considered to be acting on the crane 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, loads 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.

45.2.1 *Ship to shore container cranes*

Because the choice of type of container crane can be made after the design of the civil engineering works the loading information given as follows can be used for the initial design. Figure 35 gives typical dimensions for rail-mounted ship to shore container cranes. In service the maximum load is in the seaward two legs. Under storm conditions the maximum loads are in one of the corner legs. Maxima are in the range 4 000 kN to 6 000 kN, depending on the duty and dimensions.

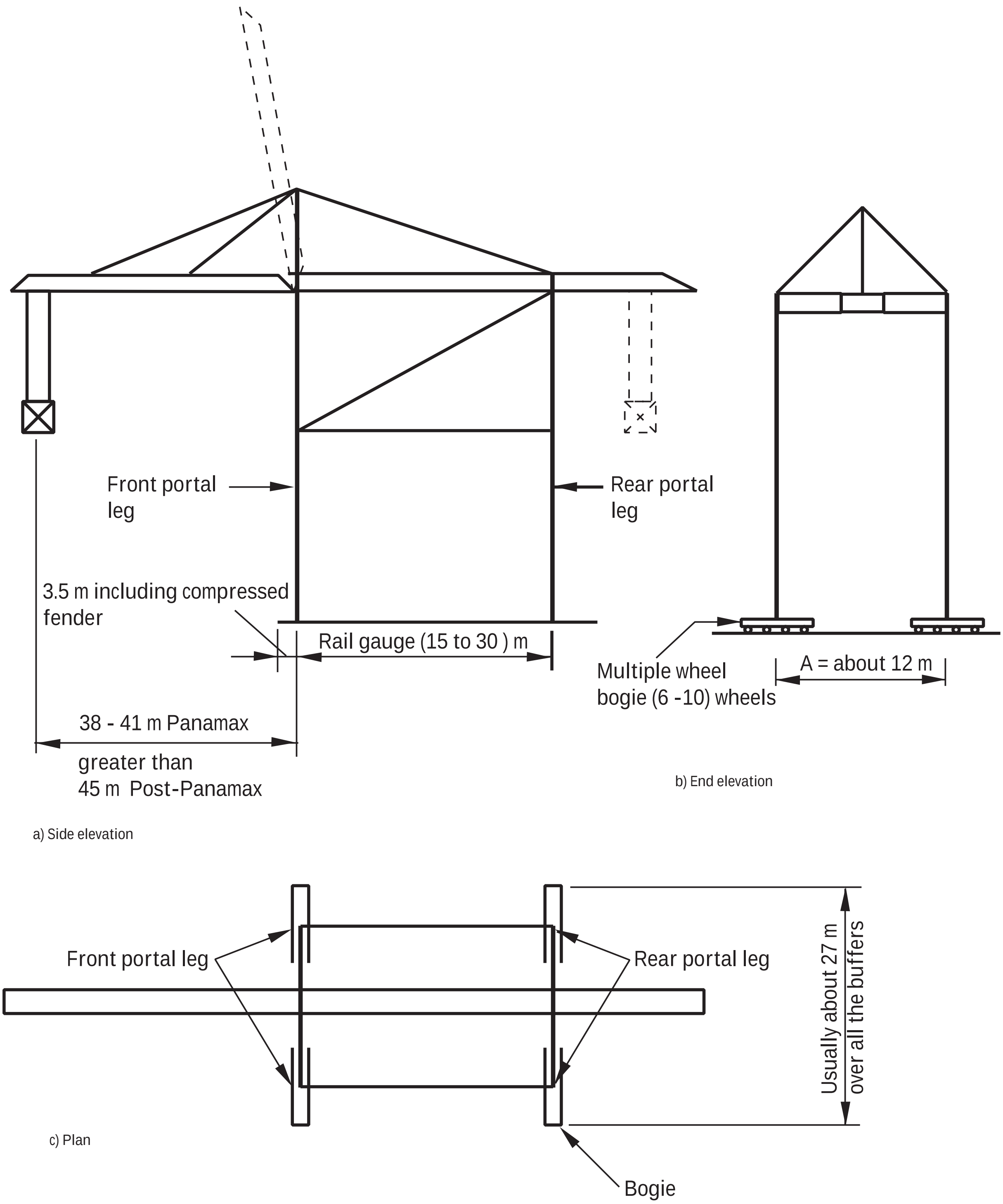

Figure 35 — Typical container crane dimensions

Wheel loads can be limited by increasing the number of wheels in each bogie, as can be seen from Figure 36, subject to any restrictions on the overall dimension between buffer faces.

Wheel loads can also be minimized by adopting a wide rail gauge, the maximum being usually 30 m. Typical maximum wheel loads for a 53 tonne lifting capacity container crane range from 500 kN to 750 kN.

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 can be restricted;

b) crossings can be numerous and curve radii smaller;

c) shunting locomotives only can 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.

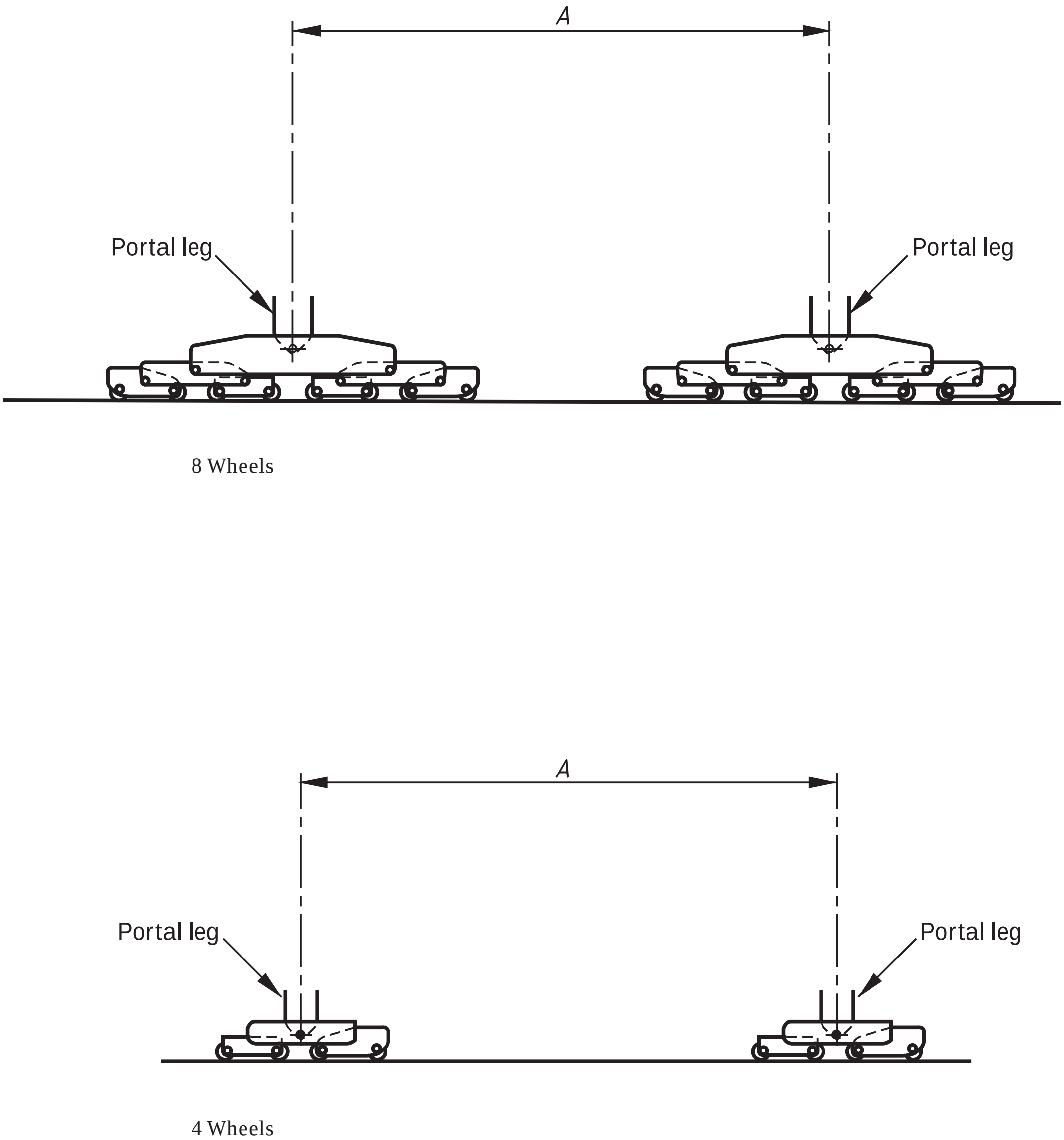

Figure 36 — Container crane bogie with alternative number of wheels

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 be given to the local effects of HB loading, details of which are given in BS 5400-2, and can 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*

Rubber-tyred port vehicles can impose considerably higher loads or local load intensities than highway traffic. Values of equivalent uniformly distributed loading are given in Table 11 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 loading*

This represents the loads from fork lift trucks (FLTs). Table 12 gives nominal loads and dimensions 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 that gives 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 intervals of 0.4 m to 0.6 m between their centres. It should be assumed that wheel loads are uniformly distributed over either a square or circular contact area and have the effective contact pressure quoted in Table 10. The exception to this is where the capacity is less than 5 t, in which case solid rubber tyres can be used and the contact area should be assumed to be a rectangle. The length of the rectangle parallel to the axle should be 150 mm. For heavy forklift trucks it might be feasible to reduce the load intensities by increasing the number of wheels per axle from 4 to 6.

Table 11 — Equivalent uniformly distributed loading for rubber-tyred port vehicles

Vehicle	Payload capacity t	Maximum laden mass t	Equivalent uniformly distributed load 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	30	50	12
(for containers)	40	70	15
Straddle carriers	10	20	10
(other)	20	36	15
	50	92	25
Tractor/roll trailer systems	20		10
	40		15
	80		20

Table 12 — FLT wheel loading: container handling duties

Payload	Length of container ft	Maximum front axle loads kN	No. of wheels on front axle	Average load per wheel kN
28	20 (part full)	665	4	166
32	20	685	4	171
35	40 (part full)	780	4	195
42	40	900	4	225

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45.6.2.1 *Fork/front lift truck loading*

Wheel loads for small FLTs in the 5 t to 10 t payload range are normally no greater than those for highway traffic. The larger FLTs that are used for handling containers can seriously damage paving that has been designed for lorries, for which the maximum permitted individual wheel load is 50 kN.

Table 12 gives typical wheel loads for FLTs that are used for handling containers.

45.6.3 ***Side loaders***

Local wheel loading from side loaders used for the handling of containers and other cargoes is covered by HB loading (see **45.5**). However, side loaders also impose outrigger or jack loadings and provision should be made for these loads. Typical values of side-loader jack reactions are given in Table 13, which can be used in the absence of more precise information. The jacks are all located in a straight line at the spacing quoted in Table 13.

45.6.4 ***Straddle carriers***

HB loading (see **45.5**) covers local wheel loadings from straddle carriers used in the handling of containers and other cargoes.

45.6.5 ***Mobile cranes***

HB loading (see **45.5**) covers local wheel loadings from mobile cranes. Provision should also be made, however, for the outrigger reactions and bearing pressures that might 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 might 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 14. 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 given in Table 14. The other outrigger loads can be calculated as sharing the sum of the maximum lift plus the machine weight, less the outrigger loads already calculated. Because the contact areas can be varied by the type of spreader used and by the use of packing, no values have been given of imposed bearing pressures, but pressures in excess of 1 000 kN/m^2 can develop unless restrictions are imposed.

45.6.5.1 *RT loading*

This represents roll trailers with hard-rimmed tyres and their associated tractors. The plan dimensions are shown in Figure 37 and the loads are shown in Table 15 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.

45.6.5.2 *Rubber-tyred gantry cranes*

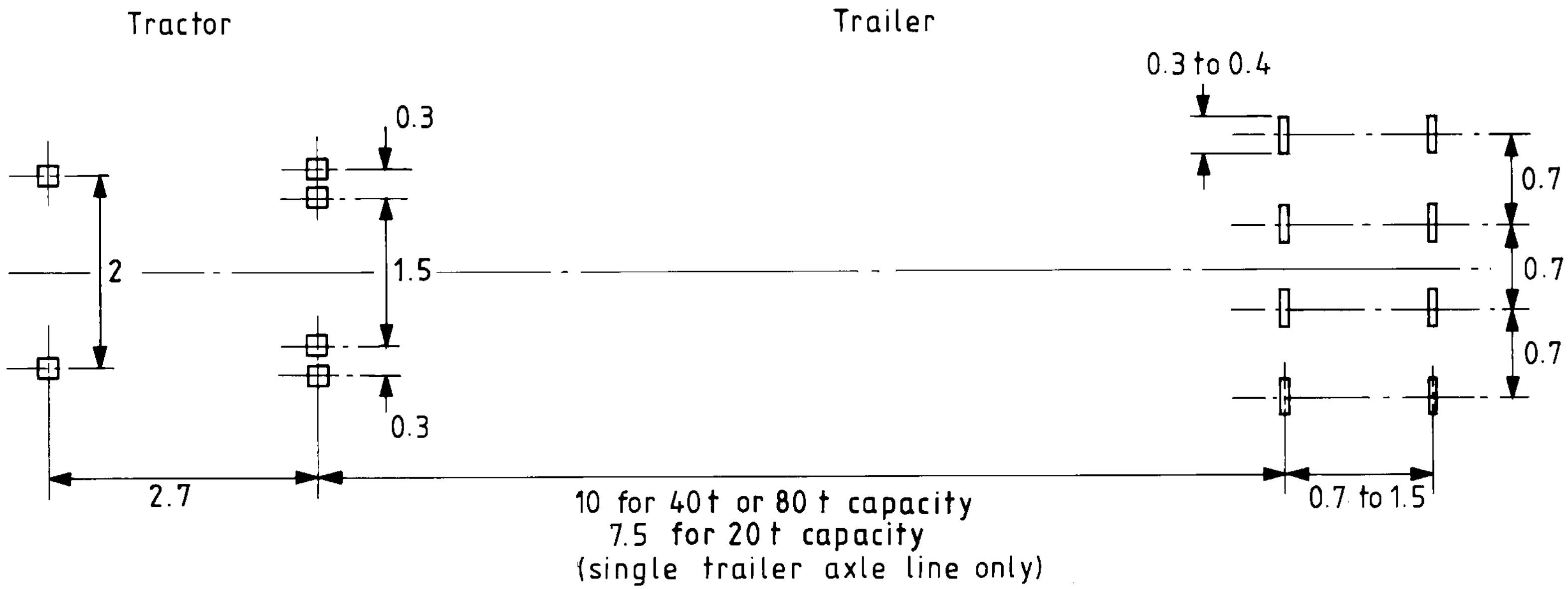
Due to the wide range available, it is recommended that details be obtained of the particular equipment when it is proposed to use such cranes. Large gantry cranes for container handling can impose individual wheel loads of up to 450 kN with contact pressures of 830 kN/m^2.

Table 13 — Side-loader jack reactions

Payload capacity t	Mass unladen t	Number of jacks	Jack spacing m	Jack load kN	Contact pressure 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 14 — Mobile crane outrigger reactions

Load-moment capacity range t·m	Maximum lift capacity range t	Typical mass of crane t	Outrigger spacing m	Maximum outrigger reaction 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 25	100	8.0	550
300 to 500	30 to 36	130	8.0	800
500 to 900	30 to 40	200	8.0	1 000
900 to 1 300	50 to 80	240	10.0	1 500



All dimensions are in metres.

Figure 37 — Dimensions of RT vehicle

45.6.6 ***Tracked cranes***

Where caterpillar tracked cranes are used, the imposed loading should be taken, in the absence of more precise information, from Table 16 according to the maximum size of crane expected. The maximum contact pressures can be imposed as a uniform pressure under one track or as the maximum of a triangular distribution under both tracks. The contact area should equal that required to support the weight of the crane and its load within the given limits. Tracked cranes are likely to cause local damage to blacktop and, to a lesser extent, to concrete surfaces unless protective mats are used.

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 can be taken as shown in Table 17.

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 can 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 can 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, might also have to be taken into account.

Where rubber-tyred gantry cranes are operated, the wheel loads should be ascertained for the specific equipment.

In the absence of specific information, for straddle-carrier container operations, laden carriers can be considered as equivalent to either:

a) 6 wheels, each imposing 130 kN, arranged in two parallel lines; or

b) 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 can be derived. The damaging effect of one pass of a vehicle transmitting, for example, a 10 t axle load is normally greater than that of two passes of the same vehicle transmitting a 5 t axle load. As a conservative estimate, therefore, the throughput can be taken to be in units equal to the heaviest unit load. Alternatively, a more precise spectrum can be used if sufficient information on traffic patterns is available.

Table 15 — 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

Table 16 — 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

Table 17 — 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

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 that cause horizontal displacements. In addition, the loading is very largely of a dynamic nature and can give rise to larger displacements than the same loading applied statically.

There are many areas in which movement and vibration problems can 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 can be classified into two general groups: rigid and flexible. Rigid structures are those that cater for horizontal loading by carrying it mainly in direct compression and/or tension. This group includes filled earth structures such as quay walls and breakwaters, and structures incorporating raked piles. Flexible structures are those that 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 can be classified into four groups:

a) cyclic;

b) impulsive;

c) random;

d) static and long term cyclic.

Each group of loadings causes a different type of response in the structure and requires 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 [16].

Conservative assessments of the displacement of both flexible and raked pile rigid structures can be made using the approximate methods given as follows. For flexible structures in open sea or in water more than 30 m deep, the results can be too conservative and a more exact analysis might be necessary.

47.2.2 *Cyclic loads*

The displacement under cyclic loading 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 can be obtained by modelling the structure as a single-degree-of-freedom system. In this model the stiffness is 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

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impulse, the mass oscillates at the natural frequency f_N such that:

$$f_N = \sqrt{\frac{1}{2\pi}\left(\frac{K}{m_e}\right)}$$

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 is:

$$d_c = \frac{P}{K}\sqrt{\left\{\frac{1}{(1-(f_c/f_N)^2)^2+(2qf_c/f_N)^2}\right\}}$$

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 can be taken as equal to 0.01.

The square root term in the previous 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 previous expressions to obtain approximate dynamic response, the stiffness of elements of the structure can 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 lies 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 a model of the actual mass distribution. This actual mass distribution can be estimated by assuming a simple pinned support, which is in the direction of the motion and at the node at which the equivalent mass is to be placed, e.g. horizontally at deck level on a piled jetty. A static analysis can then be performed by using the distributed masses as loads, to give the reaction at the assumed support. This reaction can 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 the water externally entrained by the structure, including that entrained by the attached marine growth.

Typical values of added mass of entrained water are given in Table 18 for a number of cross-sections.

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Table 18 — Added mass of entrained water

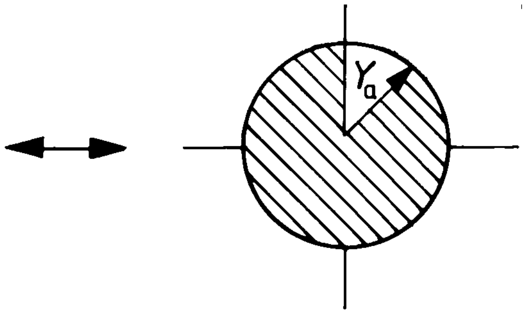
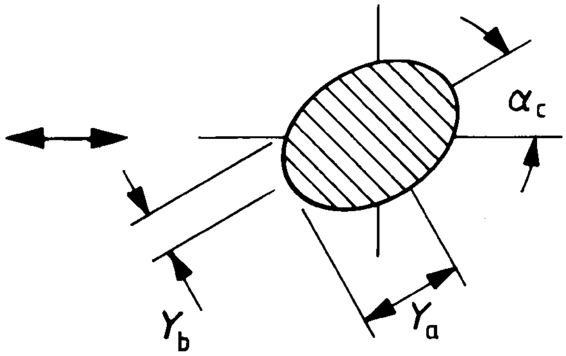
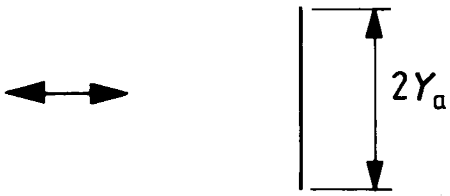
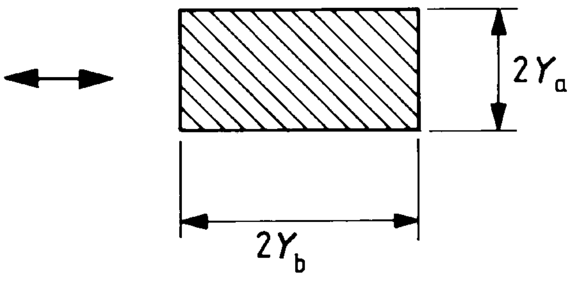
<table>
<tr><th colspan="2">Cross-sectional shape of prismatic member</th><th colspan="2">Added mass of entrained water per unit length*</th></tr>
<tr><td>Circular section</td><td></td><td colspan="2">$\rho\pi Y_a^2$</td></tr>
<tr><td>Elliptical section</td><td></td><td colspan="2">$\rho\pi(Y_b^2 \cos^2 \alpha_c + Y_a^2 \sin^2 \alpha_c)$</td></tr>
<tr><td>Flat plate</td><td></td><td colspan="2">$\rho\pi Y_a^2$</td></tr>
<tr><td rowspan="9">Rectangular section</td><td rowspan="9"></td><td colspan="2">$K_1\rho\pi Y_a^2$</td></tr>
<tr><td>Y_a/Y_b</td><td>K_1</td></tr>
<tr><td>0.1</td><td>2.23</td></tr>
<tr><td>0.2</td><td>1.98</td></tr>
<tr><td>0.5</td><td>1.70</td></tr>
<tr><td>1.0</td><td>1.51</td></tr>
<tr><td>2.0</td><td>1.36</td></tr>
<tr><td>5.0</td><td>1.21</td></tr>
<tr><td>10.0</td><td>1.14</td></tr>
</table>

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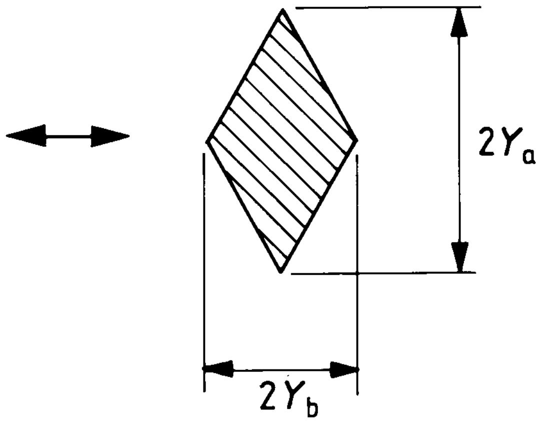
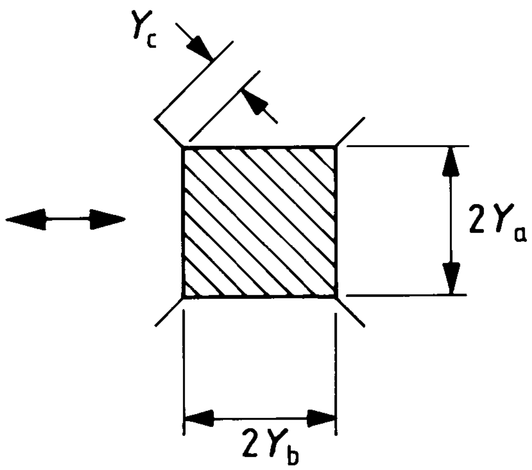
Table 18 — Added mass of entrained water *(continued)*

Cross-sectional shape of prismatic member		Added mass of entrained water per unit length*	
Lozenge section		$K_2\rho\pi {Y_a}^2$	
		Y_c/Y_a	K_3
		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_a	K_3
		0.05	1.61
		0.10	1.72
		0.25	2.19
*ρ is the water density			

Unless better information is available it can be assumed that marine growth has the following maximum values, below Lowest Astronomical Tide (LAT).

Depth below seabed m	Load kg/m^2
0–10	250
10–20	200
20–30	125
30–50	80

The possibility of seaweed fouling by kelp, which can reach 3 m in thickness in some North Sea offshore installations, should be checked.

Vortex shedding from circular sections produces a particular type of cyclic loading in that 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 can have some effect on structural elements, although significant amounts of energy are unlikely to be present at the natural frequencies of the structural elements. These are 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.

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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 mooring lines;

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. Guidance on the design of fenders is given in BS 6349-4:1994.

Normal wave loading is random, because 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 can be made by subjecting it to a series of loads resulting from a regular wave train of maximum design height with differing frequencies. Calculations can then be carried out using the methods described in **47.2.2**.

Within a harbour the loadings from wave-induced vessel motion is 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 is 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 requires the assistance of a specialist and it might 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 can occur if the natural frequency in roll of the vessel is below 0.25 Hz.

Seismic activity can cause very large displacements, because the distribution of energy in an earthquake often shows a marked concentration at the low frequency end of the spectrum, within which fall most of the natural frequencies of maritime structures.

47.2.4 ***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 can be used to calculate the resulting movements.

47.2.5 ***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 5 (see clause **36**).

47.3 Acceptability criteria

47.3.1 ***Structural adequacy***

A fluctuating force at or near the natural frequency of the structure or individual member of the structure produces 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 can occur, e.g. a jetty head and its approach structure can 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 are associated with high horizontal velocities and accelerations. Mechanical equipment and plant on the structure 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 should be checked against overturning. Movements and vibrations can also adversely affect rigid buildings on the structure.

Consideration should be given to the psychological effect of sway on pedestrians, in that small movements seem to be much larger than they really are, giving rise to disquiet. In the case of structures intended largely for pedestrian or passenger traffic, designs should be sought that minimize this problem.

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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 aid in the interpretation and evaluation of the information obtained.

The study should include assessment of the characteristics of soil or rock formations, which can be retained by structures or provide their foundations or which can be incorporated in or affected by earthworks in the form of dredging and reclamation. It 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 can be predicted. The next stage in the geotechnical design process is the calculation of earth pressures and earth resistance, which can 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 normally 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 as follows.

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) [4];

b) Ordnance Survey maps and old maps;

c) meteorological data;

d) national and local government records;

e) geological maps and memoirs;

f) aerial and satellite photographs;

g) information 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 that can 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 that can result from construction of the new works.

In particular, it should be remembered that many maritime sites are areas of recent geomorphological development and that such processes are more active if the site is nearer to the shore. The timescale of these changes is often comparable with the lifespan of the project and they can often 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 often result from coastal rather than river transport and are usually of medium to coarse size. They can arise from small embayments and would therefore be very local whereas they can occasionally be from long lengths of foreshore.

Organically formed deposits are frequently of significance. They are mainly calcareous, of which coral and calcite-excluding algae that cement detrital material are two examples. All such materials are subject to leaching and can 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 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 normally 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 are 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 groundwater 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 [30].

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 explosives might be needed to facilitate removal by dredger.

Standard penetration tests can be made in boreholes drilled by rotary or cable percussion methods in granular soils to obtain a measure of their relative density and therefore 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. This is particularly the case when carrying out an exploration in dense soils at a shallow depth below the seabed. Seabed or down-hole supported equipment is, however, 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.

