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Part 1-2: General – Code of practice for assessment of actions

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Contents

List of tables

Summary of pages

This document comprises a front cover, an inside front cover, pages i to vi, pages 1 to 138, an inside back cover and a back cover.

Foreword

Publishing information

This part of BS 6349 is published by BSI Standards Limited, under licence from The British Standards Institution, and came into effect on 30 June 2016. It was prepared by Technical Committee CB/502, *Maritime works*. A list of organizations represented on this committee can be obtained on request to its secretary.

Supersession

Together with BS 6349-1-1, BS 6349-1-3 and BS 6349-1-4, this part of BS 6349 supersedes BS 6349-1:2000, which is withdrawn.

Relationship with other publications

BS 6349 is published in the following parts:

- Part 1-1: *General – Code of practice for planning and design for operations*;
- Part 1-2: *General – Code of practice for assessment of actions*;
- Part 1-3: *General – Code of practice for geotechnical design*;
- Part 1-4: *General – Code of practice for materials*;
- Part 2: *Code of practice for the design of quay walls, jetties and dolphins*;
- Part 3: *Code of practice for the design of shipyards and sea locks*;
- 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*;
- Part 8: *Code of practice for the design of Ro-Ro ramps, linkspans and walkways*.

Information about this document

A full revision of BS 6349-1:2000 has been undertaken and the principal change is to split the document into four smaller parts:

- BS 6349-1-1: *Code of practice for planning and design for operations*;
- BS 6349-1-2: *Code of practice for assessment of actions*;
- BS 6349-1-3: *Code of practice for geotechnical design*;
- BS 6349-1-4: *Code of practice for materials*.

The principal changes in respect of the actions content are:

- incorporation of information regarding partial factors for limit state design approaches and actions previously covered in other parts of the BS 6349 series;
- substantial changes to content relating to sea-state and loads, movements and vibrations, to reflect scientific and technological advances since preparation of the previous version of BS 6349-1.

This revision also updates and replaces the recommendations given in BS 6349-2:2010, **5.1**, **5.2**, Annex A and Annex B, which will be removed from BS 6349-2 at its next revision.

Use of this document

As a code of practice, this part of BS 6349 takes the form of guidance and recommendations. It should not be quoted as if it were a specification and particular care should be taken to ensure that claims of compliance are not misleading.

Any user claiming compliance with this British Standard is expected to be able to justify any course of action that deviates from its recommendations.

Text introduced or altered by Corrigendum No. 1 is indicated in the text by tags [C1⟩ ⟨C1]. Minor editorial corrections are not tagged.

Presentational conventions

The provisions in this standard are presented in roman (i.e. upright) type. Its recommendations are expressed in sentences in which the principal auxiliary verb is "should".

Commentary, explanation and general informative material is presented in smaller italic type, and does not constitute a normative element.

Where words have alternative spellings, the preferred spelling of the Shorter Oxford English Dictionary is used (e.g. "organization" rather than "organisation").

Contractual and legal considerations

This publication does not purport to include all the necessary provisions of a contract. Users are responsible for its correct application.

Compliance with a British Standard cannot confer immunity from legal obligations.

Section 1: General

1 Scope

This part of BS 6349 gives recommendations for the assessment of actions for the planning and design of maritime works.

2 Normative references

The following documents, in whole or in part, are normatively referenced in this document and are indispensable for its application. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

Standards publications

BS 6349-1-1:2013, *Maritime works – Part 1-1: General – Code of practice for planning and design for operations*

BS 6349-4:2014, *Maritime works – Part 4: Code of practice for design of fendering and mooring systems*

BS EN 1990:2002+A1:2005, *Eurocode – Basis of structural design*

BS EN 1991 (all parts), *Eurocode 1 – Actions on structures*

BS EN 1992 (all parts), *Eurocode 2 – Design of concrete structures*

BS EN 1993 (all parts), *Eurocode 3 – Design of steel structures*

BS EN 1994 (all parts), *Eurocode 4 – Design of composite steel and concrete structures*

BS EN 1995 (all parts), *Eurocode 5 – Design of timber structures*

BS EN 1996 (all parts), *Eurocode 6 – Design of masonry structures*

BS EN 1997 (all parts), *Eurocode 7 – Geotechnical design*

BS EN 1998 (all parts), *Eurocode 8 – Design of structures for earthquake resistance*

BS EN 1999 (all parts), *Eurocode 9 – Design of aluminium structures*

ISO 21650:2007, *Actions from waves and currents on coastal structures*

NA to BS EN 1990:2002+A1:2005, *UK National Annex for Eurocode – Basis of structural design*

NA to BS EN 1991-1-3, *UK National Annex to Eurocode 1 – Actions on structures – Part 1-3: General actions – Snow loads*

Other publications

[N1] AMERICAN SOCIETY OF CIVIL ENGINEERS. *Seismic design of piers and wharves*. ASCE 61-14. Reston, VA: ASCE, 2014.