Plate bearing tests should be carried out in situations where site investigations are land-based and where close estimates are required for the deformation of heavily loaded structures. This can include, for example, the lateral and vertical movements of sills supporting caisson-type gates for locks and graving docks. 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. 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. The possible risks for instability due to a rotational slip (see **54.2**) should also be investigated. This encompasses the shoreside works and any dredging of the seabed for berths on the water side of the site (see Figure 38).

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Boreholes for jetties should be sited along the line of the berthing heads and mooring structures and along the line of the approach structures. At least one borehole should be close to the planned position of each major component of the structure and, in general, boreholes should not be more than 50 m apart.

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 38).

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 39).

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 39).

Boreholes drilled in rock formations should be taken to a depth sufficient to explore the thickness and characteristics of any weathered rock that 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.

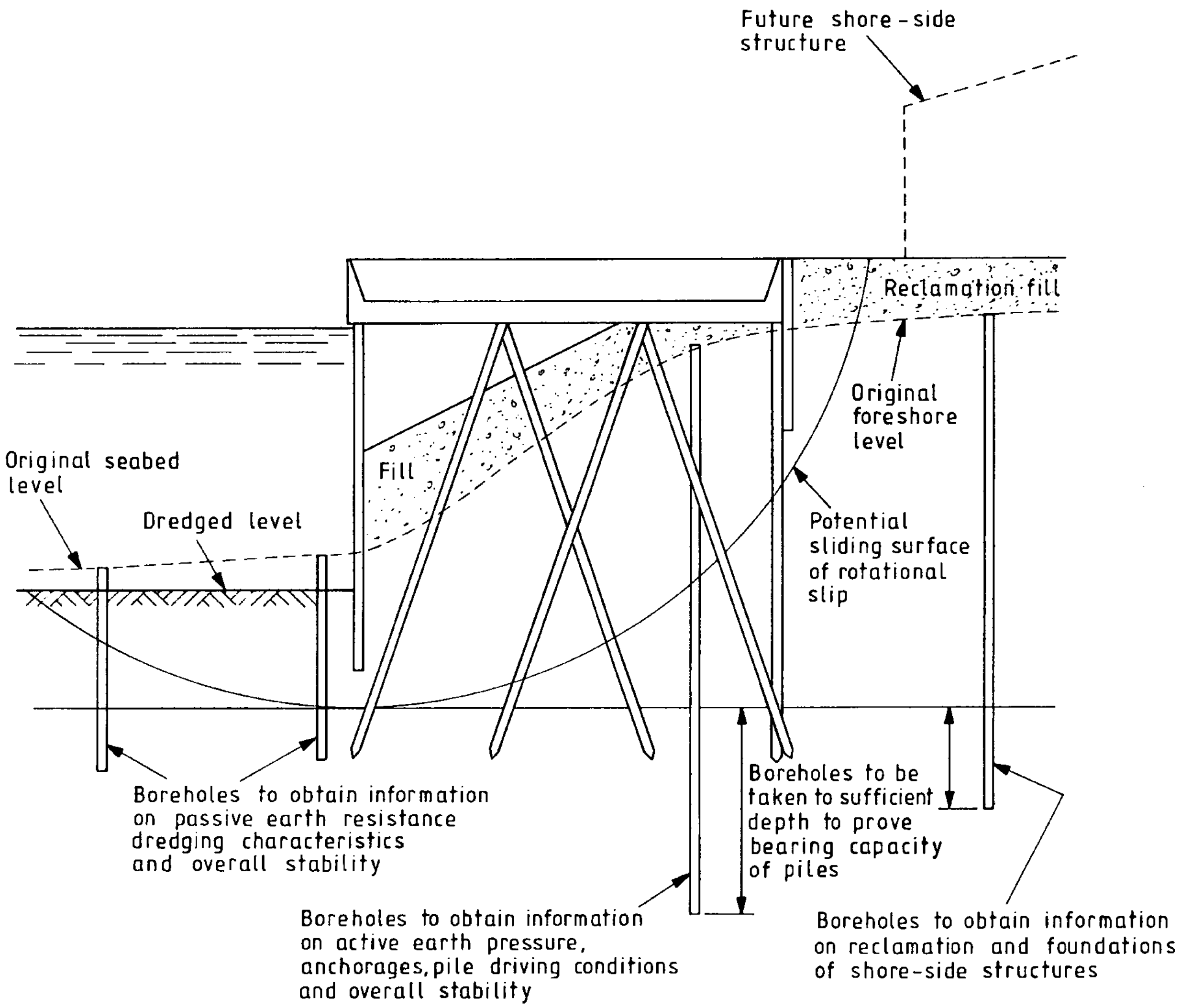

Figure 38 — Location and depth of boreholes for piled wharf

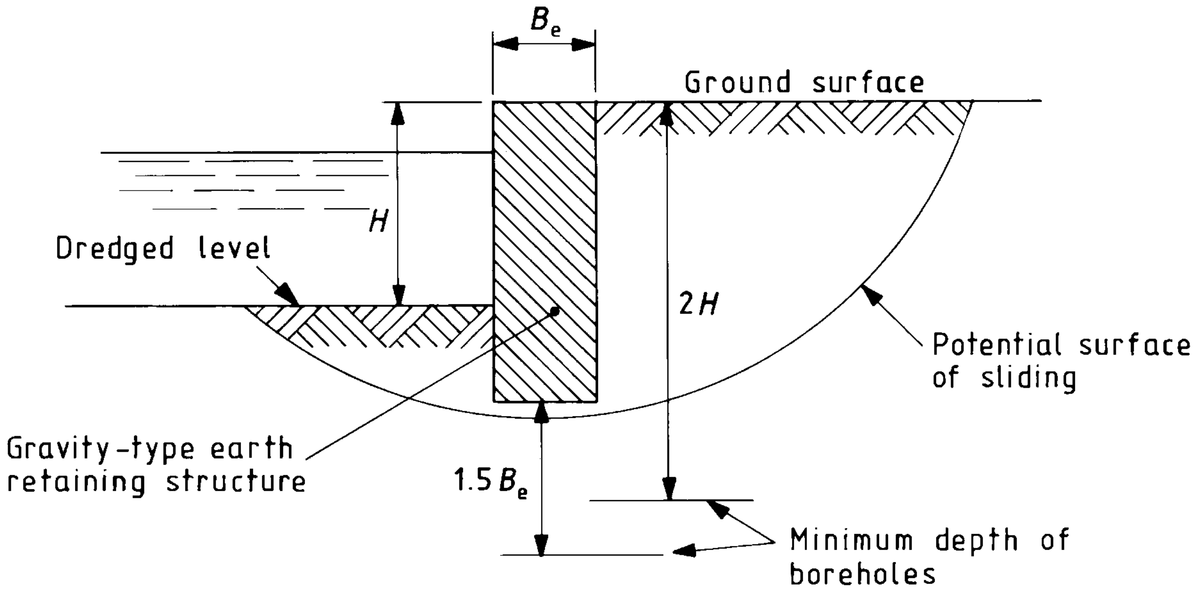

Figure 39 — Depth of boreholes in relation to retained height of soil and width of quay wall

Dredging below the seabed can involve removal of soil from below the designed dredge level to conform to 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 Groundwater 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 can cause corresponding fluctuations in piezometric head in the groundwater. The effects on groundwater 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 groundwater 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 groundwater at various positions back from the waterside face can indicate the relative influences of saline water and non-saline groundwater on piezometric levels.

The possible existence of groundwater 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 groundwater lowering schemes are proposed to enable the construction of dock basins or lock chambers, the groundwater 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.

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 such as strutted reinforced concrete walls or mass gravity structures anchored to rock (see **6.4.3.1**). Methods of making these measurements are described elsewhere [31].

49.9 Detection of underground movements at depth

Where maritime works are to be constructed on sites suspected of having suffered previous instability it might 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 might arise as a result of earthworks associated with shoreside structures. For further guidance see **5.1.3.5** of BS 6031:1981.

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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 groundwater 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.

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.

Further guidance on geophysical surveys is given in BS 6349-5:1991.

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

The feasibility of driving piles to the required penetration for resisting compressive, uplift and lateral loading needs to be confirmed. Pile driving and loading tests might be appropriate for this purpose. Tests done in advance of the main construction programme can prevent heavy costs being incurred later due to delays and changes in design, which can often happen because of 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 employed for the construction of the permanent works.

The resistance of piles to lateral loads or the deflection of a pile under a given lateral load is dependent on the stiffness of the pile as well as the soil conditions below ground level. Field trials are advisable in order to confirm the pile performance and should be carried out using a pile of the same dimensions and stiffness as are used for the permanent works.

Similarly, installation and pulling trials on ground anchors (see clause **53**) can confirm the design capacity of anchorage systems. The trials can include testing anchors to failure under pull-out load, or they can 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 denatured excavations, it might be advantageous to make trial excavations to assess the suitability of a particular system of groundwater control and to confirm estimates of pumping rates made on the basis of field permeability tests (see **49.7**).

Trial dredging of soils or rocks is not always feasible or desirable, because certain types are dependent on the kind of dredger used and the manner in which it is operated. Trial dredging, though, might be desirable as a means of making full-scale experiments to determine the safe side slopes of dredged areas. The resulting excavation can also be used to study siltation rates (but see **14.4.2**).

49.12 Studies related to constructional materials

A site investigation might 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 groundwater, 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 can 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.

Deterioration due to freeze/thaw and wetting/drying should be considered.

Studies of the potential corrosion of steel structures (see clause **66**) should include measurements of the electrical resistivity of the soil and the presence or absence of sulfate-reducing bacteria (see **59.2.1** and **66.8**). 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 analysis in accordance with DD 175.

The site investigation should include observations on existing maritime structures in the locality, with particular reference to the durability (see **57.2**) of locally available stone when used for slope protection and in breakwaters, the extent of barnacle growth, and attack by maritime organisms (see **13.5.1** and **60.2**) on submerged structures, such as piling, fenders and groynes. 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 might be required of the size of stones that 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 might 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.

50 Properties of the ground

50.1 Average properties for preliminary design

It might 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 19 can 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 that are critical to the stability of the structure. Where necessary, special methods of sampling and testing in these layers should be adopted, in order to obtain parameters of the desired quality and relevance for each particular geotechnical problem. In the course of this detailed examination, attention should be paid to the geology and geomorphology of the site, including its stress history and the occurrence of layering or lenses of different materials. In addition, the effects of disturbance caused by construction operations, dynamic loading during the service life of the structure, and the strain-dependent and time-dependent characteristics of the soil should all be taken into account.

Consideration is given in **50.2.2** to **50.2.7** 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.

Table 19 — Mobilized angle of friction

Type of structure	Relative soil density	Angle of friction δ_m/δ_{max}
Masonry walls with horizontal movement	Loose	0
	Dense	0.5
Light anchor walls	Loose	0
	Dense	0
Sheet pile walls freely embedded	Loose	1.0
	Dense	1.0
Sheet pile walls driven into rock	Loose	0
	Dense	0.5

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Guidance on the properties of the ground in relation to various structures can be found in BS 6031 and BS 8004. For coastal sites these codes should be read in conjunction with **6.3.2.2** to **6.3.2.7**, 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 [32]. 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 40). This should be considered in relation to the large deformations that 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 $2\varphi_r/3$. Mobilization of this maximum value requires movement between the structure and the soil. The amount mobilized, δ_m, for passive resistance can be taken from Table 19.

Because sands and gravels are highly permeable, groundwater shows a rapid response to any fluctuation in the water levels of an adjacent waterway. On a falling tide this can result in outflow and erosion due to rapid drawdown of groundwater from unprotected slopes. Sands and gravels are also susceptible to erosion because of 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 more critical design factor 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 [32].

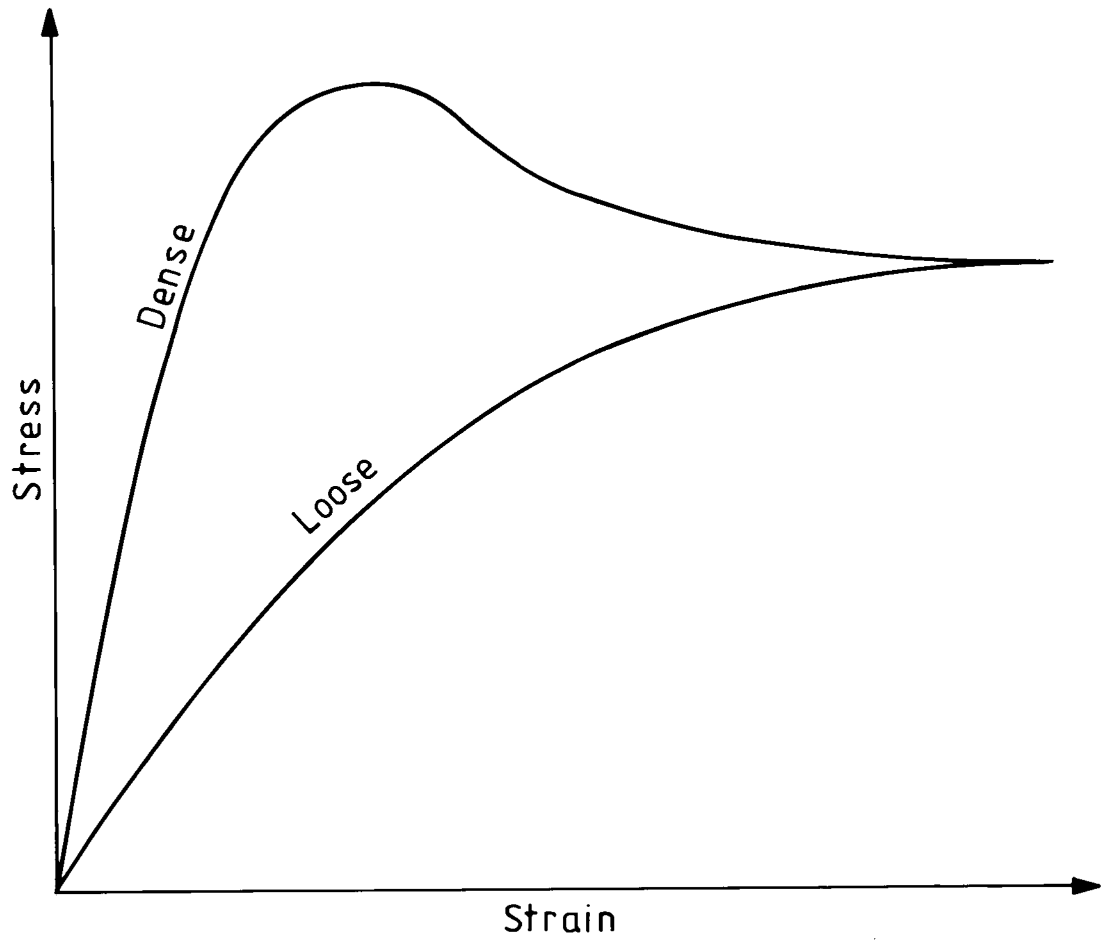

Figure 40 — Plane strain shear diagram for sand

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 [33].

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.

Sands and gravels can be cemented in varying degrees by a saline or calcareous matrix, which can impart an appreciable cohesive strength to the soil. The possibility of complete or partial loss of the cementitious value of the matrix should also be considered. This loss can 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 that 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 pressure 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 previously for sands and gravel (**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 loose soils, can 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 that 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 [34]. 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 the loss of undrained shear strength of the mass of clay surrounding the piles. Consequently there is a reduction in the passive resistance on the face of any nearby sheet piles, which might be forming an earth-retaining structure on the landward side of the wharf.

Vibrations from pile driving or earthquakes can cause the 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, because they measure strengths corresponding to the stress conditions at the level at which the test is made. Subsequent dredging for slopes and berths, though, reduces 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 depends 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.

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The stress-strain relationship for normally-consolidated clays is shown in Figure 41. 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 can vary according to the direction of application of the deviator stress to the test specimen. This is because of the effect of discontinuities in the specimen or to differences in the ratio of vertical to horizontal stress in the soil in situ [34]. In the triaxial compression tests the effects of pore pressure changes in the test specimen should also 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 might 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 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 41) 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. The orientation of the principal fissure system within the test specimen should therefore be considered when assessing triaxial compression test results.

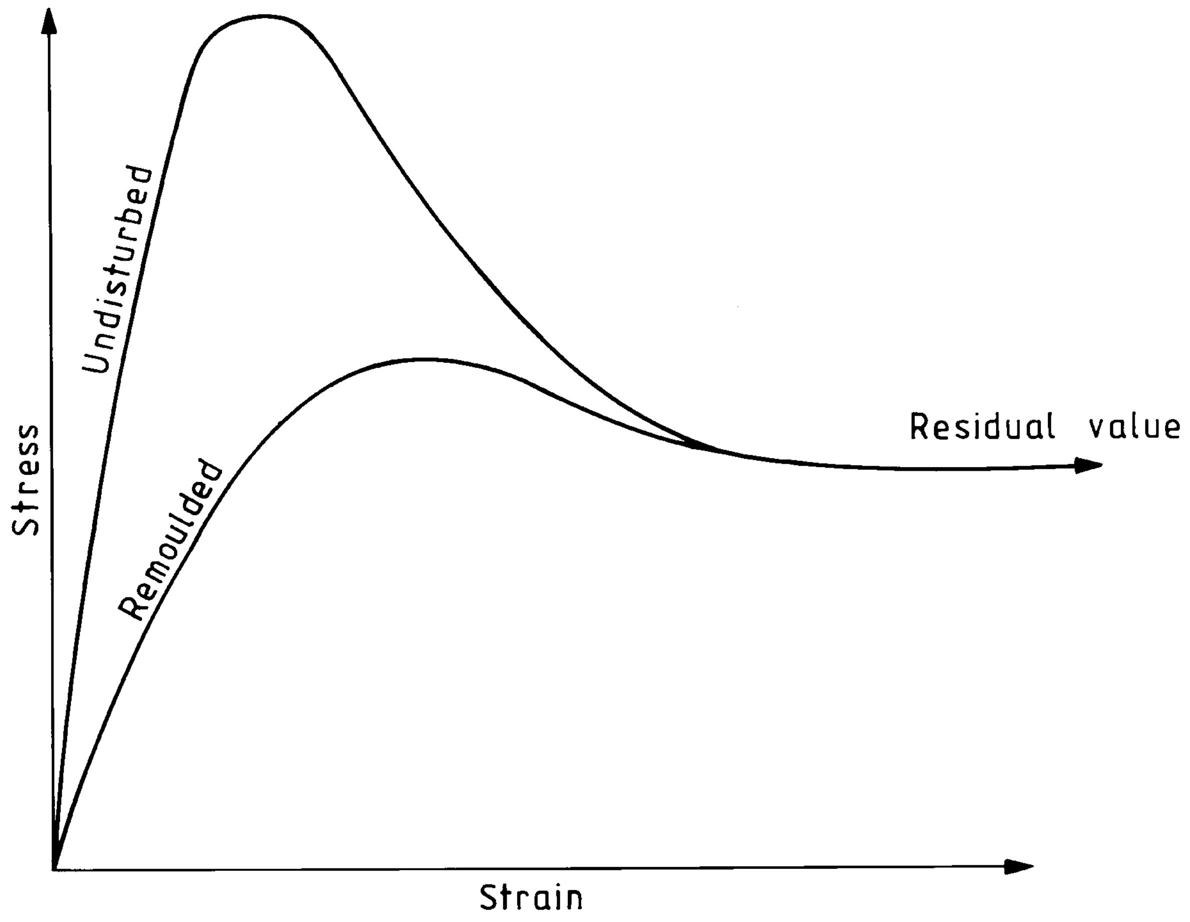

Figure 41 — Plane strain shear diagram for normally-consolidated clay

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 causes 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 can have a laminated structure, which has similar effects on mass permeability to those of a fissure system.

50.2.6 ***Rocks***

In maritime structures the stability of rock masses might 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 might need to be considered in the case of underwater 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. These can be, for example, 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 Figures 42, 43 and 44.

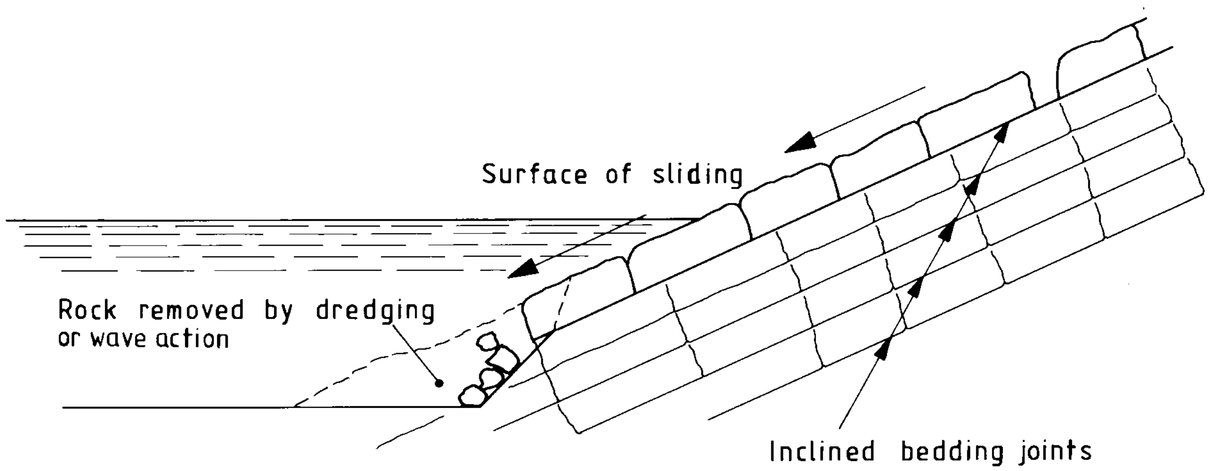

Figure 42 — Slab slide in rock

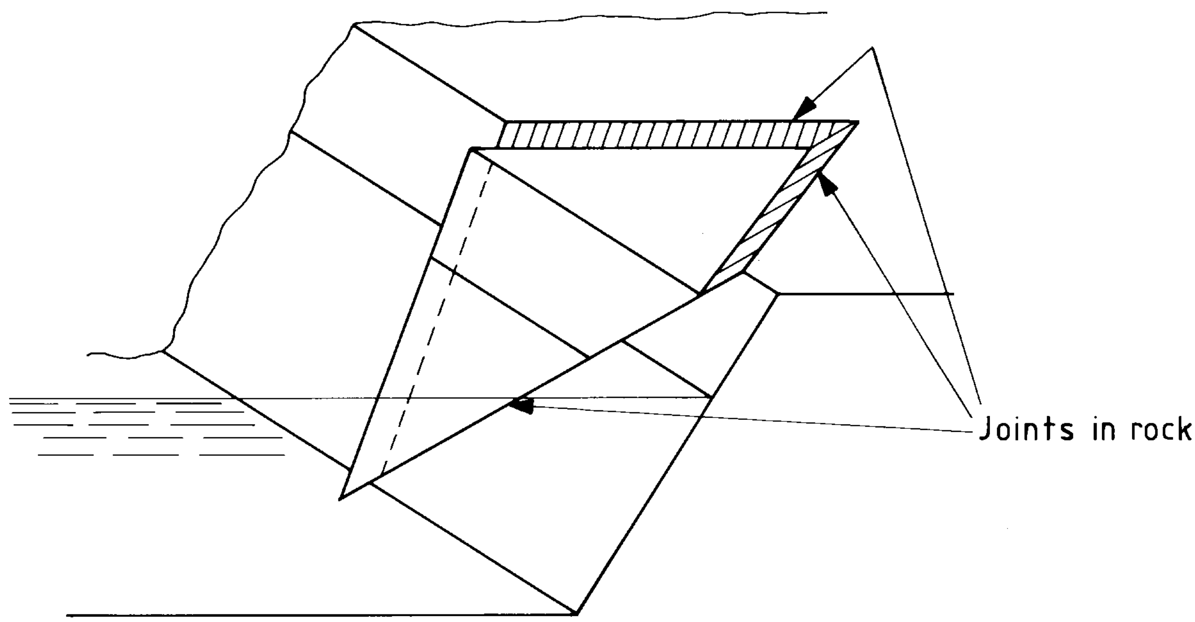

Figure 43 — Wedge failure in rock

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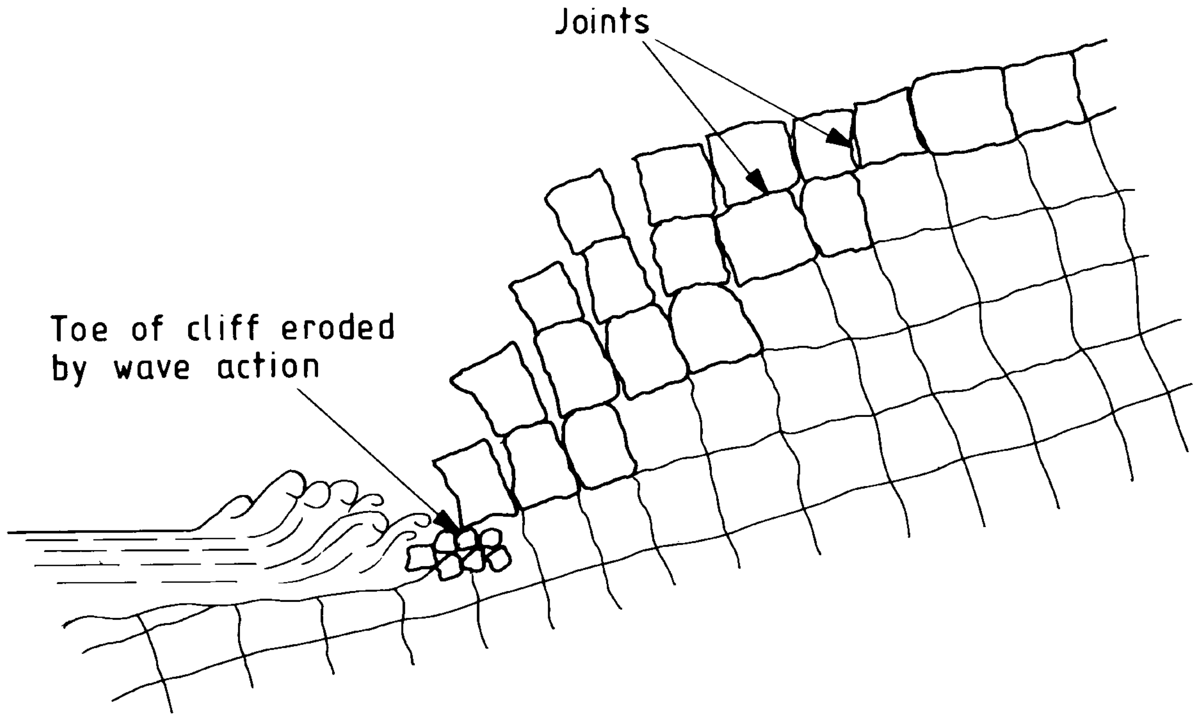

Figure 44 — Toppling failure in rock

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.

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 are in a loose state of deposition and parameters for density and shearing resistance obtained from the values given for loose conditions in Table 20 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 are in a medium dense to dense state. The density depends on the degree of compaction given to the fill and the parameters should be selected accordingly. Consideration should be given to compacting loose materials above or below water level using special techniques of deep compaction [35].

Where cohesionless soils are pumped immediately behind a retaining wall the fill above normal standing water-level might 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 undergo time dependent consolidation as a result of crushing and degradation of points of contact between rock fragments. On an unyielding surface this can amount to up to 1 % of the height of the rockfill [36] [37]. High densities with a correspondingly high angle of shearing resistance can be obtained from crushed rocks compacted above water level.

Table 20 — 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	
		Moist kN/m^3	Submerged kN/m^3	Active degrees	Passive degrees
Gravels	Loose	16.0	10.0	35	35
	Medium dense			38	37
	Dense	18.0		41	39
	Very dense			44	41
Sands, coarse or medium	Loose	16.5	10.0	30	30
	Medium dense			33	32
	Dense	18.5	11.5	36	33
	Very dense			39	34
Silts		16.0 to 18.0		24 to 27	
Clayey silts		17.0		21	
Silty clays	Normally-consolidated	15.0		15 to 18	
	Over-consolidated	20.0		15 to 18	
Glacial till				26 to 30	
Peat	Unloaded	11.0	1.0	0	
	After moderate loading	13.0	3.0	15	
Granite		25.0[a]			
Sandstone		22.0[a]			
Basalts and dolerites		17.5 to 27.5	11.0 to 16.0		
Shale		21.5 to 23.0	12.0 to 13.5		
Stiff to hard marl		19.0 to 23.0	10.0 to 13.5		
Limestone		27.0[a]			
Chalk		9.5 to 20.0	3.0 to 10.0		

[a] Measured in the solid, i.e. not crushed or broken.

The effects of weathering on rocks used as fill has to be considered. Softening at the points of contact of rock fragments results in a reduction of angle of shearing resistance. Complete degradation results 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. This can result in settlement.

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** of this part.

51.2 Design

The design of sheet-piled structures is covered in BS 6349-2:1988. Particular guidance is given as follows on the use of sheet-piled structures in maritime or riverine situations, where the method of construction and the flexibility of the wall has a direct effect on the earth pressures.

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.

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In the case of a cantilevered single-wall structure (Figure 45) 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. The pressure distribution in a dense cohesionless soil can therefore be taken as increasing linearly with depth after very small movements have occurred.

In the case of an anchored single-wall structure (Figure 46) 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 is developed over the full depth of the wedge, giving a linear distribution similar to that shown in Figure 47. Where no yielding of the anchor occurs the arched pressure distribution remains non-linear, as shown in Figure 48.

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. The initially high position of the resultant lateral force, though, 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 less than 0.1 % of the height of a wall in front of which dredging is carried out after completion. 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, double-wall, sheet pile retaining walls (see Figures 49 and 50). 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 is 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. Backfilling is done before the soil in front of the wall is dredged away. When this is completed the movement of the wall due to pressure from the small retained height of soil might be insufficient to develop active pressure conditions. As dredging takes place the wall yields and the pressure distribution changes from arched conditions at the upper level to the final assumed linear active condition. The initial and final stages are shown in Figure 51.

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 groundwater level is compacted, the additional yielding of the wall and anchorages might 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 is prevented and the distribution of active pressures is modified. At the toe the pressures correspond to at-rest conditions.

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.

The calculated passive resistance is not mobilized without movements that would be unacceptable in a permanent structure. It is common practice therefore to assume an effective resistance of approximately half that calculated.

When sheet piles are driven sufficiently deeply to achieve fixed earth support conditions the effective depths can 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 47).

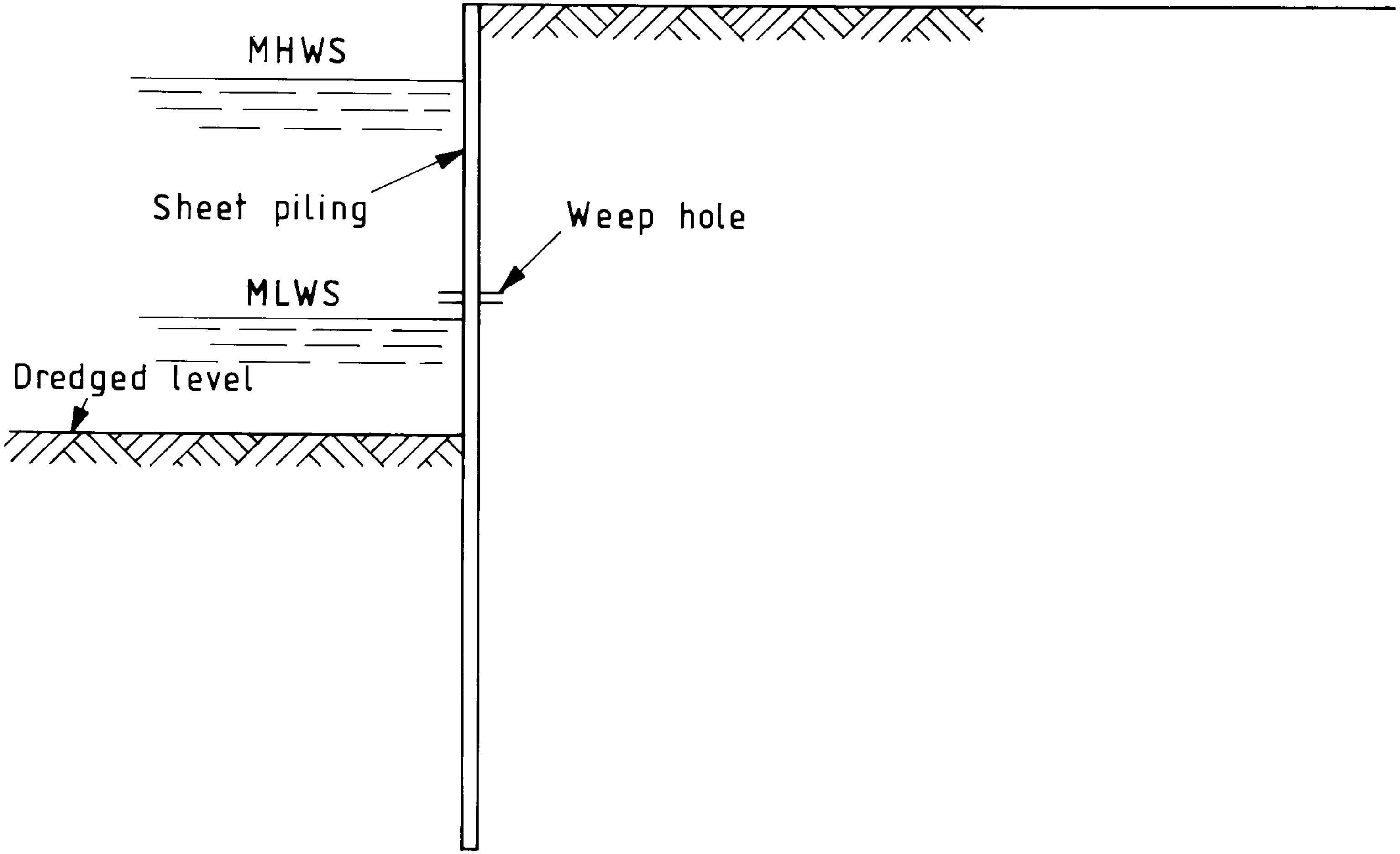

Figure 45 — Cantilevered single-wall sheet pile structure

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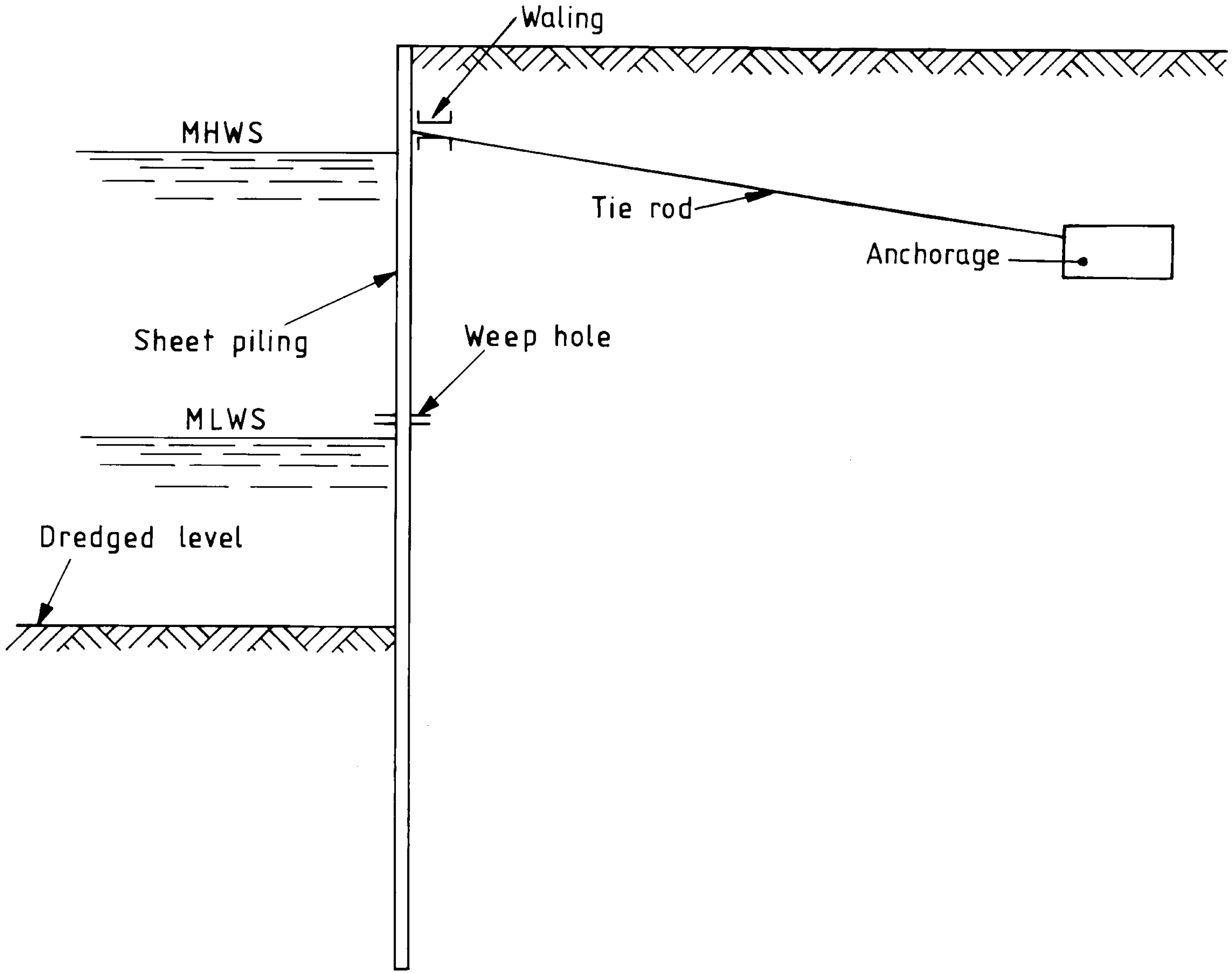

Figure 46 — Anchored single-wall sheet pile structure

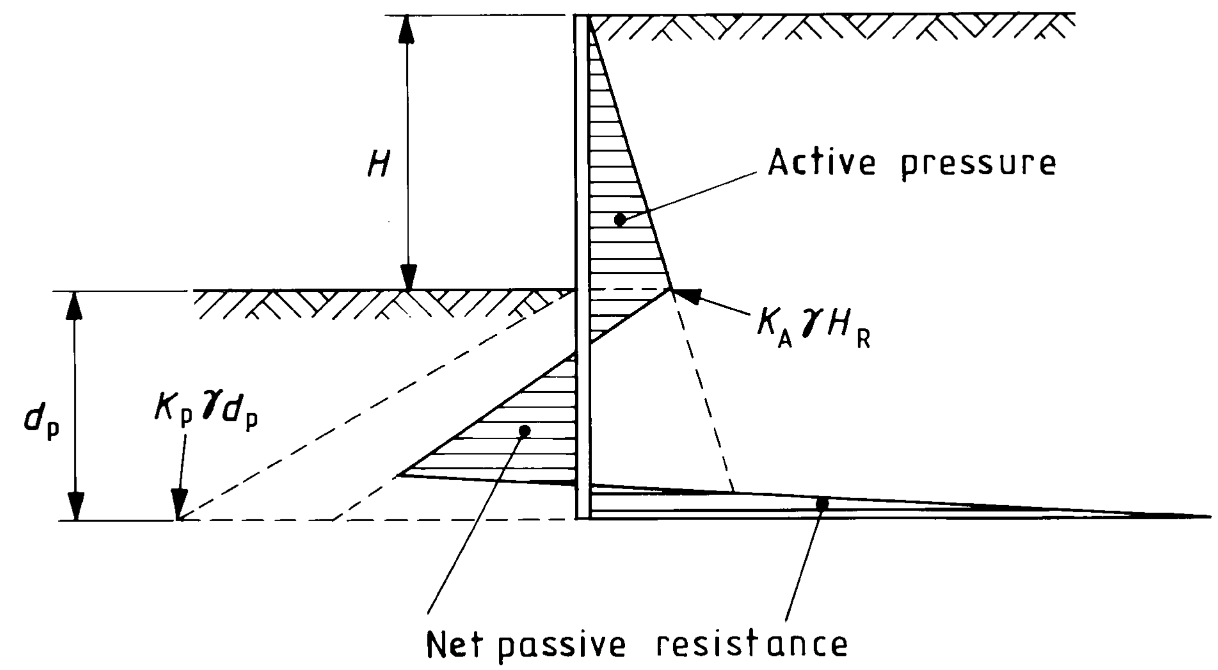

Figure 47 — Distribution of earth pressure and earth resistance on cantilevered single-wall sheet pile structure

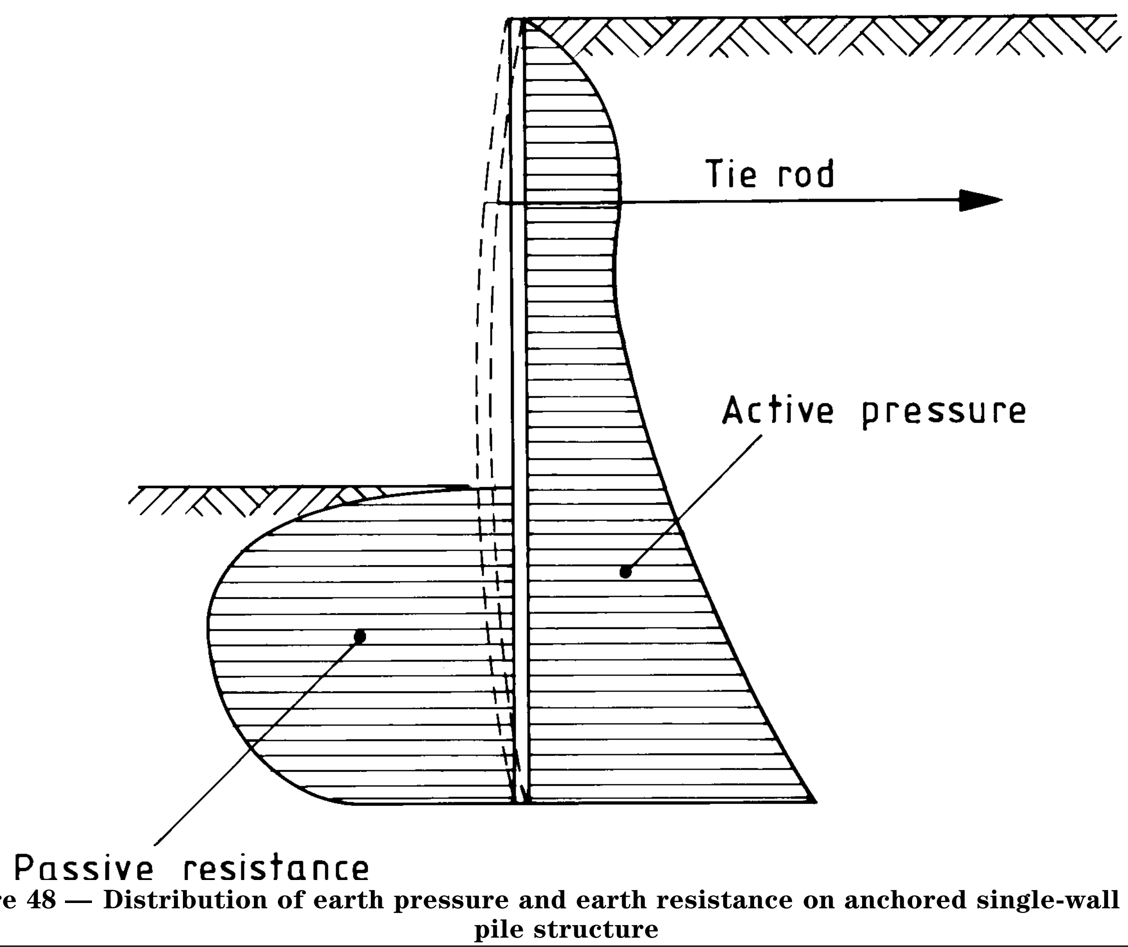

Figure 48 — Distribution of earth pressure and earth resistance on anchored single-wall sheet pile structure

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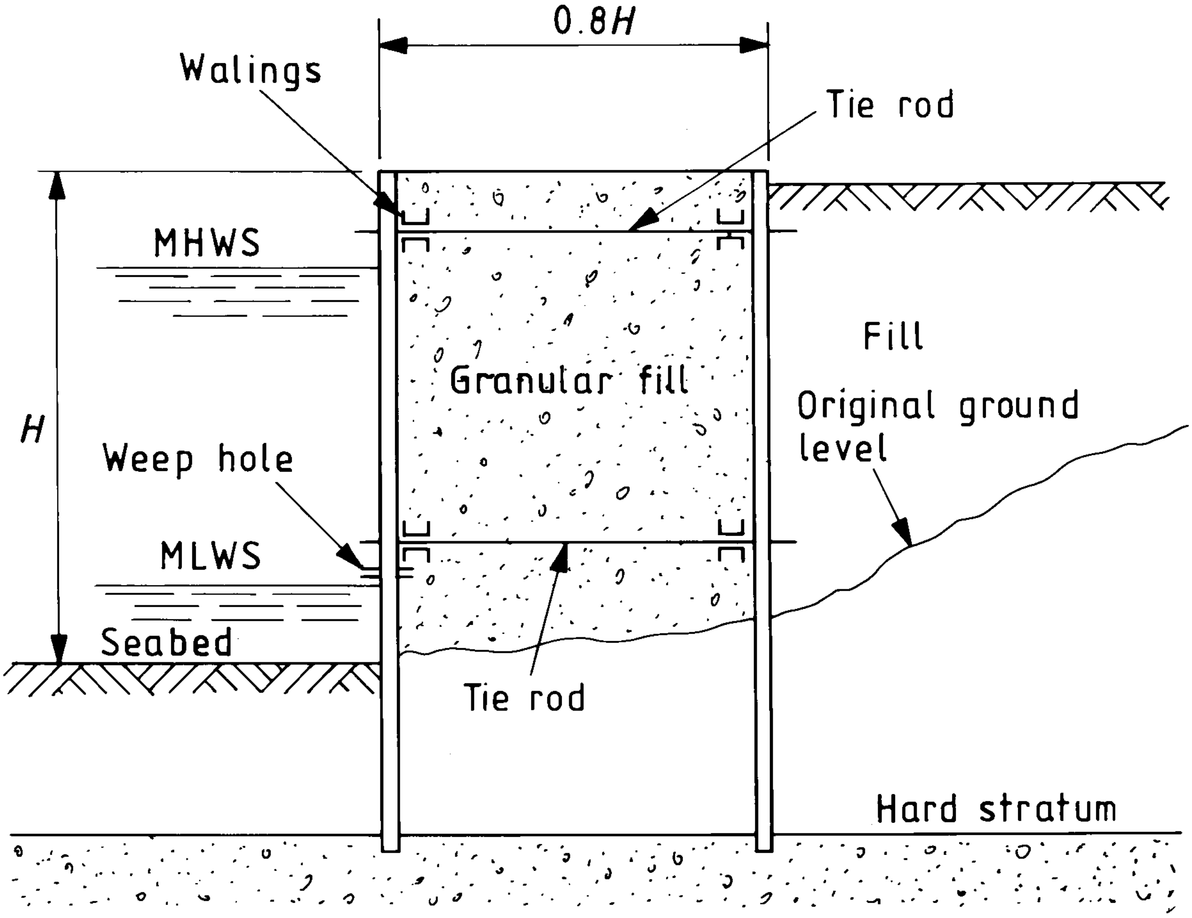


Figure 49 — Double-wall sheet pile structures — Sheet piles driven into soil below seabed

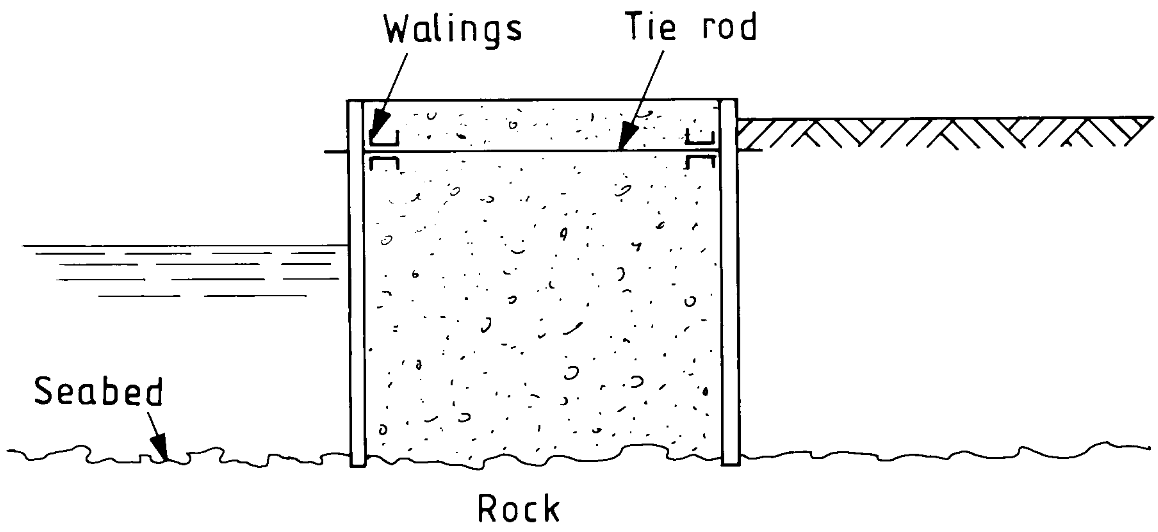
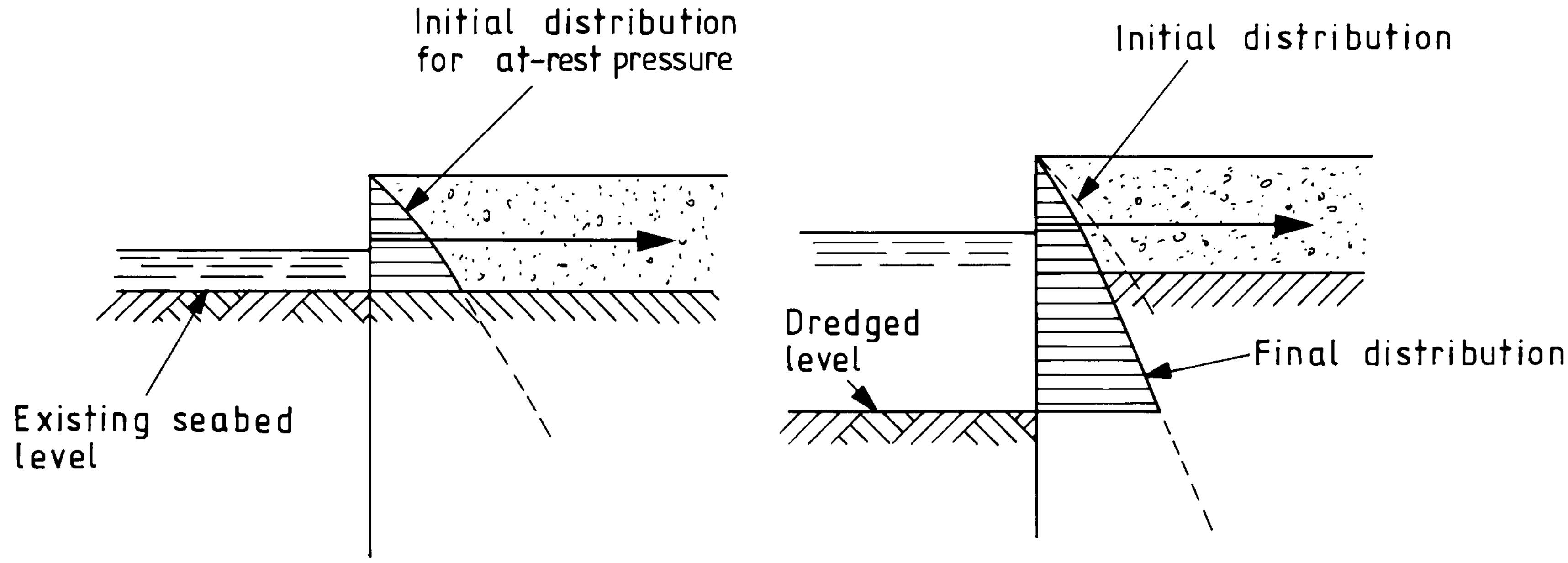


Figure 50 — Double-wall sheet pile structures — Sheet piles terminated on rock at seabed



a) Initial pressure distribution before dredging

b) Final pressure distribution after completion of dredging

Figure 51 — Active pressure distribution on anchored single-wall structure where filling is placed before dredging

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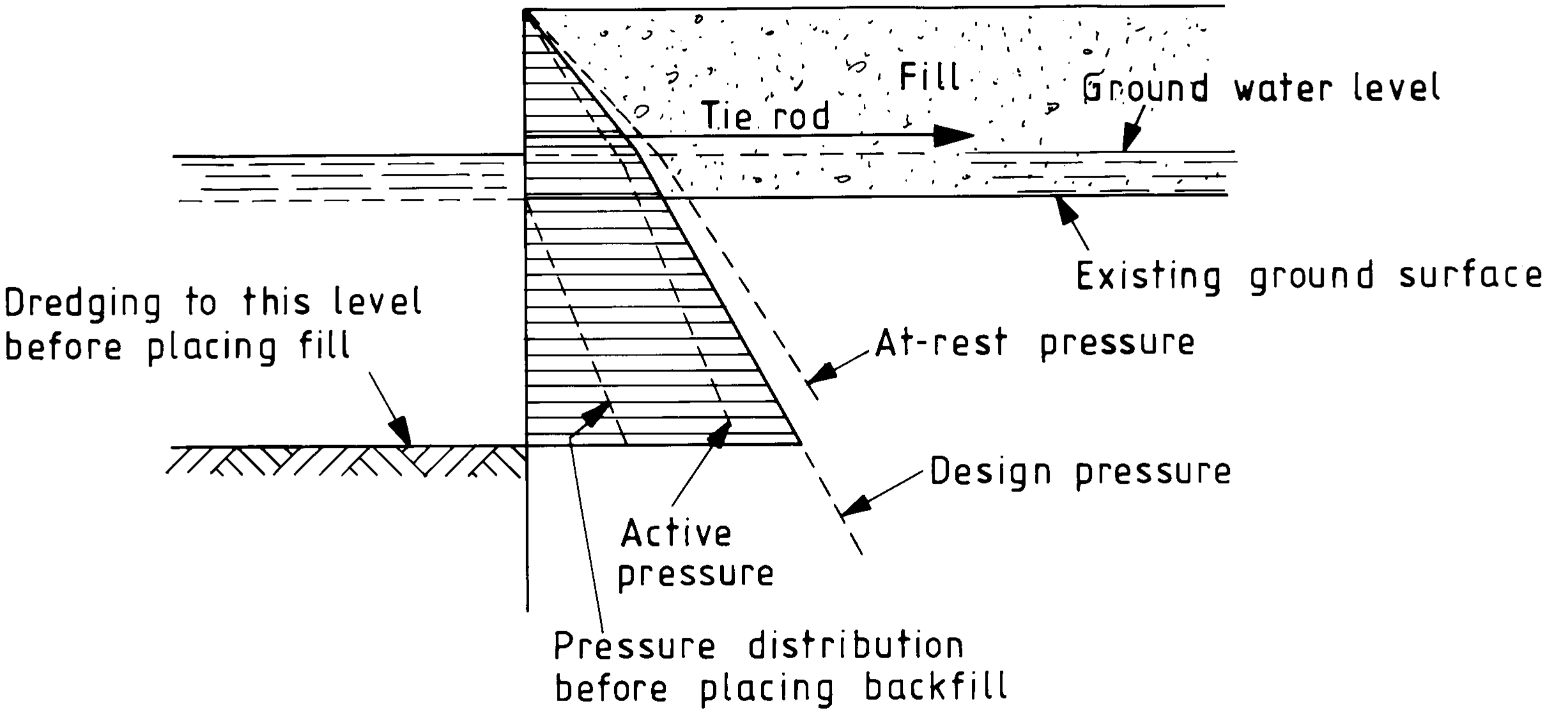

Figure 52 — Active pressure distribution on anchored single-wall structure where filling is placed after dredging

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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. Lateral earth pressures should therefore be calculated in terms of effective stresses. This is based on the assumption of zero-effective cohesion and φ' being taken as the value representative of active conditions. The pressure distribution can be taken as linear, as shown in Figure 47.