[N2] OIL COMPANIES INTERNATIONAL MARINE FORUM. *Mooring equipment guidelines*. Third edition (MEG 3). London: OCIMF, 2008.

3 Terms, definitions, symbols and abbreviations

3.1 Terms and definitions

For the purposes of this part of BS 6349, the terms and definitions given in BS EN 1990:2002+A1 and the following apply.

NOTE Where possible, definitions of meteorological and oceanographic terms are harmonized with BS EN ISO 19901, although some modifications are made to reflect the particular characteristics of the coastal environment within the scope of this part of BS 6349.

3.1.1 accidental operating condition

condition for a design situation when a facility is considered to be in operational use by ships berthing, de-berthing or in a moored condition consistent with the operating limits for the facility, but exceptional conditions occur due to deviation from facility operational procedures, or equipment malfunction

3.1.2 chart datum

local reference datum used to define water depths on a navigation chart or tidal heights over an area

NOTE Chart datum is usually an approximation to the level of the lowest astronomical tide.

3.1.3 concept design

design and engineering of the maritime works and preliminary planning for execution, in which site-specific data acquisition requirements are established and acquisition commences, and the level of definition is sufficient to select preferred technical options as the basis for detailed design

3.1.4 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.

3.1.5 design stage operating limits (DSOL)

preliminary assessment of environmental operating limits established and developed in the planning and design stages for the purposes of design of berths, channels, turning areas and other such works, and for making design-stage estimates of weather downtime

3.1.6 design working life

assumed period for which a structure or part of it is to be used for its intended purpose with anticipated maintenance but without major repair being necessary

[SOURCE: BS EN 1990:2002+A1:2005, **1.5.2.8**]

3.1.7 detailed design

design and engineering of maritime works including site-specific data acquisition and detailed planning for execution, in which the level of definition is sufficient for construction

NOTE In some industries, including the oil, gas and petrochemical industries, this phase can commence with front end engineering design (FEED) with detailed engineering completed within an engineering, procurement and construction (EPC) contract.

3.1.8 diffraction

<of waves> bending, spreading and interference of waves when they pass by an obstruction (e.g. a breakwater) or through a gap (e.g. a harbour entrance)

3.1.9 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.

3.1.10 environmental operating limits

limiting values of metocean or other environmental parameters, including wind, wave, swell, current velocity, tidal elevation, visibility and temperature, beyond which certain operations are not permitted to be carried out as set out in the facility operating manual

3.1.11 execution

activities required to construct and install maritime works, including off-site fabrication and commissioning when necessary so that the completed works are ready for handover to the owner and operator

3.1.12 extreme high water

highest level that is predicted to occur at a location as a combination of astronomical tides, positive storm surges, seiches and river flow for an extreme event of a defined return period

3.1.13 extreme low water

lowest level that is predicted to occur at a location as a combination of astronomical tides, negative storm surges, seiches and river flow for an extreme event of a defined return period

3.1.14 extreme operating condition

condition for a design situation when a facility is subject to extreme environmental conditions exceeding the DSOL (**3.1.5**) whether or not in use by ships for berthing, de-berthing, or mooring

NOTE Extreme operating conditions might include extreme environmental conditions of different return periods. Typically for permanent structures this could be events of 50-year to 100-year return periods considered as persistent design situations when designing to BS EN 1990:2002+A1, and events of 500 years to 1 000 years considered as accidental design situations when designing structures of a certain consequence class to BS EN 1990:2002+A1.

3.1.15 facility operating manual

procedures and instructions established by an operator to define procedures, environmental operating limits and other such matters to ensure safe and efficient operation of the maritime works and facilities in the operation and maintenance phase

3.1.16 gust

brief rise and fall in wind speed lasting less than 1 min

3.1.17 infragravity wave

long period wave as bound wave associated with wave grouping of swell travelling over long distances, or as free wave propagating independently after interaction of bound wave with shallow coastlines

NOTE Wave energy in the periods range 25 s to 500 s can generally be classified as infragravity wave energy. Waves of periods longer than 500 s are likely to be associated with tsunamis and tides.

3.1.18 marine facility
facility required to receive ships at a coastal marine terminal, within or outside a protected port or offshore, including but not limited to fixed berths, jetties, piers, island berths, buoy mooring facilities, and liquid cargo transfer structures

3.1.19 marine growth
living organisms attached to a structure

3.1.20 mean sea level
average of all sea levels measured at hourly intervals over a complete astronomical tidal cycle of 18.6 years

NOTE Seasonal and inter-annual changes in mean sea level can be expected in some regions, and over many years the mean sea level can change.