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 net passive resistance is 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 can be used to obtain the total active and passive forces as follows (see Figure 53):

$$P_A = \frac{1}{2}\gamma(H_R + d_p)^2 - 2c_u(H_R + d_p)$$

$$P_A = \frac{1}{2}\gamma d_p^2 + 2c_u d_p$$

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 should be used.

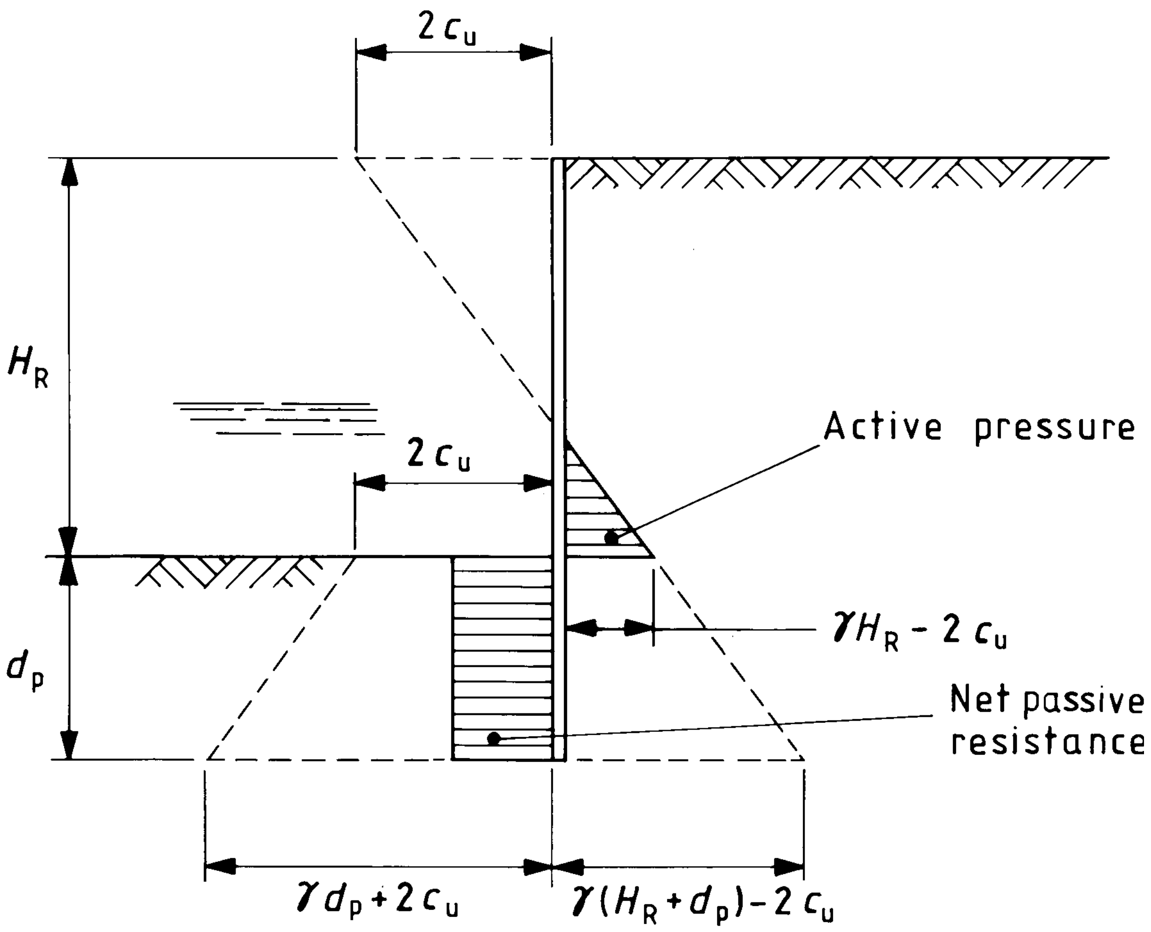

Figure 53 — Distribution of active pressure and passive resistance for total stress conditions in normally- and lightly over-consolidated clay

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51.3.3 ***Overconsolidated clays***

Dredging in front of a sheet-piled wall that is embedded in an over-consolidated clay causes the wall to deflect forwards. The clay behind the wall swells with the development of high negative pore pressures in the soil. Initially this produces 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 depends on the mass permeability of the soil (see **50.2.5**).

Similar swelling and softening of an over-consolidated clay occurs 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 can be assumed.

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 means of raising the 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.

Structures behind a retaining wall can be supported by piles driven below the potential surfaces of sliding. If they are supported in this way, pressures from the imposed loading on the piles are not transmitted to the retaining wall. Earth pressures caused by soil displacement and the effects of pore pressures in the soil require consideration, though. These pressures are retained by and beneath the wall and are induced by soil displacement as the piles are driven [38].

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 has 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.

Granular soil used to fill the cells should not be placed until the bearing piles have been driven. Bored and cast-in-place piles can be installed after completion of filling, but care might be needed to avoid the development of excessive pore pressures in any layered or laminated clays that might be below the base of the structure [39].

Superimposed loading on the deck increases 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 cause a differential hydrostatic pressure between the waterside face and the landward face of an earth-retaining structure. Fluctuations in the level of the groundwater (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 that 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 can be taken as 0.5 m as shown in Figure 54a).

51.5.1.3 *High flood flows in non-tidal rivers (weephole drainage provided)*

Where a rapid fall in level of the waterway 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 54b)]. 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. It should also be considered in relation 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 can be taken as that due to a groundwater level at mean tide level and a low tide level in the waterway. These pressures range from medium-low water (MLWS), for normal cases, to an assumed water level, down to extreme low water (ELW) (see clause **37**). For the latter case 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 extremely low tides [see Figure 54c)].

51.5.1.5 *Large tidal variations (flap valve drainage provided)*

The differential water pressure can be considered as corresponding to a groundwater level, which is 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**). For the latter case, the risks due to the effects of combined earth pressure and water pressures due to extremely low tides need to be considered [see Figure 54d)].

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 groundwater profile, as shown in Figure 55, should be considered, either 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 groundwater 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 groundwater level can 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 can be calculated from:

$$d_t = 2c_u/\gamma_s$$

where

d_t is the depth of the tension crack;

c_u is the underdrained 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 can then be taken as corresponding to the assumed groundwater 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 can exceed the drainage capacity of the weepholes or flap valves. In this 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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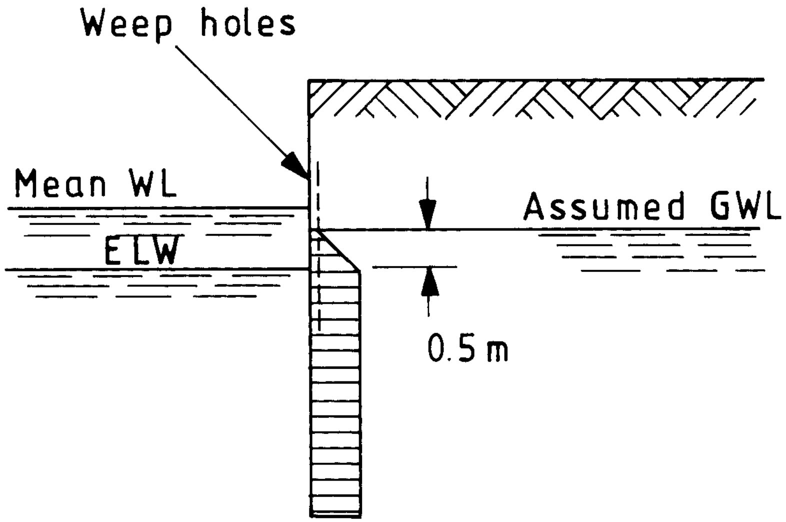
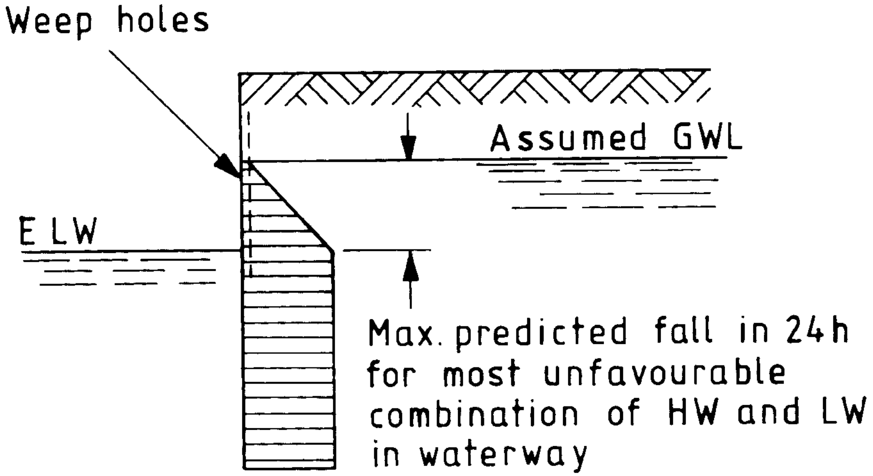
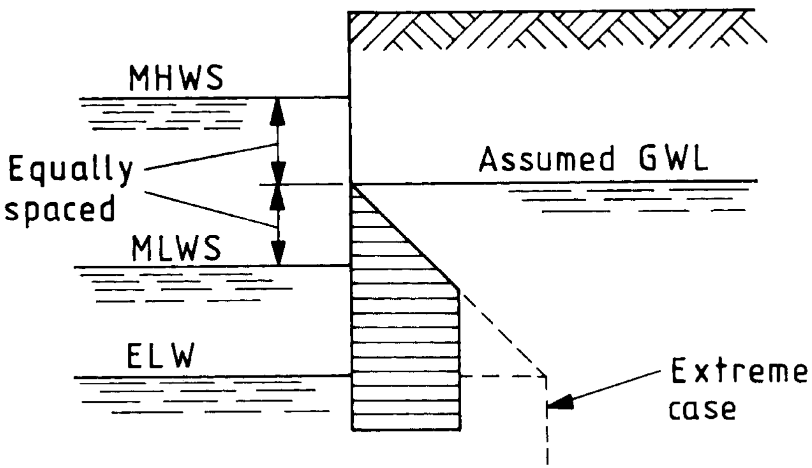


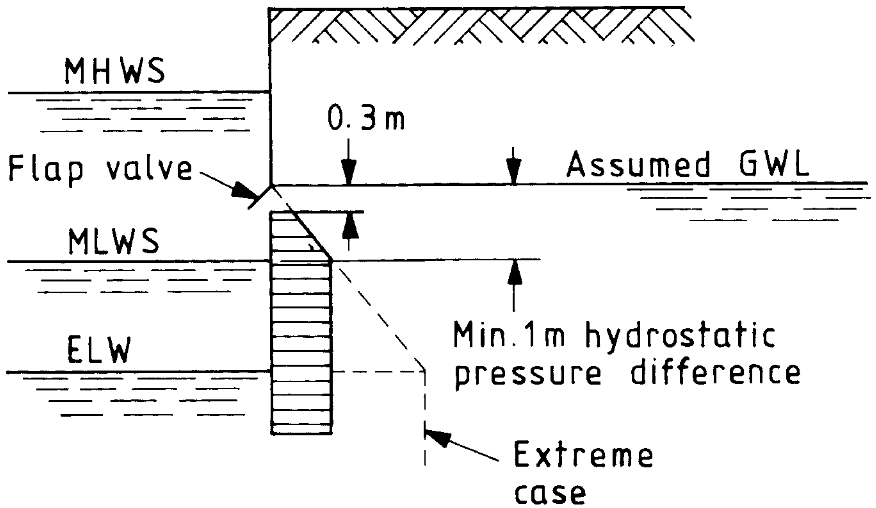
a) Minor, non-tidal



b) Large drawdown after floods



c) Tidal, no drainage



d) Tidal, with flap valves

NOTE ELW = Extreme Low Water (see **5.6**)

Figure 54 — Hydrostatic pressure distribution on waterfront structures where soil is retained to full height of structure

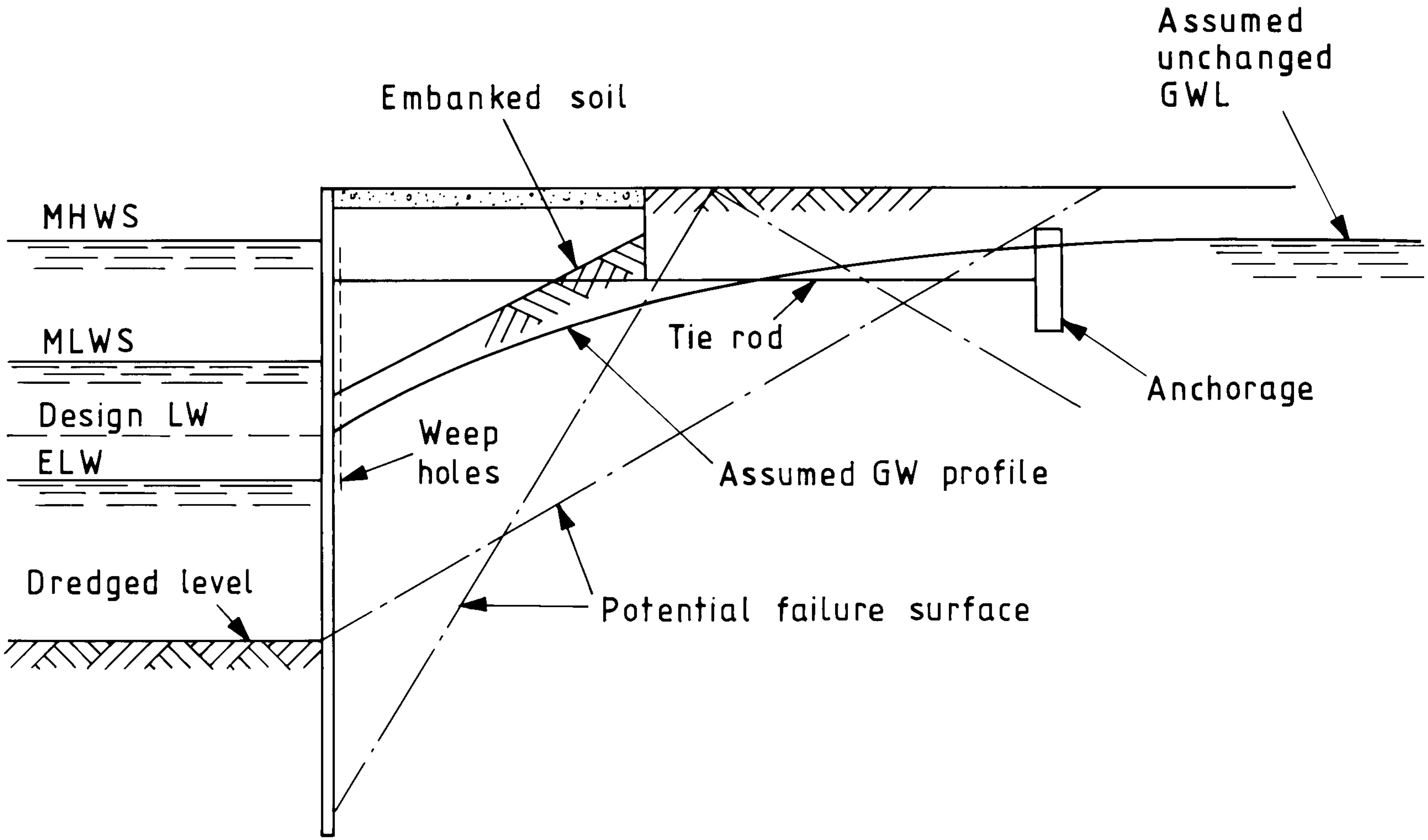


Figure 55 — Hydrostatic pressure distribution on waterfront structure where the soil is embanked behind the structure

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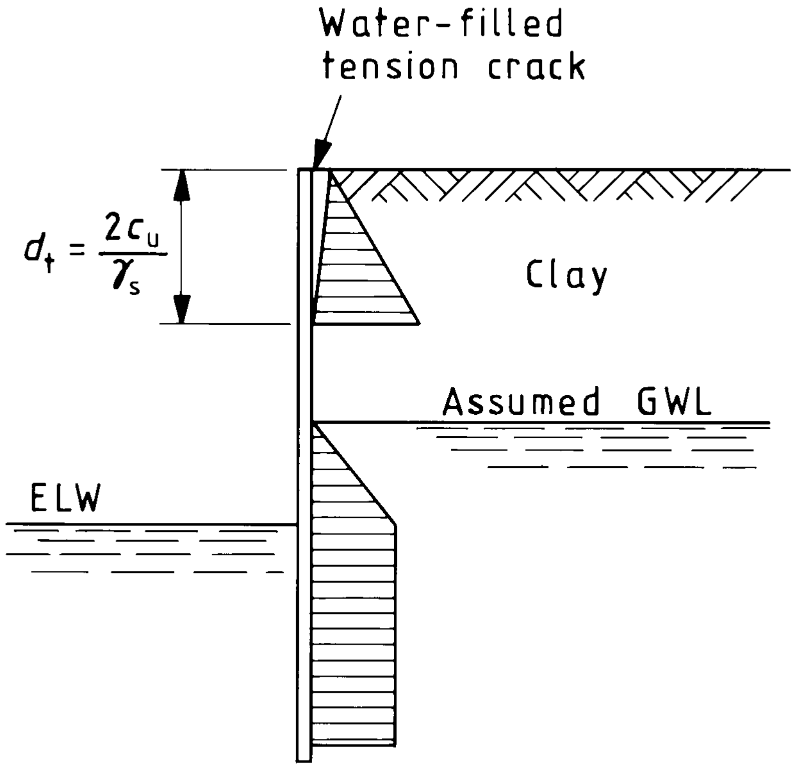

Figure 56 — Hydrostatic pressure behind waterfront structure backed by clay

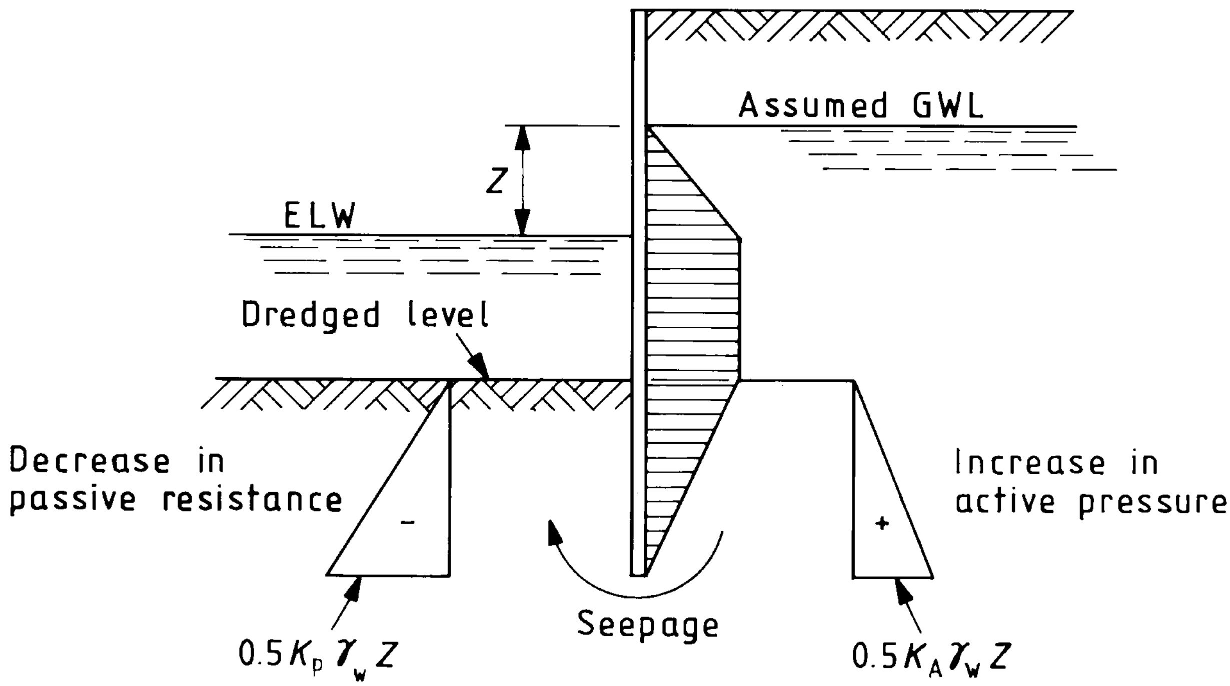

Figure 57 — Effects on hydrostatic and soil pressure distribution where seepage takes place beneath retaining structure

52 Gravity structures

52.1 General

Gravity structures can be considered with advantage:

— where skilled supervision and labour are not readily available;

— where foundation conditions are good or can be economically improved;

— where heavy impact or abrasion is likely;

— where the water is aggressive to embedded steel reinforcement.

Guidance on the design of gravity structures is given in BS 6349-2:1988.

Soil exploration should be carried out on the following:

a) the earth to be retained;

b) the ground in front of the wall;

c) the foundation layer;

d) the underlying soil layers.

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 in the UK for many years.

52.3 Plain concrete walls

The simplest form of gravity wall is plain 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 hardened concrete can be introduced into thick 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 should be kept to a minimum. 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.

52.4 Concrete blockwork

The use of precast plain concrete blockwork, with units typically each of 10 t to 80 t, provides advantages:

— when working time available on site is limited;

— when construction takes place under water;

— when the use of steel reinforcement is to be avoided.

Conditions required for this technique include the presence of firm material near foundation level, a long length of construction and the availability of heavy plant to handle the blocks. Concrete blockwork can 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.

52.5 Monoliths

In poor ground and where great depth is required, concrete monoliths can be adopted. These can be of precast blocks or cast in situ as the monolith is sunk. 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 that 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

Reinforced concrete caissons are similar in form to open topped boxes and can 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 can consist of piles or have been formed by levelling an area of the bottom. The retained ground can be formed by subsequent reclamation. After sinking of the caissons, filling of some or all cells with sand, lean concrete or other suitable material is essential to provide mass for stability.

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 might also need to be considered.

Guidance on the design of the above anchorage systems is given in BS 6349-2:1988.

Resistance to hydrostatic uplift beneath the floor of a lock chamber or dry dock can be provided by anchors that mobilize the gravitational weight of a body of ground beneath the floor (see Figure 58).

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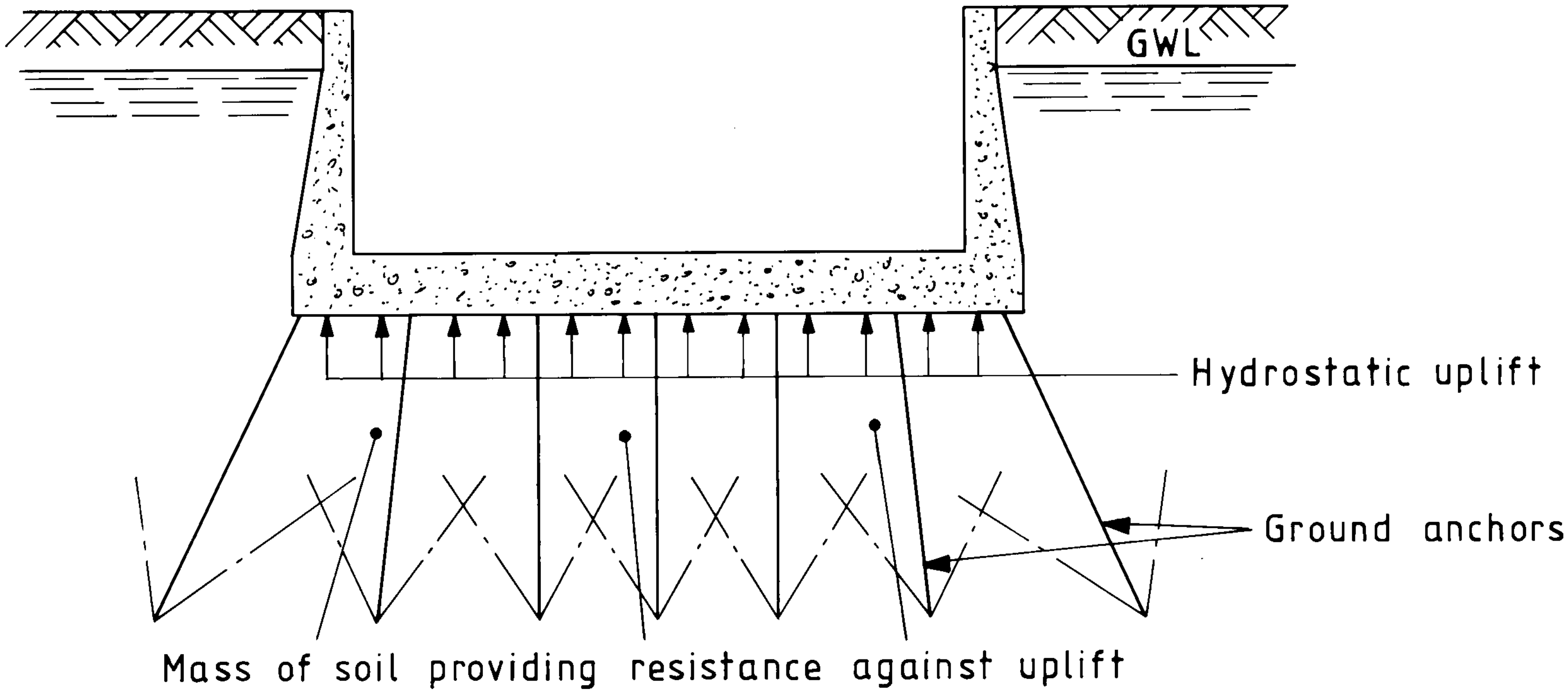

Figure 58 — Anchorage of lock or dry dock against hydrostatic uplift

53.2 Methods of anchorage

Methods of anchorage against uplift are described as follows.

Reference should be made to BS 6349-2:1988 for guidance on the design of anchorage systems for horizontal forces.

53.2.1 *Injected tendons*

Injected tendon anchors can 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.

Reference should be made to **2.5.4** of BS 6349-3:1988 for the need for caution in using highly stressed steel tendons with respect to brittle failure.

Injected tendon anchors can be used to restrain uplift forces on the underside of the floors of lock chambers, dry docks or other below ground structures. The tendons can be installed vertically or as a combination of vertical and inclined members (see Figure 58). 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 can be considered to act as a cone with its apex at the tip of the anchor (see Figure 59). The half angle of the cone depends 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 Figures 60 and 61). 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.

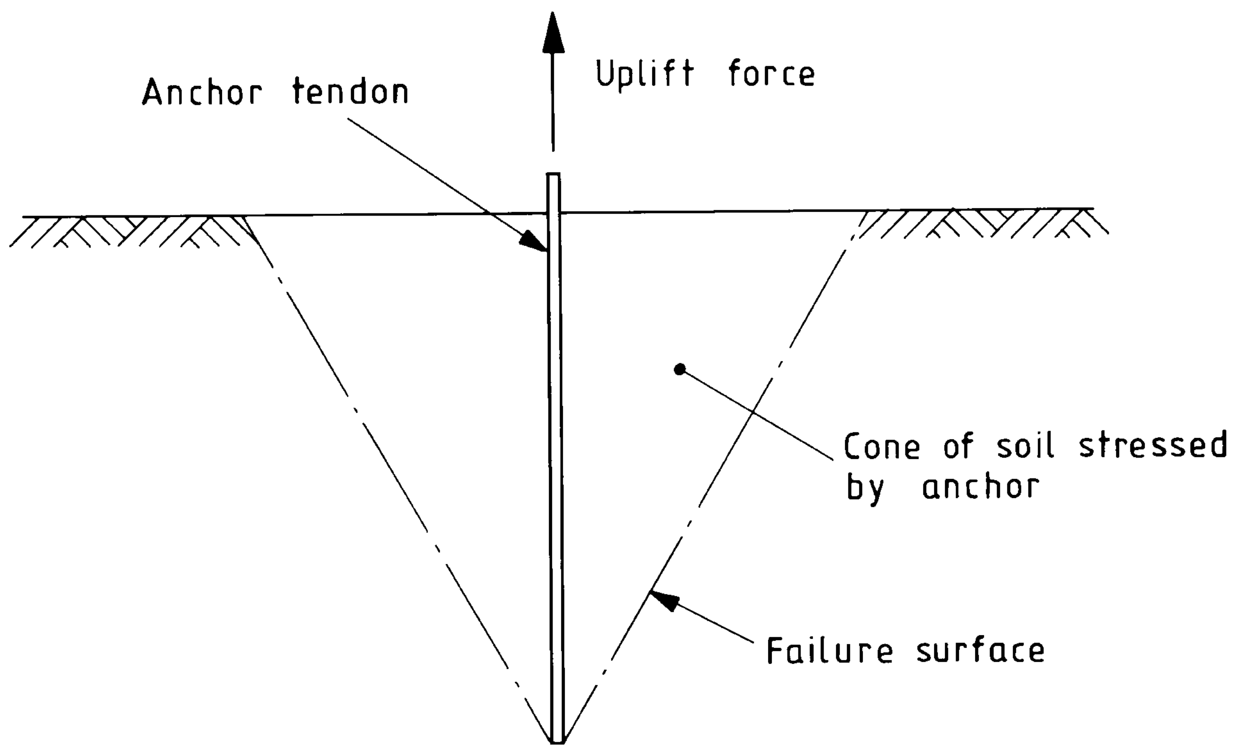

Figure 59 — Failure of vertical anchor tendon due to uplift of cone of soil or rock

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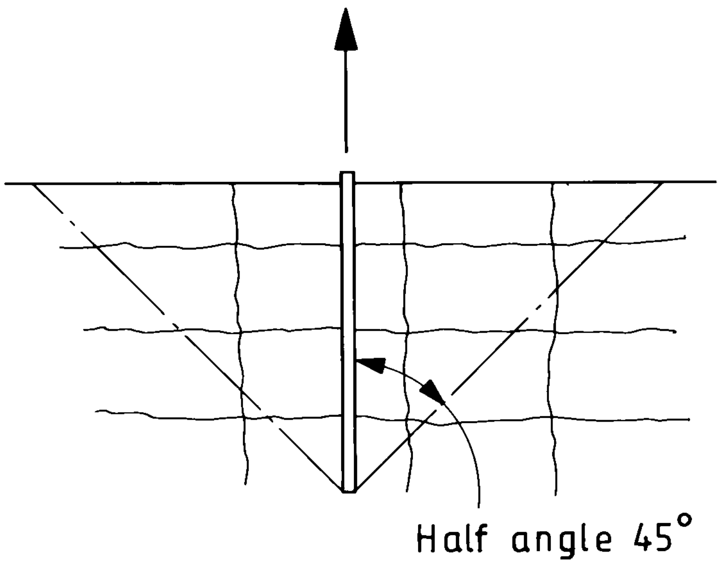

Figure 60 — Modes of failure of anchor tendons in rock — Massively bedded rock

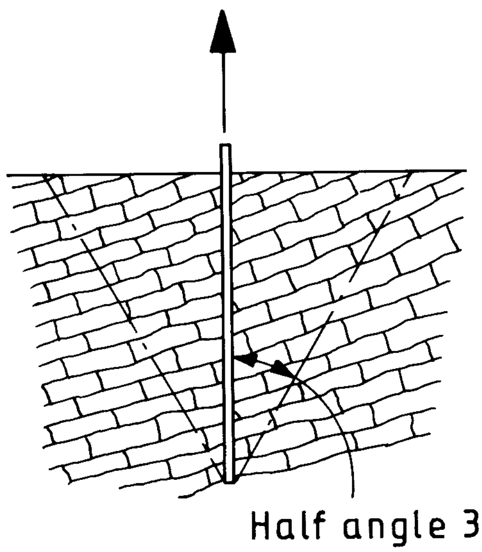

Figure 61 — Modes of failure of anchor tendons in rock — Thinly bedded rock with horizontal or near-horizontal bedding joints

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 can occur where the floor is underlain by soft clay.

In selecting soil parameters for the design of injected tendon anchors to resist vertical uplift forces, consideration should be given to the effects of cyclic loading on the soil. Cyclic loading can be caused by variations in hydrostatic pressure 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 BS 8081:1989.

53.2.2 *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 can 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 shipload 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 BS 8004. 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.

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 that 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 retractable mandrel or by screwing a spiral plate into the soil.

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 can be used to calculate the uplift resistance [40].

The effects of cyclic 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.

Reference should be made to BS 6349-3:1988 for the need for most careful consideration before adopting such types of anchors, in view of previous failures.

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:

$$R = c' + (\sigma - P_u) \tan \varphi'$$

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 that can be mobilized on the potential plane of sliding.

Consideration should be given to the effect of tidal variations on pore pressures in the soil behind the slope. In the case of partly submerged slopes, the effects of variation in level of the groundwater from landward sources should also be considered.

The fabric of the soil, which is the presence of fissures, layers or laminations of permeable soil interbedded with impermeable soils, has an effect on variations of pore pressure. The way that these discontinuities provide a means of drainage from soil and 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 should be given to numerical analyses to determine the wave-induced pore pressures and effective stresses [41].

Increase in pore pressure in the soil behind a slope can 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 the gradient of a slope becoming steeper should be considered. Steepening can be the result of erosion of the toe of a slope by 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 upwards surge of waves.

Steepening of the upper part of the slope can result from material being dumped at the crest, or, alternatively, soil being deposited by the natural processes of accretion. The effect of this steepening should be considered.

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.

Special consideration should be given to large-scale instability, which is caused by flow slides on a regional scale. These flow slides can be in loosely deposited littoral sands or in deltaic deposits. 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 62) 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 63). Rotational slides can be non-circular in form in

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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 64).

Rotational sliding also takes place where an embankment or structure is placed on weak soil that is unable to support the imposed loading (see Figure 65).

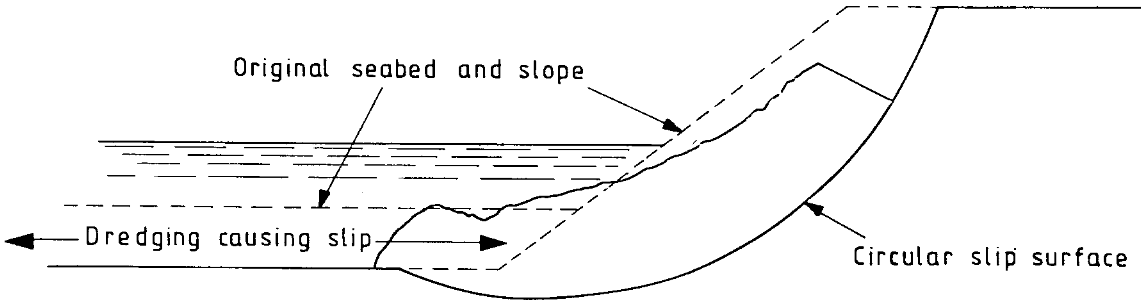

Figure 62 — Types of slope failure — Rotational shear slide on circular sliding surface

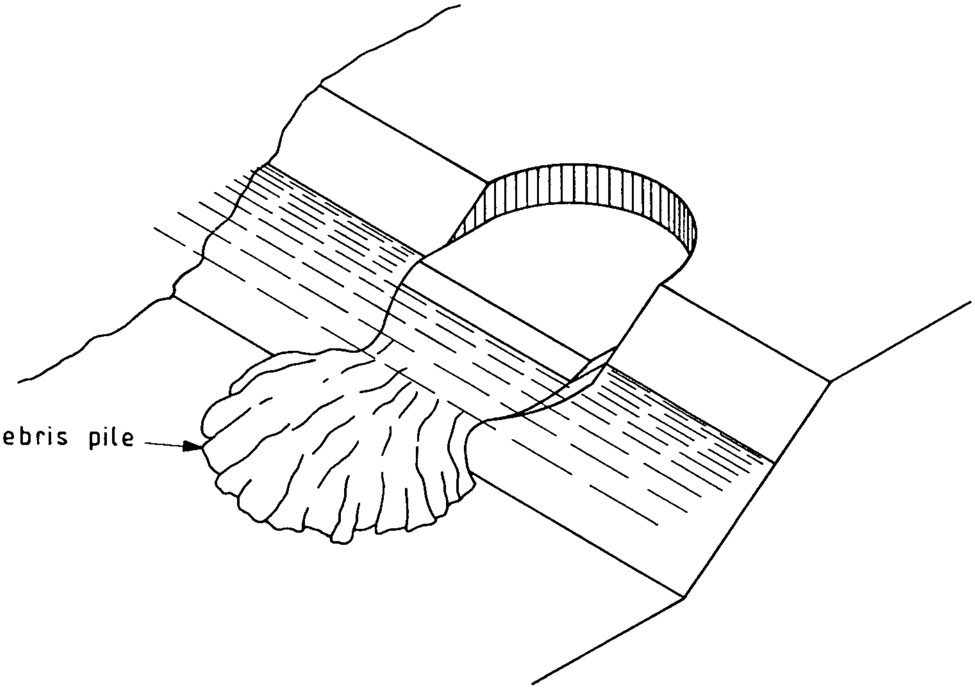

Figure 63 — Types of slope failure — Three-dimensional rotational slide

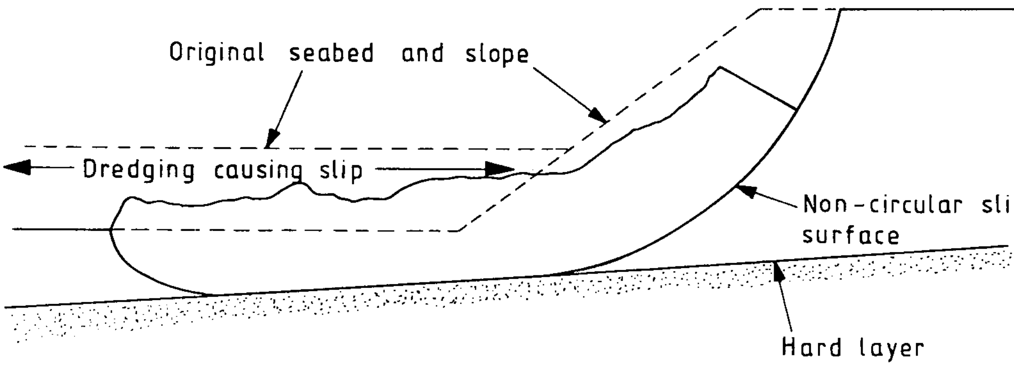

Figure 64 — Types of slope failure — Non-circular rotational slide

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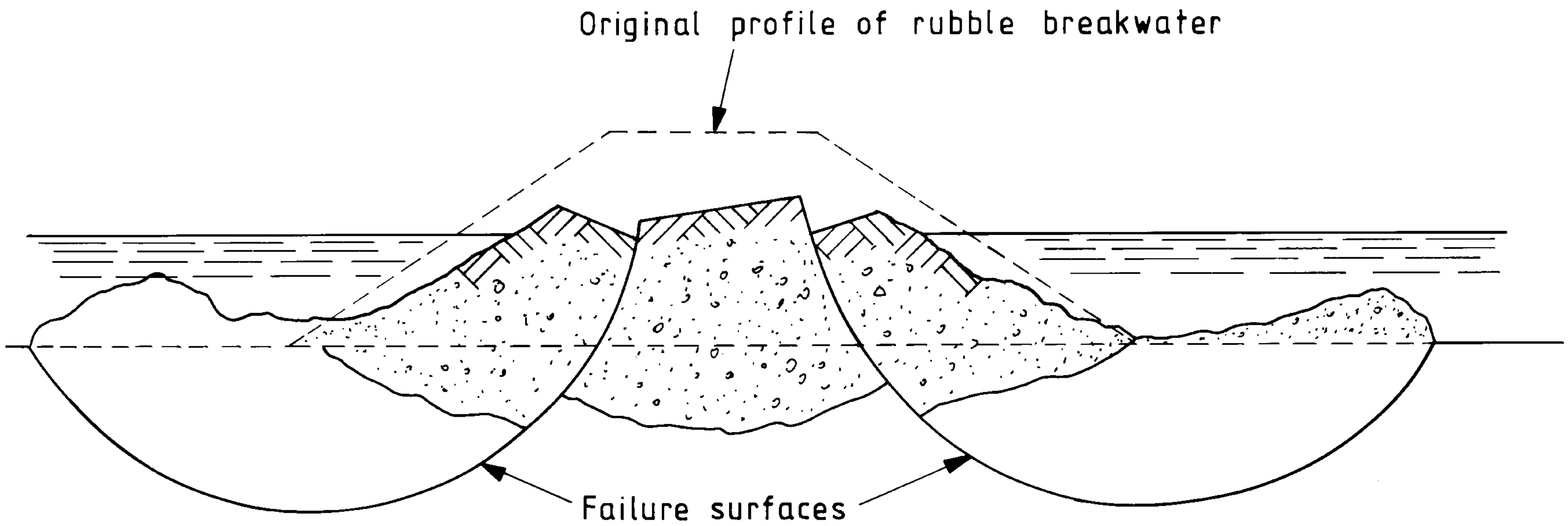

Figure 65 — Failure of embankment due to shear failure of supporting soil

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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 might 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 66) or failure can 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 slides are forms of translational movement where the sliding mass remains more or less intact. A slab slide (see Figure 42) 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 the 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 that intersect behind the slope (see Figure 43).

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 move down the slope in a random, unsorted form. This movement is either slow in a creep movement, or rapid at times of saturation of a slope caused by a sudden surge of high waves, or by heavy rainfall, or as a result of surface water being diverted 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. The latter happens because local steepening by over-dredging or erosion causes failure. 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 44). They can 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 back into the slope until undercutting provokes a collapse. The collapsed materials then move down the slope in the form of a debris flow.

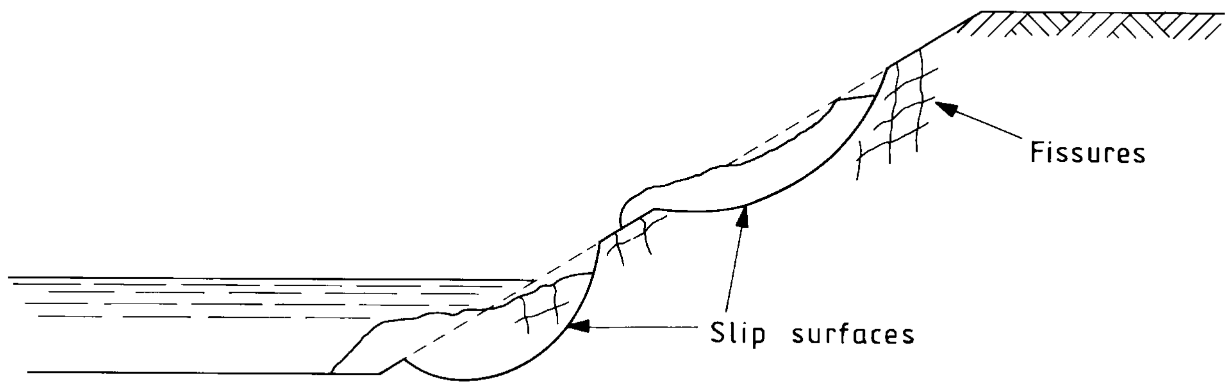

Figure 66 — Compound slide

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 the effects of variations in pore pressure within the soil mass caused by the factors already described (see **54.1.1**) and 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 needs to be considered and the stability analyses, both in cohesionless and cohesive soils, should be made in terms of effective stresses. Stability analyses 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 also be controlled. This is to allow time for the pore pressures to dissipate, which results in strengthening of the soil.

The possibility of local slips or falls occurring on the face should be considered when preparing designs for the alignment and profile of a slope that has been formed by dredging or by placing material to form an embankment. The overall stability against the various forms of failure described in **54.2** should also be considered.

Local slips or falls can 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 mattresses (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.

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 can 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.

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. It might be desirable to choose angles flatter than those required as a minimum factor of safety, in order to avoid frequent maintenance dredging, or to meet aesthetic criteria for above-water slopes.

Typical underwater slopes for various soil types are given in BS 6349-5:1991, Table 11.

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 can be varied to conform to the engineering behaviour of each formation. Slope angles can also be varied in previously water-bearing soils by adopting a steep slope approaching the angle of repose of the soil located above the highest groundwater level. Alternatively, 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 can be adopted.

Where steep upper slopes are adopted, consideration should be given to the overall stability of the earthworks. Where necessary a berm should be introduced between the two differing slope profiles.

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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 can 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. This is 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 by two considerations. The first is the factor of safety against failure in the underlying soil and of differential water pressure within and on each side of the embankment. The second is the need to avoid erosion and overtopping of the structure by wave action. Guidance on the design of breakwaters is given in BS 6349-7 and [47].

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 usual 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. This could cause instability or the formation of mud waves at the toe of the slope that could cause problems in removing the heaved soil from below water level.

Embankments and training walls can 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 might 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 67). Where the outer protective embankments are placed in successive stages as shown in Figure 67, the height of each stage should be controlled to prevent the formation of mud waves that could become trapped within the core material and cause instability of the embankment.

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 above-water 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. It is good practice, however, to make periodic inspections during construction and in the early months following completion, when grassing or other methods used to control erosion in above-water 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 can 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 can 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 can 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.

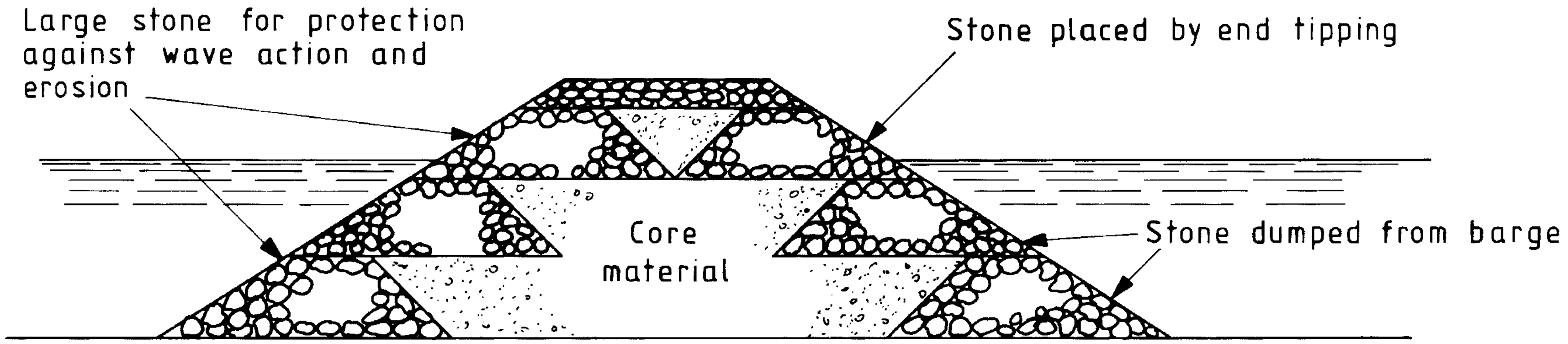

Figure 67 — Embankment built in stages with core material protected by dumped stone

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Inspections 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 might 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 might 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 against erosion by currents, waves, surface and subsoil water.

54.5.2 *Underwater slopes*

Disturbance and transport of soil particles by flowing water (see clause **14**) causes erosion of the soil on the seabed. Erosion can 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 propeller wash. Bow thrusters can pose particular problems. 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 might be needed in these areas if the currents are strong. Guidance on the design of anti-scour aprons is given in BS 6349-7.

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 might 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 means could be, for example, 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 is designed so that the finest filter material does not move into the adjacent coarser filter layer under the influence of flowing water. Alternatively, the filter can consist of other materials, including geotextile mesh or brushwood mats protected by a layer of crushed and graded stone and then the stone or precast concrete block armouring (see Figure 68). Guidance on the design of single or multi-layer filters is given elsewhere [47] [48].

Ground surfaces beyond the influence of waves or tidal water movements can 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.

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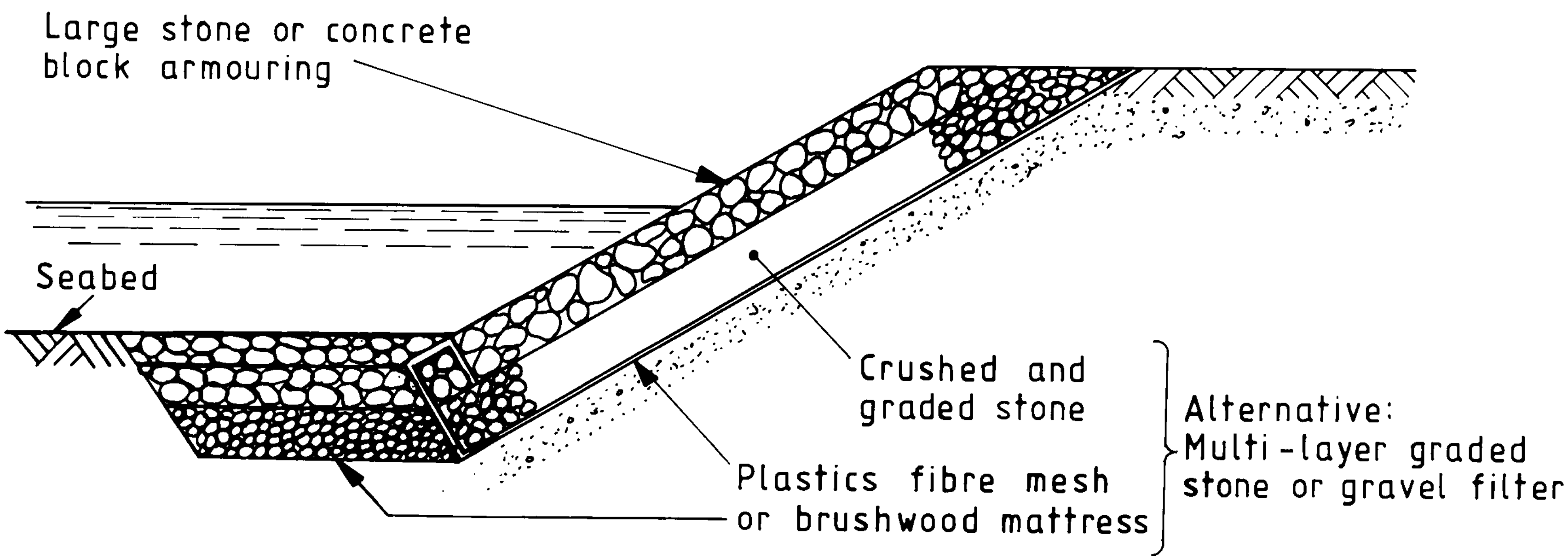

Figure 68 — Slope protection by rock or concrete armouring backed by filter layer

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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 has 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 difficult and not usually economic. 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 depends 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 counterweight 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.

Where scouring of seabed material from the toe of a slope causes a rotational shear slide 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 might be a feasible remedial treatment for submerged or partly submerged slopes.

Consideration should 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 that could result in instability of slopes.

55 Use of thixotropic liquids in excavations

55.1 Types of structure

Thixotropic liquids, referred to as bentonite mud (although strictly speaking it is a slurry), are used as a means of stabilizing excavations in weak ground for shoreside structures such as quay walls and lock chambers, thereby obviating 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 mud. Concrete is poured through a tremie pipe, displacing the mud, which is then reused or discarded. This form of construction is often referred to as diaphragm walling. It is discussed further in BS 6349-2:1988.

Substructures for quay walls and locks can be a series of rigid gravity walls in box form, or counterforts. These are continuous facing walls forming the waterside or berthing face and a series of retaining walls projecting from the landward side of the wall. Quay walls can 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, can be constructed on rectangular or cruciform trenches excavated with the support of bentonite mud.

Further information on the various structural forms that are in common use is given in **4.4.4** of BS 6349-2:1988.

Bentonite mud can 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 groundwater table. Guide walls are constructed to retain the mud 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 mud 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 can 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**).

The concreting of 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 can 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 mud

The vertical face of a trench excavated under bentonite mud is supported by the hydrostatic head of mud above groundwater level. The general principles that govern the stability of slurry-supported trenches are reviewed elsewhere [49]. It is essential to maintain a head of bentonite mud in the trench above the level of the groundwater table with an adequate margin of safety to provide for a rise in groundwater level at the time of high tides or surges.

Provision should also be made for temporary lowering of the mud level due to overbreak in the excavation, or loss of fluid to seepage, either through the soil or through interstices in open gravel or fill material.

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. A period of six to eight hours after concreting has been completed is usually sufficient.

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 head of mud above groundwater and the nature of the ground itself govern the stability of the trench excavation. Consideration should also be given to the dimensions of the excavating equipment in relation to the length and width of the panel.

It is necessary to maintain a tight control of bentonite mud quality, particularly prior to concreting. The key control parameters are viscosity, sand content, mud density, filter cake and filter loss.

In permeable soils, the quality of the bentonite has to be sufficient to ensure the formation of a suitable filter cake at the sides of the trench. This inhibits water loss from the trench, so that the full head of mud (above groundwater level), and therefore the stability of the trench, is maintained.

The viscosity of the mud should not restrict concrete flow at the base of the panel during concreting.

The sand and silt content should be reduced to acceptable limits prior to concreting; the thixotropic quality of the bentonite mud should be sufficient to keep remaining particles in suspension. A suitable density difference between the concrete and mud is required to avoid mixing and contamination during concreting.

Care is required, particularly when constructing load-bearing walls, to avoid contamination of the concrete by thickened mud at the base of the panel.

In unstable ground, it might be necessary to substitute the mud with a self-hardening slurry or lean mix concrete to stabilize the trench. When this has achieved sufficient strength, excavation can re-commence utilizing bentonite mud to support the trench.

Periodic checks should be made on the density and other properties of the mud. This is to ensure that it does not become flocculated, or excessively diluted by groundwater, or contaminated by soil particles. The result of these phenomena could be instability in the trench or deterioration 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 mud should conform to 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 can be obtained from the Federation of Piling Specialists, 39 Upper ELmer Road, Beckenham, Kent, BR3 3QY.

Attention should be paid to the effect of saline water on the properties of the mud and where necessary special forms of bentonite or processing methods should be used to prevent undesirable flocculation of the mud in the trench.

Measures should be taken to prevent discharge of used mud or uncontrolled escape of mud from the trench to adjacent waterways, watercourses or sewers where these discharges would result in pollution or blockage of drainage systems.

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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 normally considered to be materials in their own right.

Some elements of maritime construction, such as pavements and piling, which can utilize a variety of materials, are also included, because the special requirements of the maritime situation have a bearing on the choice of materials that 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) rails;

i) bituminous materials.

Protective measures and treatments cover a wide range of methods that 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 with good durability. It should be free from laminations and weak cleavage planes and should be of such a character that it would be resistant to disintegration or erosion 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 normally 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 that are usually the most durable. Sedimentary and metamorphic rocks require more care in selection but the best sources can provide suitable materials. Whenever possible, an engineering geologist should be consulted.

Further guidance on the selection of rock is given in [47].

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 as follows are intended for the primary armour stone and suggested limiting values are given. Lower limits can 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 more than 30 % for standard test fraction.

d) *Ten per cent fines*. Value should be not less than 100 kN.

e) *Soundness*. Loss in mass after 5 cycles should be not more than 12 % for sodium sulfate or 18 % for magnesium sulfate, by the test methods given in BS EN 1367-2 . Where necessary, a representative sample of large stone should be crushed to provide sizes acceptable for testing by this procedure. If samples do not exhibit microfractures and show the rock to be isotropic, the test sample size can be reduced to 10–20 mm for reasons of economy [47].

f) *Aggregate abrasion value*. Value should be not more than 15 %.

g) *Block integrity*. Value should be not more than 5 % when subjected to the drop test specified in Appendix 2 of [47].

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.

57.3.2 ***Single-sized materials***

Single-sized materials normally 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 mass. At least 75 % by mass 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 to require individual stones to be 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 stones are $3.6m_{50}$ and $0.22m_{50}$, respectively.

Bedding and filter materials are required to prevent the scouring of one material through another. 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}. For design of filters see [47].

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

The environmental conditions affecting maritime structures are usually much more severe than for land-based structures. Consequently, the specification for concrete, in terms of both materials and workmanship, should focus on the concrete being constructible and durable, as opposed to having structural strength alone. The same factors apply to design and detailing.

The structural design of the elements of maritime structures, in the UK, has hitherto been carried out in accordance with the principles of BS 8110-1, BS 5400-2 and BS 8007.

The general principles for specification, production and assessment of compliance of concrete, as a material, are set out in BS 5328.

At the time of writing, however, all of these documents are in the process of revision or replacement by European Standards and National Application Documents, for example, DD ENV 1991-1:1994, Eurocode 1 and DD ENV 1992-1-1:1992, Eurocode 2.

All of the documents include recommendations and/or mandatory provisions for concrete in structures in a seawater environment. Designers and constructors should be aware of these general codes and of the latest published authoritative advice on the effects of sulfates, chlorides and general deterioration processes. Nevertheless, the recommendations of the codes and standards mentioned are replaced by the recommendations of **58.1** to **58.4** of this document where they apply, in the case of maritime and estuarine structures.

This clause applies primarily to concrete structures in the UK. Many maritime and estuarine structures can be classed as "special structures" and demand specific requirements other than those prescribed under general building codes. For this reason it is necessary to maintain an overview of environmental conditions of exposure (such as conditions of aridity and higher or lower temperatures). This is particularly important if this code of practice is used outside the UK where ambient conditions can be more severe.

Consideration should be given to environmental conditions at all stages of construction, as well as for the completed structure. During construction, maritime works are particularly sensitive to adverse weather conditions, which might hinder access to the works, prevent the use of floating plant and cause damage to work both above and below high water level. Weather conditions can limit construction activity to certain "seasons" or "windows" and can affect various transient load conditions such as towing, sinking and grounding of floating elements.

58.2 Type of construction

Some of the factors affecting design and/or the method of construction are as follows.

a) The work might be permanently underwater where access is difficult and visibility is negligible.

b) The work might be within reach of waves at every tide and therefore subject to wave action and scour and contamination by seawater.

c) The periods between tides in which the work is accessible might be very short and it might 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 they can be formed by simple shuttering, which can be easily fixed and is grout-tight, rigid and strong. Complicated shapes should be avoided, as should thin cross-sections in which the cover to reinforcement is sensitive to the accuracy of steel fixing.

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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, the specification of practical tolerances, and ensuring that they can be achieved are important.

g) Thick sections and massive structures built in separate pours create restraint to shrinkage during cooling from the elevated temperature resulting from the heat generated by the hydration of cement. This can cause early thermal cracking [50]. Structural members and joint spacing should be designed to limit early thermal cracking and/or steel reinforcement provided in order to control the size and spacing of cracks.

h) Erosion of concrete due to abrasion can be a cause of serious damage to maritime structures. It can occur at water level with floating objects or at bed level where beach materials can continually be washed against the structure.

i) In cold climates, freezing and thawing and impact of ice can affect concrete adversely.

j) The concrete can suffer chemical attack.

58.3 Durability

58.3.1 ***General***

Design for durability of concrete in maritime structures (which can include elements of buildings, bridges or tunnels in coastal locations) is dependent upon the recognition of the specific exposure conditions that affect the various elements of a structure, and the adoption of appropriate design, detailing, materials and workmanship to suit these conditions. The specification of the concrete materials and details, such as location of construction joints and cover to reinforcement or prestressing steel (if any) is an integral part of the design process. It is not prudent to produce structural design and details without parallel design and selection of materials and details, such as cover, relating to durability.

The maritime environment can be very aggressive to the concrete itself (i.e. the mixture of aggregates and cement paste) in terms of physical weathering, abrasion and chemical attack. It can also damage embedded metal or reinforcement as regards corrosion. The assessment of durability in conjunction with maintenance strategy is a fundamental part of the design process.

The durability of concrete is governed primarily by the pore structure of the concrete, its permeation characteristics and chemical nature, which are dependent on appropriate mix design, a sufficiently low water-cement ratio and appropriate cementitious materials together with selection of cover to reinforcement appropriate to the mix and the specific cement type.

Durability is not, in itself, a limit state but is the means by which the structural limit states are maintained for the lifetime of the structure. Being inherently time-related, design for durability is directly related to the intended operational life and maintenance strategy for the structure.

The phrase "design working life" is explained in clause **16**, with reference to DD ENV 1991-1:1994, Eurocode 1. The definition is similar to that for "service life" in BS 7543.

It should be noted that most current design codes and standards do not provide a rational framework to design concrete for specific life periods, but deal with durability only by prescriptive means. For concrete maritime structures it is recommended that, where appropriate, systematic durability design methods are adopted as a basis for achieving specific required design working lives. Appropriate factors of safety should be applied to the method. In order for the "design life" (see BS 7543) to meet the requirements of the "design working life", the former should exceed the latter by a rational margin.

In a systematic durability design method, the designer should categorize elements of the structure in relation to the severity of exposure, the performance required for the elements and the maintenance strategy to be employed. In addition to an acceptable "model" of the deterioration process, the designer should consider the limits to be defined for durability failure. In the case of reinforcement corrosion, durability failure can include any or all of the following, leading to either failure in serviceability or the ultimate limit state:

— onset of corrosion;

— rate of propagation of corrosion;

— cracking resulting from reinforcement corrosion;

— spalling and/or loss of steel and/or concrete section.

These considerations require assessment of the consequences of deterioration to the type of structure and the capability for repair or replacement at any interval within the required design working life. The acceptable risks and the likely probability of failure should be considered as these affect the design choices.

Systematic (or explicit) design methods are developing, but are still at the transition stage. As a result the specification has almost certainly to be expressed in prescriptive terms, i.e. with mix limitations for concrete properties and cover to reinforcement, all selected from tabulated values.

58.3.2 ***Deterioration processes***

Design, specification and detailing with the objective of achieving a specific design working life within a range of probabilities requires a knowledge of the various deterioration mechanisms for concrete materials and recommended analytical models of the processes. Explanation and guidance on the different mechanisms are given in various publications (see [51] [52] [53]).

There are a number of deterioration processes that are specifically recognized in relation to concrete in or adjacent to seawater. The relative significance of various processes depends upon the macroclimate, i.e. the geographical location of the site, and on the microclimate, i.e. the position of a structural element in relation to the fluctuating water level.

Most of the time-related deterioration processes for concrete are covered by guidance in general codes and publications. The most serious process affecting maritime structures is that of chloride-induced corrosion of reinforcement or prestressing steel, with consequent cracking and bursting of the concrete cover and loss of steel cross-section due to corrosion. The concrete can also be weakened by the action of sulfates and by salt weathering of the surface. Physical processes of weathering and abrasion should also be considered.

In situations where chloride-induced corrosion has been avoided by appropriate design and specification, or by designing with unreinforced concrete or non-ferrous reinforcement, the effects of sulfates could prove to be a critical factor in the longer term. However, in the medium term, sulfate attack is reduced in the presence of chlorides and is less in warm water conditions than cold water conditions.

In colder climatic regions, freeze–thaw damage is an important consideration, the risk of damage being greater in concrete saturated by salt water.

The most important factors affecting the durability of concrete and reinforced concrete in seawater are:

– the pore structure of the concrete, as determined by the cement type and the water-cement ratio and unit water content;

– the influence of the moisture state of the concrete on the various transport mechanisms by which water and dissolved deleterious salts and gases are transported within the pore structure of the concrete;

– the influence of the macroclimate and microclimate on the moisture state;

– the chemistry of the cementing system;

– the resistance of the reinforcing system to corrosion.

In saturated concrete, chloride ions are transported by the slow process of diffusion. In dry concrete, chloride ions are transported into the cover zone much more quickly by the processes of absorption, wick action and hydration suction. However, the presence of chlorides affects only the loss of passivity of the steel. The corrosion process needs oxygen to be available for the cathodic reaction. Saturated concrete stifles the flow of the oxygen and so greatly reduces the rate of corrosion. The moisture state of concrete depends on the macroclimate and the microclimate of a particular element of the structure. In structures subject to tidal action, the microclimate depends on the relationship of each member to the fluctuating seawater level and, in particular, the amount of time it remains dry after inundation by seawater or spray. Concrete that is continually submerged remains saturated and, in cold and temperate climates and even in hotter climates, regularly wetted and surface dried concrete within the tidal range remains saturated and is less at risk from chloride-induced corrosion. The zones most at risk from chloride-induced corrosion are those subject to an unbalanced wetting and drying cycle, such as can occur in the following situations:

– locations subject to annual changes in mean sea level and tidal range due to seasonal, barometric or other reasons;

– locations such as dry docks, locks, slipways;

– areas adjacent to bollards, mooring winches and salt-water hydrants;

– areas at and just above high water level, which are subject to splash and irregular inundation;

– concrete exposed to seawater conditions in bridge piers, and underground structures and tunnels;

NOTE Underground structures can be subject to irregular inundation due to flooding and immersion during the construction process, and direct leakage and transport of moisture through joints, construction joints or the concrete.

– arid conditions, natural or manmade, which exacerbate all of the previous conditions.

The relative influence of the various transport mechanisms for chlorides, i.e. diffusion, absorption, wick action, permeation by pressure head, and the conditions in which salt concentration occurs are illustrated for a schematic maritime structure in Figure 69.

Where concrete remains in direct contact with chloride-bearing water or is in frequent contact with such water, the main local transport process is likely to be diffusion. In other locations, such as in the irregularly wetted upper part of the diagram, other, more damaging, processes can occur. The zones most at risk, as itemized above, correspond with the latter situation.

The surface of unreinforced concrete should be designed with freeze–thaw, abrasion, salt weathering and sulfate attack in mind, where applicable. The surface of reinforced concrete has the same design requirements as for unreinforced concrete, together with the selection of cement type in conjunction with cover to reinforcement appropriate to that cement type, for a given exposure condition.

Sulfate attack is more likely to present a risk in continually wetted concrete and is greater in colder water conditions. Salt weathering and freeze–thaw damage is more likely to present a risk in the intertidal and splash zone areas.

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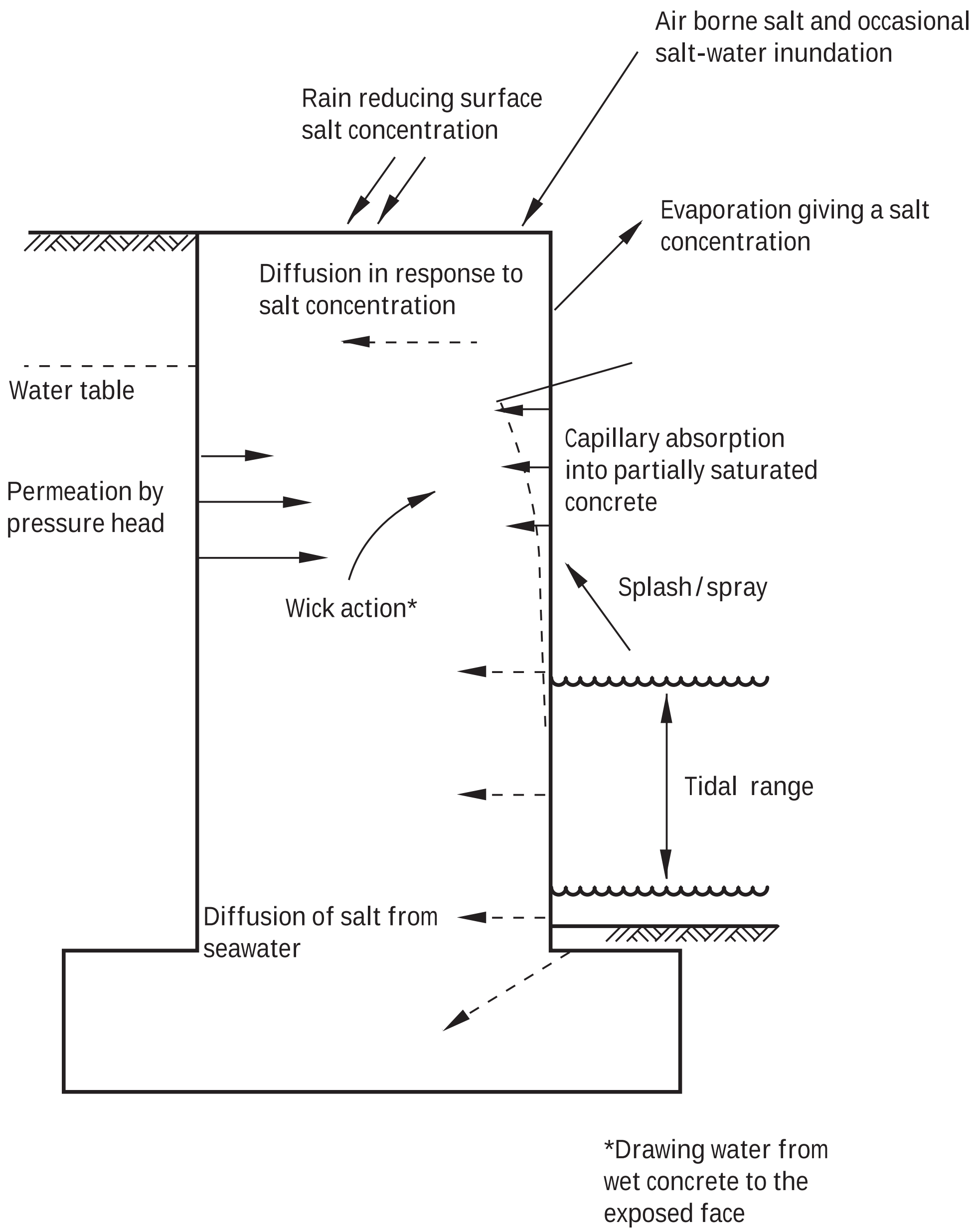

Figure 69 — Schematic diagram of the chloride transport processes in a maritime structure (Acknowledgement: The Concrete Society.)

58.3.3 *Exposure classification*

58.3.3.1 *General*

The various exposure condition classifications specific to the various separate deterioration mechanisms are described in [54]. In most situations and in seawater, a combination of more than one mechanism has to be considered, but nevertheless the classifications are more specific and realistic than in previous codes and are suitable for use.

However, the approach proposed for general building construction is not necessarily appropriate to the whole spectrum of design, detailing and specification for maritime and estuarine structures, which can be classified as “special structures”. These structures are covered by this code and should be under the control of the engineer and designer.

58.3.3.2 *Chlorides*

The moisture state, as explained in **58.3.3.4** and **58.3.2**, is critical to the processes of chloride penetration and subsequent electrochemical corrosion. The most important macro-climatic factors are temperature and rainfall, which control the rate of chemical reactions and the drying out characteristics of the cover concrete. Rainfall, humidity and the location of a member in relation to seawater level fluctuation control the wetness of the concrete and thus the mechanisms for the penetration of chlorides to destroy passivity and of oxygen to fuel the corrosion process. The wetting and drying depth in a temperate climate might not exceed 20 mm. In arid conditions, which can be either natural or due to enclosed conditions, the wetting and drying depth can be as great as 75 mm to 100 mm.

There are four main subdivisons of macroclimate that affect chloride-induced corrosion:

– cold with freezing;

– temperate;

– hot wet;

– hot dry.

Three classifications of micro-environmental classification are given for chloride-induced corrosion:

XS1 Exposed to airborne salt but not in direct contact with seawater

XS2 Submerged

XS3 The tidal, splash and spray zones

This subdivision requires further differentiation for the range of conditions encountered in maritime structures. A subdivision of XS2 might be appropriate to differentiate between totally submerged or with one face submerged and one face dry (i.e. under a pressure head). In the context of maritime structures, however, depending on the macroclimate and degree of wetness, it might be more appropriate to consider subdivisions of XS2 and XS3, in ascending severity such as:

– mid and lower tidal and backfilled;

NOTE Continuity of saturation makes these cases similar to XS2.

– upper tidal and capillary rise zones;

– splash/spray zones;

– mostly dry, infrequently wetted.

In cool and temperate conditions the latter two categories are similar. The subdivision of XS2, suggested for "dry" internal faces of submerged structures, also falls into the XS3 category.

Exposure conditions for chlorides in the various macroclimatic and microclimatic conditions are put into perspective in Figure 70. Natural conditions in the UK normally correspond to the temperate zone. Suggested severity ratings for chloride-induced corrosion on a scale of 1–12 are shown in the diagram, which gives comparative ratings for four climate zones, based on relative rates of chloride-induced corrosion for the same reinforced concrete element exposed to different maritime environments corresponding to the XS2 to XS3 classification. Figure 70 demonstrates the relative severity of class XS2 to XS3 in various macro- and microclimatic conditions.

58.3.3.3 *Sulfates*

Exposure classes for sulfates in the ground and groundwater are normally related to the concentration of sulfate ions. This is described in BRE Digest 363 [55]. However, the disruptive effect of sulfates is mitigated in seawater by the presence of chlorides. In international maritime practice, for concrete made with Portland cement alone, sulfate resistance is often achieved by limiting the tricalcium aluminate (C_3A) content to the order of 8 % for moderate sulfate resistance, or to less than the order of 5 % for full sulfate resistance. Higher proportions of C_3A, up to 13 % are, however, beneficial in resisting chloride-induced corrosion. Cements or equivalent mixer combinations containing blastfurnace slag or pulverized-fuel ash and Portland cement also provide sulfate resistance.

Sulfates are included in a classification for various causes of chemical attack with the following subdivisions. These classifications do not apply directly to seawater.

XA1 Slightly aggressive chemical environment

XA2 Moderately aggressive chemical environment or exposure to seawater

XA3 Highly agressive chemical environment

58.3.3.4 *Freeze–thaw*

The exposure cases specific to freeze–thaw damage are as follows:

XF1 Moderate water saturation without de-icing salt

XF2 Moderate water saturation with de-icing salt

XF3 High water saturation without de-icing salt

XF4 High water saturation with de-icing salt

Serious freeze–thaw exposure conditions are not experienced in UK maritime waters. Freeze–thaw damage is associated with either colder temperatures than are found in the sea around the UK or the application of de-icing salts. Designers need to be aware of freeze–thaw for sea conditions outside the UK and for road and approach surfaces in the UK subject to de-icing salts. In either case, appropriate guidance should be followed.

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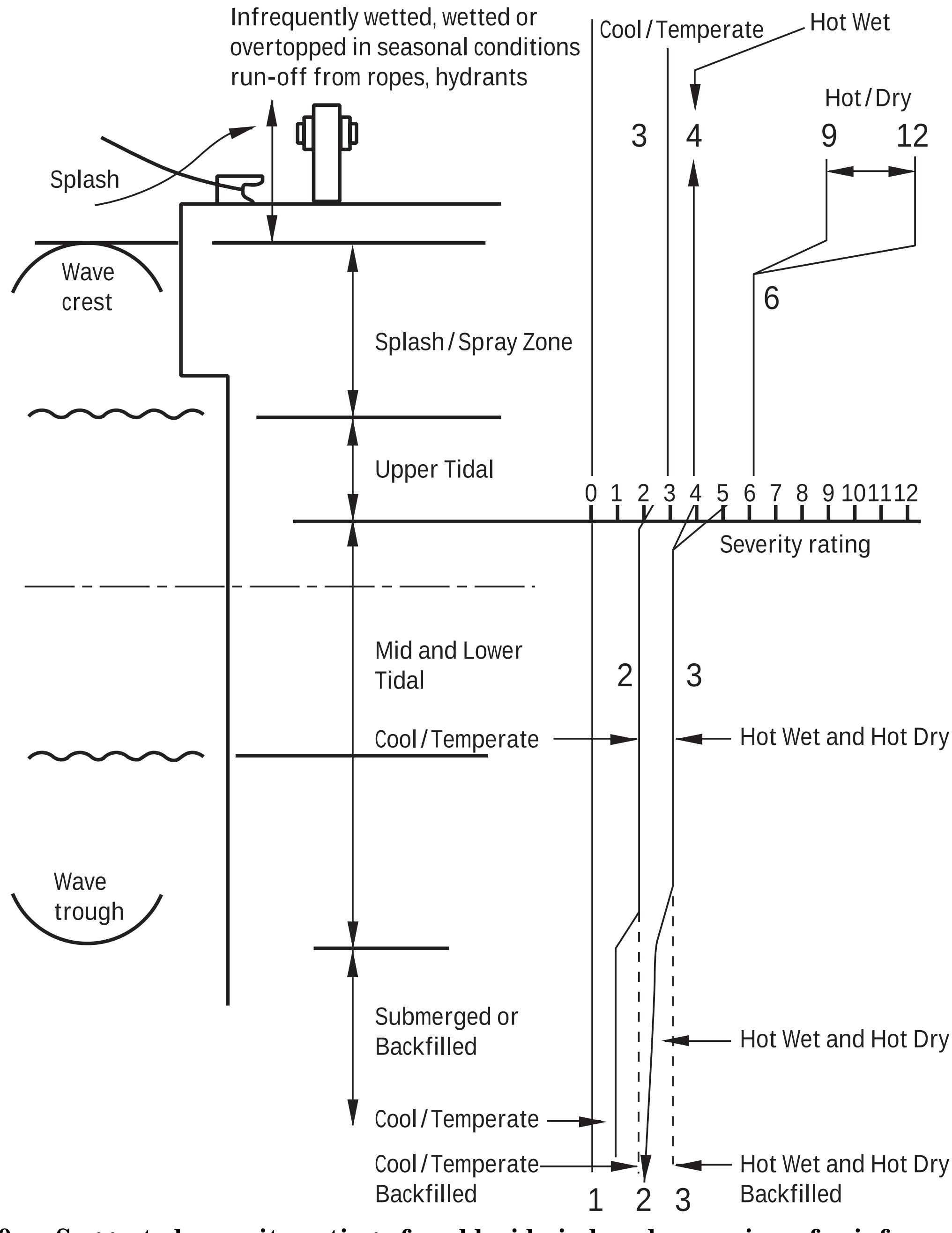

Figure 70 — Suggested severity ratings for chloride-induced corrosion of reinforcement on a scale of 1–12 (Acknowledgement: SLATER D. and B.SHARP. *The design of coastal structures*. In: ALLEN R. T., ed. *Concrete in coastal structures*. London: Thomas Telford Limited, 1998.)

58.4 Specification for materials and workmanship

58.4.1 *Cement*

Cements should conform to the British Standards given in Table 21. The choice of cement can depend upon whether the concrete is unreinforced or reinforced.

Where Portland cement is used, in UK waters, a maximum of 10 % C_3A, when determined by the method described in BS 4027, is recommended. The C_3A should not be less than 4 % for reinforced concrete in order to reduce the risk of attack of steel reinforcement by chlorides. This protection, though, applies mainly to any excess chlorides initially included in the mix, and has only a marginal effect when abundant chlorides are available from external exposure conditions. For this reason, sulfate resisting Portland cements with C_3A less than 4 % are acceptable, provided that the chloride limits for the concrete constituents are properly specified and enforced.

Alternative approaches, which combine sulfate resistance with chloride resistance, are to use combinations of Portland cement with at least 20 % pulverized-fuel ash (pfa) or 35 % ground granulated blastfurnace slag (ggbs). Higher levels of replacement (e.g. 30 % pfa or 70 % ggbs can be expected to produce significantly reduced rates of chloride ingress. The choice of replacement proportion is affected by the ruling climatic and site conditions e.g. the lower range of replacement proportions are suitable for slender members and/or colder climatic conditions if early set and strength is required by construction logistics. The higher proportions, which can be expected to provide increased long-term durability, depend on the exposure severity and are suitable for all but slender members. Either factory produced Portland blastfurnace slag or Portland pulverized-fuel ash cements can be used or the combination made at the mixer with Portland cements and cementitious components conforming to the relevant standards.

Microsilica (or silica fume) can also be beneficial when added to Portland cement in doses up to 10 % of the total cementitious content, or 5 % in conjunction with pfa or ggbs and Portland cement. [73]

Cover to reinforcement is dependent upon cement type, as set out in Tables 22 and 23.

It should be noted that the surface of concrete made with cements containing ggbs and pfa is usually less tolerant to poor curing and might be less resistant to carbonation-induced corrosion of reinforcement, freeze–thaw and abrasion surface weathering than concrete made with Portland cement without other cementitious additions. Proper curing regimes are an important factor in achieving durable concrete for all cementitious types (see **58.4.11**).

In order to minimize the risk of early thermal cracking, designers should check the criteria for degree of structural restraint and temperature differentials between adjacent pours and across thick sections, and detail joints and/or reinforcement accordingly. Temperature rises due to heat of hydration should be checked using the actual cement type and content finally used in the mix, which might exceed the minimum or nominal values initially specified. Temperature differentials might be limited by controlling cement content, varying the aggregate size (with regard to cement content) or by the use of cements containing pfa or ggbs with lower heat of hydration, or by precooling the materials, or by inbuilt cooling water pipework, or by the addition of liquid nitrogen.

Table 21 — Cements

Type	Standard	Other requirements
a) Portland cement: – Portland – Low-heat Portland	 BS 12 BS 1370	Tricalcium aluminate C_3A content by mass not more than 10 %. See **7.3.4.1**.
b) Sulfate-resisting Portland cement	BS 4027	
c) Cement containing ground granulated blastfurnace slag (ggbs) or pulverized-fuel ash (pfa): – Portland blastfurnace cement – High slag blastfurnace cement – Portland pulverized-fuel ash cement – Pozzolanic pulverized-fuel ash cement	 BS 146 BS 4246 BS 6588 BS 6610	
d) Combinations made in the concrete mixer from Portland cement and ggbs or pfa [56]: – Portland cement conforming to BS 12 in combination with ggbs conforming to BS 6699 – Portland cement conforming to BS 12 in combination with pfa conforming to BS 3892-1	Combination should conform to: BS 146 or BS 4246 BS 6588 or BS 6610	

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Table 22 — Limiting values for composition and properties of concrete classes with normal weight aggregates of 20 mm maximum size exposed to risk of corrosion of reinforcement induced by UK seawater conditions for a required design working life of 50 years

Exposure class and exposure conditions in the UK		Airborne salt			Frequently wetted		Infequently wetted. Upper tidal, splash/spray, "dry" internal faces of submerged structures		
					Submerged	Lower tidal, back-filled			
		XS1			XS2	XS2/XS3	XS3[a]		
Min. strength cylinder/cube (Mpa)[b,c]		C35/45	C30/37	C25/30			C40/50	C30/37	C25/30
Permitted cements		BS 12[d] BS 4027 BS 146 Portland slag cement BS 6588 Portland fly ash cement A	BS 146 Blastfurnace cement BS 6588 Portland fly ash cement B	BS 4246 BS 6610	In accordance with Table 24 except as below	In accordance with Table 24 except as below	BS 12[d] BS 4027 BS 146 Portland slag cement BS 6588 Portland fly ash cement A	BS 146 Blastfurnace cement BS 6588 Portland fly ash cement B	BS 4246 BS 6610
Permitted proportions for combinations (% by mass)	ggbs	≤35	35 <ggbs< 80	50 <ggbs< 80			≤35	35 <ggbs< 80	50 <ggbs< 80
	pfa	≤20	20 <pfa< 55	20 <pfa< 55			≤20	20 <pfa< 55	35 <pfa< 55
Max. w/c ratio[b,c,e]		0.45	0.50	0.55		0.50	0.40	0.50	0.50
Min. cement content (kg/m3)[b]		360	340	320		360	400	360	360
Min. cover to reinforcement[d,f]		40	40	40	40	50	60	50	40

NOTES

[a] This exposure class can vary in severity and can be worse in arid conditions, natural or man-made, such as in the "dry" side of submerged structures. See **7.2.2.4**.

[b] Minimum cement content depends on maximum aggregate size (see BS 5328). Water–cement ratio limits are the ruling parameters over minimum strength class or minimum cement content.

[c] Where there is difficulty in conforming to the strength recommendation at 28 days, because of the characteristics of the cement type or combination, then provided that a systematic regime of checking is established to ensure conformity to the free water–cement content recommendations, the 28-day strength recommendation may be relaxed.

[d] Excludes tolerance to be added to minimum cover to derive the "nominal" cover, which depends upon working conditions. See **7.3.4.9**.

[e] Maximum free water–cement ratio, in accordance with BS 5328.

[f] Add 10 mm for cover to prestressing strand.

[g] Limits for cement in accordance with BS 12, as **7.3.4.1**.

Table 23 — Limiting values for composition and properties of concrete classes with normal weight aggregates of 20 mm maximum size exposed to risk of corrosion of reinforcement induced by UK seawater conditions for a required design working life of 100 years

Exposure class and exposure conditions in the UK		Airborne salt			Frequently wetted		Infrequently wetted. Upper tidal, splash/spray, "dry" internal faces of submerged structures		
					Submerged	Lower tidal, back filled			
		XS1			XS2	XS2/XS3	XS3[a]		
Min. strength class cylinder/cube (Mpa)[b,c]		C40/50	C35/45	C30/37			C55/65	C40/50	C 30/37
Permitted cements		BS 12[d] BS 4027 BS 146 Portland slag cement BS 6588 Portland fly ash cement A	BS 146 Blastfurnace cement BS 6588 Portland fly ash cement B	BS 4246 BS 6610	In accordance with Table 24 except as below	In accordance with Table 24 except as below	BS 12[d] BS 4027 BS 146 Portland slag cement BS 6588 Portland fly ash cement A	BS 146 Blastfurnace cement BS 6588 Portland fly ash cement B	BS 4246 BS 6610
Permitted proportions for combinations % by mass	ggbs	≤35	35 <ggbs< 80	50 <ggbs< 80			≤35	35 <ggbs< 80	50 <ggbs< 80
	pfa	≤20	20 <pfa< 55	20 <pfa< 55			≤20	20 <pfa< 55	35 <pfa< 55
Max. w/c ratio[b,c,e]		0.40	0.45	0.50		0.50	0.35	0.40	0.45
Min. cement content (kg/m^3)[b]		400	360	340		360	400	370	370
Min. cover to reinforcement[f,g]		50	40	40	50	60	80	60	50

NOTES

[a] This exposure class can vary in severity and can be worse in arid conditions, natural or man-made, such as in the "dry" side of submerged structures. See **7.2.2.4**.

[b] Minimum cement content depends on maximum aggregate size (see BS 5328). Water–cement ratio limits are the ruling parameters over minimum strength class or minimum cement content.

[c] Where there is difficulty in conforming to the strength recommendation at 28 days, because of the characteristics of the cement type or combination, then, provided that a systematic regime of checking is established to ensure conformity to the free water–cement content recommendations, the 28-day strength recommendation may be relaxed.

[d] Limits for cement in accordance with BS 12. See **7.3.4.1**.

[e] Maximum free water–cement ratio in accordance with BS 5328.

[f] Excludes tolerance to be added to minimum cover to derive the "nominal" cover, which depends upon working conditions. See **7.3.4.9**.

[g] Add 10 mm for cover to prestressing strand.

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58.4.2 ***Aggregates***

Aggregates should conform to BS 882 or BS 3797 for lightweight aggregates. As a bulk materials resource, it might be necessary to make the best use of locally available and/or recycled materials of varying standards. In that case, studies of petrography, appropriate physical and chemical tests and trial mixes should be made.

The quality of aggregates has a lesser impact on the strength of concrete than is often supposed (other than effects of water demand, which might affect the water–cement ratio), but it does affect resistance to abrasion, freeze–thaw and surface weathering.

The soundness loss (BS EN 1367-2) should not be greater than 12 %. In accordance with BS 812-2, water absorption should not exceed 3 %, or 2 % in critical conditions, such as highly aggressive chloride or freeze–thaw exposure.

The acid soluble chloride ion limits (see BS EN 1744-1, Appendix C) and testing frequency should conform to BS 882. This is subject to the overriding limits of chloride ion in the concrete as cast, as given in **58.4.6**. Chloride ion limits should be considered for both corrosion of reinforcement and their contribution to the alkali level as they affect alkali aggregate reactivity. Special care is required in the selection and quality control of aggregates of marine origin to ensure conformity with BS 5328.

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 fines aggregate of the mix should be kept as low as is compatible with appropriate mix design.

58.4.3 ***Water***

Water should be 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. Seawater can be used in unreinforced concrete, although this increases the alkali content and risk of alkali aggregate reaction. It also accelerates set and enhances the risk of early thermal cracking. For the same reason admixtures containing chlorides should not be used.

58.4.4 ***Admixtures***

Admixtures should conform to BS 5075-1, BS 5075-2 and BS 5075-3. The use of appropriate admixtures should be encouraged to achieve the required workability for placing at the lowest water–cement ratio and unit water content as necessary for the mix to resist the exposure conditions. Other, proven admixtures for specialist and underwater concrete can be used.

The use of air entraining admixtures should be considered for the splash and intertidal zone and for any part of the structure that might be subject to freezing and thawing when in a saturated condition, although serious freeze–thaw exposure conditions are not usual in UK waters. Design for freeze–thaw should be considered for pavement surfaces subject to the application of de-icing salts (see **58.3.3.3**). Controlling the required amount of air is often difficult in stronger mixes where the cement content is of the order of 400 kg/m^3 and above. In this case, resistance to freeze–thaw in UK conditions can be obtained with more confidence by the use of richer non-air entrained mixes. Air entrainment does not enhance resistance to chlorides.

58.4.5 ***Fresh concrete***

Requirements for concrete mixes, the arrangements and conditions for the production and quality control of concrete, and the conditions for conformity to specified requirements should all be in accordance with BS 5328.

Mix design should be carefully carried out to achieve the necessary strength and the closed structure required for durability with minimum water–cement ratio and unit water content to achieve the required workability for the size of members, size of pour, distance and methods of handling concrete and climatic conditions.

In accordance with BS 8110-1, special variations in mix design might be necessary to overcome differences in the characteristics of local aggregate, i.e. shape and texture (with the consequent effect on water demand) and alkali–aggregate reactivity.

58.4.6 ***Hardened concrete***

The minimum requirements for the strength of concrete in the hardened state should be determined from consideration of the limit state requirements of BS 8110-1 and BS 8110-2, and/or, as appropriate, BS 5400. The requirements for achieving durability usually outweigh structural considerations of strength. For each cement type, higher strength concretes (i.e. with a lower water–cement ratio) normally have a higher durability than lower strength concretes. Provided that the concrete is of a quality that is adequate to resist the degree of exposure to chemical attack, freeze–thaw and abrasion, a trade-off can be made between concrete quality and cover to provide appropriate resistance to corrosion of reinforcement. Selection of cover to reinforcement depends on concrete quality and cement chemistry.

Procedures for testing for compliance and acceptance testing should conform to BS 5328.

The strength and closed pore structure required for durability are primarily controlled by the water–cement ratio. The cement content of a mix is a secondary consequence of the water demand for a given mix of materials, i.e. it is the product of the water demand required for workability divided by the designed water–cement ratio. The water demand of a mix depends upon the size and grading of aggregate and of the fines content and can be reduced by the use of admixtures. Cement content can also be reduced by increasing the aggregate size and/or reducing the fines content. Excessive cement contents exacerbate the risks of early age thermal cracking, alkali aggregate reactivity and, even at equal water–cement ratios, involve higher unit water contents that increase moisture movement and air and water permeability and decrease freeze–thaw resistance. Nevertheless, for concrete in maritime conditions, it is prudent to specify a minimum cement content, as well as the ruling water–cement ratio, in order to guarantee an appropriate volume and richness of cement paste for chloride binding and chemical resistance.

In the specific cases of concrete exposed to seawater in the UK, unreinforced concrete mixes for design working lives up to 100 years should conform to the limits given in Table 24.

For reinforced concrete, mixes and cover to reinforcement for 50-year and 100-year design working lives should either be derived by systematic durability design methods or conform to the prescriptive limits given in Tables 22 and 23. Note that the concrete qualities set out are minimum recommendations for the conditions described. Subject to a rational design basis, cover to reinforcement and concrete parameters can be adjusted between the values set out.

Higher concrete qualities or values of cover to reinforcement may be required on the grounds of structural strength, for specific exposure conditions, or to reduce the probability of durability failure.

Chloride limits given for mix constituents are subject to the overriding limits for the designed mix, which should be confirmed by measurement on the hardened concrete, both in trial mixes and in the works.

In accordance with the test method in BS 1881-124:1988, the acid-soluble chloride ion content (percentage by mass of cement), as estimated from the constituents, should not exceed the following limits.

Prestressed and heat-cured	0.10 %
Reinforced with PC C_3A < 4 %	0.15 %
Reinforced PC or cement containing pfa or gbbs	0.20 %
Plain concrete	0.50 %

For tests on hardened concrete, the limits are as follows.

	Limit of 95 % of results	**No results more than**
Prestressed and heat-cured	0.10 %	0.15 %
Reinforced with PC C_3A < 4 %	0.15 %	0.20 %
Reinforced PC or cement containing pfa or gbbs	0.20 %	0.25 %
Plain concrete	0.50 %	0.65 %

Table 24 — Limiting values for composition and properties of plain concrete with normal weight aggregates of 20 mm nominal maximum size exposed to UK seawater conditions for a required design working life in excess of 50 years[a]

Exposure class and exposure conditions in the UK		**Submerged, intertidal and splash**	
Min. strength class cylinder/cube[b,c] Mpa		C30/37	C25/30
Max. w/c ratio[b,c,d]		0.55	0.55
Min. cement content kg/m[b,c]		320	320
Permitted cements		BS 12[e] BS 4027 BS 6588 BS 146	BS 4246 BS 6610
Permitted properties for combinations % by mass	ggbs	≤ 50	50 <ggbs< 80
	pfa	≤ 35	35 <pfa< 55

NOTES

[a] The recommendations in this table are also appropriate for an intended design working life of 100 years.

[b] Minimum cement content depends on maximum aggregate size (see BS 5328). Water–cement ratio limits are the ruling parameters over minimum strength class and minimum cement content.

[c] Where there is difficulty in conforming to the strength recommendations at 28 days, because of the characteristics of the cement type or combination, then, provided that a systematic regime of checking is established to ensure conformity to the free water–cement ratio and cement content recommendations, the 28-day strength recommendations may be relaxed.

[d] Maximum free water–cement ratio, in accordance with BS 5328.

[e] C_3A not greater than 10 %.

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58.4.7 ***Steel reinforcement***

Black, uncoated, steel reinforcement should conform to BS 4449, BS 4482 and BS 4483. Cutting and bending of reinforcement should conform to BS 4466. Measures should be taken to ensure that reinforcement is maintained free from salt deposits in transit, storage and during fixing. Where, during the construction process, exposed reinforcement is immersed in seawater, it should be checked for salt contamination and pitting corrosion other than surface rusting, and cleaned if necessary.

Stainless steel reinforcement should be of higher grade, equivalent to austenitic steel in accordance with BS 6744, Type 316 S 33 (hot rolled austenitic steel). Fusion-bonded epoxy coated reinforcement should conform to BS 7295.

58.4.8 ***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 and BS 5400.

The additional recommendations in [57] should also be observed.

58.4.9 ***Cover to reinforcement***

Cover to reinforcement refers to the minimum distance from the surface of the concrete to any steel reinforcement links, tendons or sheaths. The thickness of the binding wire is not included, but ends of binding wire should be turned away from the cover zone and into the body of the concrete.

The required cover to reinforcement for a specific exposure condition and a specific design working life is dependent upon three principal parameters:

– the quality of the concrete as determined by the water–cement ratio;

– the cement type;

– the placing tolerance.

The nominal cover (i.e. the figure used in the design and detailing and shown on the drawings in BS 8110) should be the minimum value (either calculated by an explicit durability method or from the tables herein) together with a tolerance.

There are two recommended methods for calculating the minimum cover (before addition of tolerance or placing allowance) to minimize the risk of corrosion to reinforcement for a 50-year minimum or 100-year design working life. The first is a systematic method and the second is to select according to the exposure conditions, type of cement and maximum water–cement ratio, as given in Tables 22 and 23.

The required tolerance ranges from between ±10 mm to ±15 mm for normal in-situ construction to between ±5 mm to ±10 mm for more precise or factory-controlled work. Tolerances of ±5 mm and below should only be used when conformity is demonstrated by IE trials and quality control procedures (refer to Eurocode 2). Where problems exist, e.g. difficulties of access, visibility, or opportunity for inspection and maintenance, cover should be increased. Increased cover is required in particularly vulnerable positions, for example horizontal surface tops.

Increased cover might conflict with a requirement to control the size and spacing of flexural cracks. In maritime conditions, however, the requirement for cover usually governs, and cover should not be reduced in order to control flexural crack widths. Any limiting values should rather be increased pro rata to the greater cover, as allowed by expressions that relate acceptable flexural crack widths to a proportion of the cover thickness. It is not necessary to sum the effects of early thermal [see **58.2**g) and **58.4.1**] and flexural cracking.

58.4.10 ***Additional protective means against reinforcement corrosion***

Where the risk of reinforcement corrosion is likely to be extreme, or where structural and detailing feasibility rules out large values of cover, either the quality of the concrete as regards its resistance to chloride-induced corrosion can (within limits) be increased or other protective measures adopted. Consideration can be given to the use of stainless steel reinforcement or, subject to experience, reinforcement protected by fusion-bonded epoxy coatings. Other measures include surface coatings, but only high grade, high build surface coatings are suitable for maritime work, and these require first class surface preparation.

Cathodic protection can be used for the protection of concrete reinforcement or the conventional protection of steel structures. The dangers of corrosion of adjacent structures (either reinforced concrete or steelwork) should be considered in each case. If an impressed current system is to be used, care should be taken to prevent stray currents from causing such corrosion.

58.4.11 ***Curing***

The closed pore structure of the surface concrete depends upon the degree of hydration relative to the water–cement ratio and binder type. It is less clear whether continuous wet curing is essential in wet temperature conditions but, in hot and dry conditions, water loss from the surface can result in incomplete hydration. As a related topic, excess bleeding can result in an excessive water–cement ratio at the top surface, whereas some bleeding can be beneficial in avoiding desiccation and plastic cracking.

Concrete made with Portland cement usually requires shorter curing periods than concretes made with pfa or ggbs. In large sections these differences are marginal but in thin or slender sections they can be significant. Pfa and silica fume tend to reduce bleeding. Bleeding performance needs to be checked in concretes containing ggbs.

Curing of massive elements, for example caissons and massive piers or units, presents logistical difficulties for fixing of the curing materials, and securing plastic or other sheeting against wind and weather. Such practicalities should be considered at the design stage, as opposed to placing reliance on measures that might be too difficult to carry out. Where concrete can be maintained within a humidity-controlled environment, curing conditions can be enhanced.

Indiscriminate wet curing with cold water and/or the premature removal of formwork can lead to thermal shock. For formed surfaces it is advisable to leave forms in place as long as is feasible. Controlled permeability formwork liners can be used.

The lower the water–cement ratio, the lower the theoretical duration required for curing, which means that low water–cement ratio concretes are more tolerant to shortfalls in curing. Saline water should not be used for curing.

58.4.12 ***Underwater concrete***

Underwater concrete can be carried out in situ by placing conventionally mixed fresh concrete by the methods listed here or by the process of pumping cement grout into previously placed aggregate. Guidance on developments in underwater concreting is given in [58].

The principal methods of placing fresh concrete under water are:

a) Tremie type methods. Contact between the concrete face and water is carefully controlled by injecting fresh concrete only into the mass of previously placed but fluid concrete, by tremie, pumping, underwater skip or hydrovalve. These methods are suitable for plain concrete and simple, massive reinforced concrete sections.

b) Contact between fresh concrete and water prevented, as in prefilled bagwork or pumping into collapsed flexible forms. This method is not suitable for reinforced concrete.

c) By introducing admixtures and/or additives that give enhanced cohesive, self-levelling and self-compacting properties. These methods are suitable for reinforced sections.

d) Grouting with or without admixtures or additions, for grouted aggregate concrete or the grouting of connections between structural members, fissures, voids or joints.

Reinforced concrete members with concrete placed underwater should be designed for structural strengths and tolerances that can feasibly be achieved in situ and appropriate to the logistics of construction and quality control.

For normal working methods, the mix design should be carried out in the same way as for work in the dry, while bearing in mind that the concrete should be free-flowing, cohesive and self-compacting. A concrete that is highly workable and can be pumped has the characteristics required for most underwater applications. High strength concrete is not normally necessary. The properties of concrete properly placed underwater are broadly similar to those of the same concrete placed in the dry. The quality of the finished product is controlled by the quality of the materials and workmanship, together with the care taken to obviate defects such as areas of washout, silt inclusions etc.

Adjustment of the cement content and the use of either water-reducing or specialist admixtures can achieve the high workability required. It is usually unnecessary for cement contents to exceed 425 kg/m^3, and a range of 350 kg/m^3 to 425 kg/m^3 is common. A minimum slump of 125 mm is normally required. The high workability is best measured using the flow table (BS 1881-105). This test has the added advantage of showing whether the concrete is cohesive or prone to segregation.

Concrete can also be placed underwater as precast units and used either as structural units or as permanent shuttering for freshly placed concrete underwater.

Patented processes are usually employed for the grouted aggregate method of placing underwater concrete, where the forms are first filled with coarse aggregate and then grouted by sand–cement grout. Colloidal characteristics are injected to fill the voids completely. Portland cement grouts are normally used, together with pfa or ggbs to improve the fluidity of the grout. Reference should be made to BS 3892-3 for guidance on the use of pfa in cementitious grouts. Admixtures are often used.

59 Structural steel and other metals

59.1 General

The most commonly used metals in maritime structures are weldable structural steels (see **59.2**). Other metals that 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 copper nickel alloys (see **59.4**).

All metals suffer more from corrosion in a maritime environment than inland. This is mainly due to:

a) the formation of galvanic cells, with anodes and cathodes being formed either between different metals (e.g. weld metal) or within the one material due to varying conditions (e.g. differential aeration) or because of the presence of strong currents 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 planned preventative measures and/or maintenance by owners.

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 might be preferable to provide grade S 355 steel designed at the stress levels appropriate for grade S 270 steel. This yields a thicker section with a greater allowance for corrosion before the structure is endangered. It might, however, be more appropriate to quantify a specific corrosion allowance from corrosion rates forecast from knowledge of local conditions.

Fabrication details should be kept as simple as possible and be designed to avoid corrosion and facilitate maintenance. This applies especially to on-site connections where bi-metallic contacts should be avoided and 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 pre-installation painting under factory-controlled conditions, provided that these operations are carried out where adequate inspection and supervision are possible. Inadequate or construction-damaged coatings (e.g. from pile driving) could exacerbate corrosion conditions.

It is recommended that a maintenance strategy is developed for all marine structures to ensure that periodic inspection is carried out enabling corrosion or other deterioration to be identified and dealt with at an early stage before it affects the integrity of the structure.

59.2 Structural steel

The steels most commonly used in maritime structures are weldable structural steels, which should conform to BS EN 10025 for structural sections, BS EN 10248 for hot rolled sheet piling and for tubular piles, BS EN 10210 for hot formed, and BS EN 10219 for cold formed sections.

Steel grades in accordance with BS EN 1025 that are available are:

Grade S 275 minimum yield stress 275 N/mm^2;

Grade S 355 minimum yield stress 355 N/mm^2.

Steel sheet piling can also be supplied in steel grades in accordance with BS EN 10248 as follows:

Grade S 270 GP minimum yield stress 270 N/mm^2;
Grade S 355 GP minimum yield stress 355 N/mm^2.

59.2.1 ***Corrosion rates***

Marine environments usually include several exposure zones with differing degrees of aggressiveness. The corrosion performance of marine structures therefore requires separate consideration in each of these zones. The average and upper limit values for the different exposure zones are given in Table 25. The rates apply to each face exposed to the environment of the zone. The rates given in the table should be regarded as applicable to uniform or general corrosion and can be used to assess the (theoretical) design life of a structure.

Localized higher rates of corrosion can occur due to several mechanisms; these conditions, applicable corrosion rates and preventative measures are discussed in **59.2.2** on concentrated corrosion.

Table 25 — Typical rates of corrosion for structural steels temperate climates

Exposure zone	Corrosion rate mm/side/year	
	Mean[a]	Upper limit[b]
Atmospheric zone: – above splash zone and where direct wave or spray impingement is infrequent	0.04	0.10
Splash zone: – above mean high-water to a height depending on mean wave height and exposure to wind	0.08	0.17
Tidal zone: – between mean high-water and mean low-water spring level	0.04	0.10
Intertidal low water zone: – between low-water spring and 0.5 m below LAT	0.08	0.17
Continuous seawater immersion zone: – from 0.5 m below LAT to seabed level	0.04	0.13
Below seabed level or in contact with soil		0.015 max

[a] The rate is for each face exposed to the environment of the zone.

[b] The upper limit figures are the 95 % probability values.

NOTE Concentrated corrosion rates are given in **59.2.2**

59.2.2 ***Concentrated corrosion***

In marine environments, accelerated corrosion can be caused locally by several mechanisms, which are outlined in the following list. These forms of accelerated or localized corrosion are referred to as "concentrated corrosion".

a) Repeated removal of the protective corrosion product layer, particularly in the low water or immersion zones by the action of fendering systems, waterborne sands and gravels or repeated stresses. The area where the rust layer is repeatedly removed becomes anodic to the unaffected areas.

b) Bi-metallic corrosion, where steel is electrically connected to metals having nobler potentials or where weld metals are significantly less noble than the parent material.

c) Accelerated corrosion in the low water zone associated with microbiological activity, often referred to as Accelerated Low Water Corrosion (ALWC), which has been found in British estuarial waters since the 1980s and also in other parts of the world. Its effect is normally concentrated in discrete areas, within a 1 m range, centred between mean low water level and LAT. It is often randomly located along a structure, but where it occurs on sheet piles, attack is generally located at outpan corners and the adjacent web on "zed" sheet pile sections, on outpans on a "U" section, and centrally on straight section piles. Because of the differences in pile section thickness at these corrosion sites, a similar strength section could yield a different life if ALWC occurs.

Forms of concentrated corrosion also occur just above seabed level.

Concentrated corrosion is unlikely to cause catastrophic global failure of a sheet-piled structure, owing to the continuous nature of such a structure. It may be more critical on a king pile or other structural member. If corrosion of a sheet-piled wall were allowed to continue to the point of perforation, however, loss of retained material could lead to collapse of adjacent surfacing or structures supported on the backfill.

Corrosion rates for concentrated corrosion can be typically 0.5 mm/side/year, averaged over time, to the point of perforation of the member. The corrosion is often of a pitting form, and can be increased, for example by repeated removal of the corrosion products. Rates of 0.8 mm/side/year have been recorded in UK coastal waters.

59.2.3 ***Measures against corrosion***

It is uneconomic to design an entire wall on the basis of concentrated corrosion rates. In situations where concentrated corrosion is likely, therefore, the high rates of corrosion within the affected area should be evaluated by developing a specific solution, using one of the following methods.

a) use of a thicker steel section, a higher grade of steel or increase the pile thickness locally by the addition of steel plates;

b) use of a high quality protective coating;

c) use of an electrical bonding system and cathodic protection. (see **66.7**);

d) optimization of the design to ensure that high bending moments do not occur where corrosion rates are highest.

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Further detailed measures are given in clause **66.**

It is recommended that, where appropriate, one or more of the measures just described is adopted to provide the desired effective life; or alternatively, a practical in-service repair is designed for implementation after a planned period. Structures should be inspected at regular periods in order that any unusual corrosion activity can be detected at an early stage. It should be noted that it is also possible that physical inspection could encourage corrosion by the removal of localized fouling [see **59.2.2**a)].

At an early stage pitting corrosion for ties and anchors that are tension loaded can be of more serious concern. In this case even small corrosion pits can cause stress concentrations that could promote failure below the nominal design and/or yield stress of the material. Allowance should be made for this.

59.2.4 ***Type of steel***

Welded connections between structural elements are in common use on maritime structures, either exclusively or in conjunction with bolted connections, so only steels that are suitable for welding should be used. In most cases, welding by manual metal arc is favoured and steels that conform to the relevant British Standard (see **59.2**) should be selected, rather than other steels, which might require more specialized welding techniques.

59.2.5 ***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 the relevant British Standard (see **59.2**) for various design minimum temperatures and various thicknesses. The requirements specified in the relevant British Standard (see **59.2**) are adequate for normal maritime structures, but stricter requirements might apply where brittle fracture at low temperatures is of particular concern.

59.2.6 ***Chemical composition***

The chemical composition limits and carbon equivalent values specified in the relevant British Standard (see **59.2**) are adequate to provide satisfactory weldability for maritime construction where lamellar tearing risks are low.

Weather-resistant steels are not recommended for maritime structures, because their better resistance to inland atmospheric corrosion is not realised 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 therefore occurs at the interface of a partly embedded member. In such cases, special precautions should be taken.

Chemical composition of steel 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 corners that allow re-entry, 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 and to 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, lamellar tearing can occur. In such cases, requirements additional to the properties specified in the relevant British Standard (see **59.2**) 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 with anything other than a simple tar coating, the mill scale that normally coats steel sections, plates and bars on delivery should be removed. If it is not removed and is allowed to weather off, galvanic corrosion is promoted by cells formed between the mill scale and the parent and deep pitting can result (see **59.2.2**).

59.2.7 ***Quality of material***

The required quality of the steel should be considered in respect of discontinuities, such as laminations in plates and forgings, and blowholes and sand inclusions in castings. The appropriate design requirements in relation to service stresses and fabrication procedure should also be taken into account.

Plate material required to have a quality level in respect of laminations and inclusions should be examined in accordance with BS 5996.

Castings and forgings can 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.

59.2.8 ***Welds and welding***

Consumable materials for welding of steel in maritime structures should conform to BS EN 440, BS EN 449, BS EN 756 and BS EN 760.

59.2.9 ***Bolts and nuts***

The use of bolts and nuts should conform to BS 5950 and BS 4190 (ISO 272, ISO 885, ISO 888 and ISO 4759/1).

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 these elements. The current standards classify alloys of aluminium according to their chemical composition, denoted by a number that 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 EN 515, parts 1 to 4 of BS EN 485; parts 1 to 4 of BS EN 573, parts 1 and 2 of BS EN 586, parts 1 and 2 of BS EN 603, BS EN 604-1:1997, parts 1, 2, 7 and 8 of BS EN 754, parts 1 to 8 of BS EN 755, parts 1 to 3 of BS EN 1301, BS 1473, BS 1490 and BS 8118, which deal with the specification and use of aluminium alloys for general and structural engineering purposes.

59.3.2 *Selection of alloy*

BS 8118 provides guidance on the selection of alloys. Although alloy 6082 (formerly H 30) is the alloy normally 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 5083 (formerly N8), 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 principle alloys for structural use, reference should be made to BS 8118.

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 that 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 structural elements loaded in-plane.

The coefficient of linear thermal expansion of aluminium is between (23×10^{-6}) per °C 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 BS 8118, section **7** 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 conform to the requirements of the relevant British Standards where appropriate as follows:

a) *Stainless steel* (BS EN 10088 for sheet, plate, strip, bars, rods and sections). Molybdenum-bearing, corrosion-resistant steels are preferable. Grade 316 is the minimum anti-corrosive grade that should be used. Austenitic stainless steels are liable to pitting corrosion in seawater in conditions where oxygen is excluded.

b) *Cast iron* (BS 3468 and BS 4844).

c) *Wrought iron*. Has good corrosion resistance.

d) *Brass and bronze* (BS 2870, BS 2871, BS EN 12165, BS EN 12166, BS EN 12163, BS EN 12164, BS EN 12167, BS 5290 and BS 5567 for wrought materials; BS 1400 for castings). Brasses usually suffer from dezincification in seawater and should be avoided.

e) *Monel metal*. Has high corrosion resistance but is expensive (see **66.4**).

60 Timber

60.1 General

The use of timber in maritime structures should be in accordance with BS 5268-2.

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 [59].

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 that normally 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 or cargo handling equipment, or the wearing away of timber by pedestrian and vehicular traffic.

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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 particular 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 is retarded on the exposed faces of timbers and decay mainly proceeds outward from joints and protected faces.

60.2.4 ***Fungal decay***

The rise in moisture content that 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, so timber that is sufficiently deeply immersed in water or is embedded in impermeable soil is not 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 that would encourage retention of moisture should be avoided.

60.2.5 ***Marine borers***

The two most common types of marine borer that 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 that are capable of absorbing wood preservatives or are naturally durable. 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.

Natural resistance to fungal decay varies from species to species. Variation is also observed within single species and in some timbers the heartwood core can 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 that greatly increase the life of the less durable timbers when subject to both maritime and freshwater conditions. Protective treatments include various processes in that timber is impregnated with preservative solutions under pressure. These should conform to 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 that 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, ekki(azobe), balau and basrolocus. Many other species can be used, however, and choice depends 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 can 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. In these cases 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 depends largely on a combination of availability and required durability. Experience of rubbing strips made of plastic (**66.3**) indicates that they are preferable to timber on a whole-life basis.

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 can then be selected for impact and abrasion resistance.

60.3.5 *Sea defences*

Among the timbers that 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 can 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 can 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 BS 5268-2. Bolted fastenings should conform to BS 4190. Washers should conform to BS 3410, which includes coverage of large square and round washers for use on timber. Connectors for timber should conform to BS 1579.

Other fastenings that 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 conform to 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 conform to BS 1494. Parts not embedded in concrete should be treated with suitable bituminous compounds or paint.

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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 BS 8004 and particular considerations relevant to the maritime environment are given in **61.2.2** to **61.2.5**.

Reference [34] gives detailed guidance on pile design and construction.

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 might 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 that 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 drive 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 retractable 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 groundwater 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 normally withstand attack from quite high concentrations of sulfates in soils and groundwaters (see **58.3.2**). Piles can also be damaged by abrasion [see **58.2**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 usually have an adequate resistance to corrosion. The portion of the piles above the seabed in maritime structures, in disturbed ground or in corrosive soils can be protected by cathodic protection or other methods, as described in clause **66**, 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 reduced 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 rubbing strips spiked or coach-screwed to them.

The working stresses in timber piles should not exceed the permissible green stresses, given in BS 5268-2, 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 BS 5268-2 should be adequate for softwood. 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.

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 that 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 to impact from harbour craft or lighters than reinforced concrete piles.

The working stresses in the concrete during lifting, handling and pitching of precast piles should not exceed those recommended in BS 8110-1. High stresses, which can exceed the handling stresses, can occur during driving and it is necessary to consider the serviceability limit of cracking. The cover to reinforcement for piles exposed directly to seawater should be derived as advised in **58.4.6** (including Tables 22 and 23) and **58.4.9**.

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 that, in the latter instance, is advantageous in situations where ground heave and lateral displacement have to be avoided. Steel piles can be readily cut down or extended where the level of the bearing stratum varies and the head of a pile that 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. The practice is satisfactory in the leaders of a piling frame for land structures. It might, however, 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 where the highest strength and best protective treatment are therefore 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 can be driven with a closed end in order to assist developing increased and bearing resistance over the pile base area. Where deep penetrations are required, they can be driven with open ends possible 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 solid plug within the pile can itself develop the required base 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.

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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, because there are no corners from which the soil might 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. Typical details are shown in **4.4** of BS 6349-2:1988. The materials and preservative treatments that 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 assists in close driving.

61.3.2 *Concrete sheet piles*

Reinforced concrete sheet piles are used in retaining walls in canal banks, where they are quite short and therefore 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. Typical joint details are shown in **4.4** of BS 6349-2:1988, but it is 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 assists 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 conform to **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.

Cross-sections of the various types of sheet pile that are in common use are shown in **4.4** of BS 6349-2:1988.

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.

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 pile can 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 might not be justified as they do not afford any useful improvement in corrosion resistance in the intertidal zone (see Table 25).

Information on concentrated corrosion on sheet piles is given in **59.2.2** and methods of protection in **59.2.3**. Other systems for protecting steel piling against corrosion are given in clause **66**.

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.

Pipes are manufactured in a wide variety of materials. The following is a list of pipe materials in general use, which should conform to the requirements of the appropriate standard where such exists:

1) concrete;

2) ductile spun iron;

3) glass reinforced plastics (GRP);

4) glazed vitrified clayware (GVC);

5) plastics:

i) unplasticized polyvinyl chloride (PVC-U);

ii) polyethylene (PE) in various grades;

iii) polypropylene;

iv) acrylonitrile butadiene styrene (ABS);

6) steel.

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.

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.

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 forklift trucks handling loaded containers frequently exceed 70 t (see Table 12). Systems in which four or more loaded containers are carried on a chassis can give rise to single bogie loads of 150 t. These loadings are quite different from those experienced because of 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 are usually significantly heavier in design and more costly than highway pavements. To achieve economy in planning port layouts it might 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 or concrete block construction should be adopted.

Modern cargo handling equipment can have solid rubber or plastics wheels or can 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 that, 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 that is tolerant of settlement, can be regulated and repaired from the top, or can easily be taken up and relaid should be selected. Rigid pavements that are not tolerant of

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settlement and that 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 might be desirable to provide a temporary pavement, which can serve operational needs for an interim period while settlement continues. Such a temporary pavement can be designed to form one of the lower courses of the final design pavement after regulation.

64 Rails

64.1 General

Rails are 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 and materials should conform to BS 11, BS 105 and BS 500, as appropriate. Further standards cover the requirements of fittings, including fishplates and fishbolts and are referred to in these standards.

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 because problems can arise due to differential settlement and movement between tracks and pavement.

Crane rails and rail track for special equipment can be required for a wide range of duties including the carriage of wheel loads much heavier 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 is 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.

64.2 Crane rails

The correct selection of bedding system and holding down bolt arrangement is most important, because many failures of container crane rails have been reported.

The main problem is the bow wave that is induced in the rail by a heavy moving wheel. This causes cyclic loads in the fixings and a pumping action on the bedding layer due to the inevitable presence of water.

There are two bedding fixing systems in general use:

a) a continuous support, involving a continuous rubber pad between the rail and a steel base plate, which is bedded on a pourable cement or epoxy based material;

b) an intermittent support, as in permanent way.

64.3 Adjustment of crane rails

Crane suppliers demand tight tolerances on rail straightness, track alignment and level. The bedding and fixing system selected should allow vertical adjustment to be made after the crane has been put into service.

64.4 Holding-down bolts

Where a bedding medium that is placed in situ is used, the system of using holding-down bolts that have been provided should allow the rails to be accurately adjusted to line, level and freedom from twist. The rails should also be rigid enough to hold the rails in the correct register whilst the bedding materials are placed and cured.

64.5 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 might be necessary for the clips to permit some linear displacement of the rail under thermal expansion.

64.6 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 can be accommodated by the rail sliding under the rail clips, provided that these are of a suitable design to allow such movement.

64.7 Bolted joints

Bolted joints in crane rails tend to give trouble, so rails should normally be jointed by welding, employing the usual poured in-situ method, as in permanent way work.

65 Bituminous materials

65.1 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 **66**. 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 **65.2** to **65.4** inclusive.

65.2 Bituminous materials available

Bitumen is employed in a number of ways for coast 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 normally dense and their impermeability is often a disadvantage from the design point of view. A low void content is nevertheless recommended owing to the 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 stone filled sheet asphalt also belong to this group);

b) sand mastic or asphalt mastic;

c) dense stone asphalt;

d) open stone asphalt;

e) lean sand asphalt.

65.3 Composition, mix design and application techniques

65.3.1 *Asphaltic concrete*

Asphaltic-concrete types of mixtures consist of crushed stones or gravel with a maximum size of 25 mm, graded sand, filler and bitumen. These mixes usually need to be compacted after spreading and have a certain amount of stability. Normally they are applied under dry conditions.

Asphaltic concrete is normally used for revetments. It should 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 is normally higher than for paving mixes in order to obtain good impermeability and durability. A figure of 6 % to 9 % is commonly used.

b) Penetration grade bitumens 50 pen, 70 pen or 100 pen, conforming to BS 3690-1, are normally used to facilitate compaction. The softer grades are preferred, because compaction is difficult on slopes.

c) To minimize segregation, a continuous aggregate grading with a maximum stone size of 12 mm to 25 mm is commonly used. Gap-graded mixes, which need less compactive energy, are sometimes used, though. Aggregates should have good adhesion characteristics and be angular or crushed for revetments that are placed on slopes steeper than 1:2.5.

d) A well-graded sand is advisable because otherwise high percentages of filler would have to be added in order to reduce the voids content. This can lead to the production of mixes that are difficult to handle.

e) Filler contents, i.e. materials smaller than 75 μm, are usually between 8 % and 13 %, limestone or cement being preferred.

f) In order to ensure that the mix is impermeable i.e. with 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 [60]. 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 of 5 % to 6 % 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. It should be placed in the dry and various methods are used for spreading the mixture over the revetment. The simplest is by dumping the mix directly from a truck on the formation 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. Unless slopes are very shallow, compaction rollers have to be operated by a winch system.

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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. Furthermore, problems related to adhesion between different layers are thereby eliminated.

65.3.2 ***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 below water, either as a grouting material for stone revetments or as a carpet. The material can also be used for the construction of prefabricated mattresses.

The composition of sand mastic can vary within wide limits and thus extensive use of local materials can be made. It should, however, conform to the following two requirements.

a) During application the mastic should be pourable, with 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. over about 10^9 to 10^{10} Pa, 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 finely 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, which are 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 that can lay an impermeable sand mastic carpet continuously on the seabed at depths of up to 30 m. 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.

65.3.3 ***Open stone asphalt***

Open 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. By this means the mix coats the large stones and binds the whole together but without destroying the permeability (**65.2**). Due to this special manufacturing process, open stone asphalt possesses some unique properties. For example, the aggregate is gap-graded, the mix is just pourable and no compaction is required. Open stone asphalt can be used below water but only in the form of prefabricated mats. Above-water it can be applied as a monolithic revetment or as a grouting material for very large stones. Because of its high stability open stone asphalt can be applied on slopes in much thicker layers than normal asphaltic concrete. Open stone asphalt is transported and laid in a similar manner to asphaltic concrete.

The composition of open stone asphalt can vary between certain limits, depending on the type of application and it is capable of making strong, porous yet durable revetments.

Dense (or heavy) stone asphalt can be considered as consisting of large stones (maximum 500 mm) in that the interstices are filled with a slightly overfilled asphaltic mastic or concrete.

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.

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 excessive loss of material.

65.3.4 ***Lean sand asphalt***

Lean sand asphalt consists of a mixture of locally available sand and 3 % to 6 % of bitumen, and is therefore a relatively cheap material. It is a greatly underfilled mix, the function of the bitumen being to coat the sand particles and bind them together. Usually, 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. After some time, the permeability is very similar to that of the sand from which it is made. Lean sand asphalt can be used as a filter layer, core material or as a temporary protective covering.

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 below water. It can be handled and spread by means of a back-acter excavator, bulldozers or barge.

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.

65.4 Uses of bituminous materials

65.4.1 *General*

The choice of the various materials available for different maritime protection works is summarized in Table 26.

65.4.2 *Revetments for dykes, closure dams, dunes and sea walls*

These structures are listed in order of decreasing water-retaining function and 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 normally being uneconomic.

65.4.3 *Underwater seabed protection*

Sand mastic in the form of a carpet is suitable material for the protection of sandy seabeds. In general, waves do not play a significant role in the loading conditions. Uplift pressures can develop when the carpet is applied in a closure gap of an estuary or sea arm.

65.4.4 *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 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 can accommodate differential movement due to settlement without cracking.

Table 26 — Possible 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	Groynes	Breakwaters	Sills in closure gaps
Asphaltic concrete	Revetment above high-water level	Revetment above high-water level	Aprons placed in the dry	Special cases (capping)	—	—
Sand mastic grouting	Grouting of stone revetment	Grouting of stone revetment	—	Revetment and capping Grouting up stone	Moderately attacked revetment only	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 Revetment	Toe protection Revetment	Special cases	Toe protection	Toe protection	—
Open stone asphalt	Revetment	Revetment either direct or by grouting of heavy rubble	Only as prefabricated mattresses	Revetment either direct or by grouting of heavy rubble	Grouting of heavy rubble	—
Dense stone asphalt	—	Revetment	—	Revetment	Revetment	—
Lean sand asphalt	Core, fill, filter layer	Core, fill, filter layer	—	Core	Special cases (core) filter layer	Core

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66 Protective measures

66.1 General

When designing structures that are especially vulnerable to deterioration, because of the aggressive environment in which they are sited, the life of the structure can be increased in one of two ways. Allowance can either be made for planned maintenance, as recommended in Parts 1 to 8 of BS EN ISO 12944 and BS EN ISO 14713, or for increased initial structural strength, which will ensure the required amount of life without maintenance work.

The detailed planning and selection of materials and workmanship for maritime structures, as recommended in clause **59**, can minimize corrosion and the subsequent effects of corrosion. Reference should also be made to Parts 1 to 8 of BS EN ISO 12944 and BS EN ISO 14713 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 downtime 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. The corrosion allowance should be varied in order to allow for this difference (see **59.2.2**).

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.

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 that might usefully be painted are the splash and atmospheric zones.

f) Potential for accelerated low water corrosion (see **59.2.2**).

66.2 Coating systems

Parts 1 to 8 of BS EN ISO 12944 and BS EN ISO 14713 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 that 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 that can be applied and maintained with conventional and readily available application equipment.

c) 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 that have high impact resistance and resistance to spills of solvents and corrosive chemicals, i.e. the thermosetting materials.

d) It should be noted that discontinuities in the coating could encourage corrosion by forming galvanic cells with adjacent protected steel.

66.3 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. Concrete protection could also be used against concentrated corrosion (see **59.2.3**), but see comments on interface corrosion in **59.2.5**.

The placing of concrete for protection is often done underwater 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 highly dense concrete can result from this procedure.

66.4 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.

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66.5 Steel wear plates

Sufficient additional thickness of steel can 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.

66.6 Wrappings

66.6.1 ***General***

Wrappings are described in Parts 1 to 8 of BS EN ISO 12944 and BS EN ISO 14713, but in connection with maritime work points given in **66.6.2** to **66.6.5** require special consideration.

66.6.2 ***Structural members***

Structural members can be wrapped with glass fibre impregnated with a water-repellent compound of good adhesive properties. Hessian-reinforced wrappings should be used with caution, because these are susceptible to microbiological attack.

66.6.3 ***Wrappings***

The wrappings can be applied to structural members, preferably cylindrical, prior to installation below water or, in case of maintenance, 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.

66.6.4 ***Waterproofing compounds used for wrapping tapes***

66.6.4.1 *General*

The wrapping tapes described in **66.6.4.2** to **66.6.4.6** are used extensively for the protection of tie rods on anchored, sheet-piled structures.

The tie rods and cables are also available with protective plastic sleeves applied during manufacture.

66.6.4.2 *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. Tapes impregnated with this compound should therefore not be used where they are liable to impact, such as near the berthing face of a maritime structure.

66.6.4.3 *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.

66.6.4.4 *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.

66.6.4.5 *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.

66.6.5 ***Special measures***

In flowing water, special measures have to be taken to prevent wrappings being torn off by turbulence. Flexible thick plastic jackets are available for protecting pile wrappings in the surf zone where abrasion from beach or seabed material adds to the problem.

66.6.6 ***Cathodic protection***

The electrochemical processes that accompany corrosion of submerged structural elements in seawater are described in BS 7361, 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. Cathodic protection can combat concentrated corrosion of the type described in **59.2.2**.

66.6.7 ***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 riverbed level, the rates of corrosion are negligible, irrespective of the presence of anaerobic sulfate-reducing bacteria. However, in filled ground, especially with certain clays, bacterial corrosion can occur locally and can 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 BS 7361-1:1991). By provision of adequate cathodic protection this form of corrosion can be eliminated.

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67 Maintenance

67.1 General

Maintenance includes the following:

a) regular inspections;

b) in cases where planned maintenance is required to maintain durability (**66.1**):

1) renewal of protective systems;

2) repair of structural components.

c) special inspections after a collision, severe storm or other extreme event;

d) repairs following such accident or event.

Consideration should be given at the design stage to practical solutions to potential situations requiring maintenance or repairs (e.g. ship damage to fenders or the quay, or concentrated corrosion), which have not been allowed for in the original construction.

67.2 Records

It is recommended that records of inspections and maintenance work carried out supplement design and construction records. All such records should be maintained for the working life of the structure.

67.3 Access

In order to provide safe access it is recommended that permanent fixings be provided for mooring workboats and, where appropriate, for supporting temporary staging, so that inspection and maintenance can be facilitated.

Guidance on the inspection of rubble mound breakwaters is given in BS 6349-7:1991.

Annex A (informative)

Physical properties of commonly stored cargoes

Typical values of bulk densities and angles of repose are given in Table A.1. Typical values of stacked densities are given in Table A.2. These tables can be used in conjunction with the recommendations of clause **44**.

Table A.1 — Typical dry bulk densities and angles of repose

Material	Dry bulk density t/m^3	Angle of repose 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		
Sulfur	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

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Table A.2 — 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
Fertilizers (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

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Bibliography

Standards publications

BS 65, *Specification for vitrified clay pipes, fittings and joints.*

BS 410, *Specification for test sieves.*

BS 449-2, *Specification for the use of structural steel in building — Part 2: Metric units.*

BS 534, *Specification for steel pipes and specials for water and sewage.*

BS 639, *Specification for covered carbon and carbon manganese steel electrodes for manual metal-arc welding.*

BS 718, *Specification for density hydrometers.*

BS 729, *Specification for hot dip galvanized coatings on iron and steel articles.*

BS 916, *Specification for black bolts, screws and nuts, hexagon and square, with B.S.W. threads, and partly machined bolts, screws and nuts, hexagon and square, with B.S.W. or B.S.F. threads.*

BS 970-1, *Specification for wrought steels for mechanical and allied engineering purposes — Part 1: General inspection and testing procedures and specific requirements for carbon, carbon manganese and stainless steels.*

BS 1199 and BS 1200, *Specifications for building sands from natural sources.*

BS 1210, *Materials, dimensions, tolerances, designation criteria for screws of steel, stainless steel, brass, aluminium, silicon bronze, nickel copper alloy. Suitable finishes. Preferred and supplementary sizes for countersunk head, round head and raised countersunk head screws, for both slotted and recessed head types. Recess penetration gauging appended.*

BS 1490, *Specification for aluminium and aluminium alloy ingots and castings for general engineering purposes.*

BS 2760, *Specification for pitch-impregnated fibre pipes and fittings for below and above ground drainage.*

BS 2901 (all parts), *Filler rods and wires for gas-shielded arc welding.*

BS 3294, *Specification for the use of high strength friction grip bolts in structural steelwork.*

BS 3506, *Specification for unplasticized PVC pipe for industrial purposes.*

BS 3601, *Specification for carbon steel pipes and tubes with specified room temperature properties for pressure purposes.*

BS 3796, *Specification for polythene pipe (Type 50) for general purposes including chemical and food industry uses.*

BS 5400-2, *Steel, concrete and composite bridges — Part 2: Specification for loads.*

BS 5911-103, *Precast concrete pipes, fittings and ancillary products — Part 103: Specification for prestressed non-pressure pipes and fittings with flexible joints.*

BS 5930, *Code of practice for site investigations (formerly CP 2001).*

BS 6031, *Code of practice for earthworks.*

BS 6073-1, *Precast concrete masonry units — Part 1: Specification for precast concrete masonry units.*

BS 6349-2, *Code of practice for maritime structures — Part 2: Design of quay walls, jetties and dolphins.*

BS 6564-2, *Polytetrafluoroethylene (PTFE) materials and products — Part 2: Specification for fabricated unfilled.*

BS 7361-1, *Cathodic protection — Part 1: Code of practice for land and marine applications.*

BS 7543, *Guide to durability of buildings and building components, products and components.*

BS 7613, *Specification for hot rolled quenched and tempered weldable structural steel plates.*

BS 7668, *Specification for weldable structural steels. Hot finished structural hollow sections in weather resistant steels.*

BS 8004, *Code of practice for foundations.*

BS 8007, *Code of practice for design of concrete structures for retaining aqueous liquids.*

BS 8081, *Code of practice for ground anchorages.*

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BS 8118-1, *Structural use of aluminium — Part 1: Code of practice for design.*

BS 8118-2, *Structural use of aluminium — Part 2: Specification for materials, workmanship and protection.*

CP 3: Chapter V: Part 1, *Code of basic data for the design of buildings — Chapter V: Loading — Part 1: Dead and imposed loads.*

CP 3: Chapter V: Part 2, *Code of basic data for the design of buildings — Chapter V: Loading — Part 2: Wind loads.*

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BS EN 545, *Ductile iron pipes, fittings, accessories and their joints for water pipelines. Requirements and test methods*

BS EN 598, *Ductile iron pipes, fittings, accessories and their joints for sewerage applications — Requirements and test methods.*

BS EN 772-3, *Methods of test for masonry units — Part 3: Determination of net volume and percentage of voids of clay masonry units by hydrostatic weighing.*

BS EN 772-7, *Methods of test for masonry units — Part 7: Determination of water absorption of clay masonry damp proof course units by boiling in water.*

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