3.1.21 mean wave period
ratio of the length of record to the number of zero-crossing waves

3.1.22 metocean parameters
meteorological and oceanographic design and operating parameters including wind, precipitation, atmospheric conditions, solar radiation, water levels, waves, water movements, sea ice and icebergs, water quality and physical and chemical properties and marine growth

3.1.23 normal operating condition
condition for a design situation when a facility is considered to be in operational use by ships berthing, de-berthing or in a moored condition consistent with the DSOL (**3.1.5**) for the facility

3.1.24 operation and maintenance
service usage of the completed maritime works by user, operator or owner, including planned and unplanned inspection, maintenance and repairs

NOTE Some works, such as ports and berthing structures, are usually actively operated by a harbour authority or terminal operator, whereas other works, such as coastal protection structures, might not be actively operated but are likely to be actively monitored and maintained.

3.1.25 operator
harbour authority, port operator, terminal operator or other such competent entity responsible for operating and maintaining a marine facility for use by vessels

3.1.26 reflection
<of waves> situation that occurs when waves reach an obstacle, e.g. a sea wall or a breakwater

NOTE Waves also reflect from beaches and at locations with sharp depth changes.

3.1.27 refraction
<of waves> bending of the wave propagation direction due to variations in the water depth under the waves

NOTE The part of a wave in shallow water moves slower than the part of a wave in deeper water, so when the depth under a wave crest varies along the crest, the wave "bends".

3.1.28 return period
average period between occurrences of an event or of a particular value being exceeded

NOTE The return period in years is equal to the reciprocal of the annual probability of exceedance of the event.

3.1.29 scatter diagram
graphic representation of the joint probability of two or more (metocean) parameters

NOTE Typically used with wave parameters to show the probability of the joint occurrence of the significant wave height (H_s) and a representative period (T_m or T_p).

3.1.30 sea state
condition of the sea during a period in which its statistics remain approximately constant

NOTE In a statistical sense the sea state does not change markedly within the period. The period during which this condition exists is usually assumed to be 3 h, although it depends on the particular weather situation at any given time.

3.1.31 seiche
oscillation of a body of water at its natural period

NOTE Seiches usually take the form of standing waves or sloshing/oscillations of the free surface. These oscillations can have periods from minutes in harbours and bays to over 10 h in large lakes.

3.1.32 ship
class of sea-going and inland vessels including general cargo ships, container ships, tankers and gas and liquid product carriers, cruise ships, Ro-Ro ships, bulk carriers

3.1.33 shoaling
<of waves> transformation of waves caused by change in depth alone as they enter shallower water

NOTE Shoaling occurs because the wave speed and wave length decrease in shallow water, therefore the energy per unit area of the wave has to increase, resulting in an increase in the wave height. In linear wave theory, the wave period remains the same in shoaling. Other shallow water transformation effects such as refraction arise separately from shoaling.

3.1.34 significant wave height
average height of the highest one third of the zero upcrossing waves in a sea state

NOTE In most measurement systems the significant wave height, H_s, is calculated as $4\sqrt{m_0}$, where m_0 is the zeroth spectral moment, or 4σ, where σ is the standard deviation of the time series of water surface elevation over the duration of the measurement, typically approximately 30 min.

3.1.35 significant wave period
average of the periods of the highest one third of the waves

NOTE Mean or peak wave 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 North American approaches.

3.1.36 still water level
theoretical water surface level in the absence of any wave effects

NOTE 1 Still water level is typically used for the calculation of wave kinematics for global actions and wave crest elevation for minimum deck elevations.

NOTE 2 Still water level is an abstract concept for engineering purposes calculated by adding the effects of tides, storm surge and allowances for future sea level change but normally excluding variations due to waves.

3.1.37 storm surge
change in sea level (either positive or negative) that is due to meteorological (rather than tidal) forcing

NOTE 1 Storm surges can occur on the open coast, on bays and on estuaries due to the action of wind stresses on the water surface, the atmospheric pressure reduction, storm-induced seiches, wave set-up and other causes.

NOTE 2 The term "surge" is also used in a different context to describe the longitudinal motion of a moored vessel.

3.1.38 swell
sea state in which waves generated by winds remote from the site have travelled to the site, rather than being locally generated

NOTE When categorizing wave types from a spectrum or from measurements, energy in the period range from 8 s to 25 s can typically be described as swell. Energy at periods longer than 25 s can be described as infragravity wave energy.

3.1.39 tides

3.1.39.1 astronomical tide
phenomenon of the alternate rising and falling of sea surface governed by astronomical conditions principally of the sun and the moon, which is predicted with the tidal components determined from harmonic analysis of tide level readings over a long period

3.1.39.2 lowest astronomical tide (LAT)
level of low tide when all harmonic components causing the tides are in phase

NOTE The harmonic components are in phase approximately once every 18.6 years but a level equivalent to LAT is approached several times each year at most locations. LAT does not represent the lowest sea level which can be reached, because negative surges and tsunamis can cause considerably lower levels to occur. LAT is often the level selected as the chart datum for soundings on navigational charts.

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

3.1.40 tsunami
long period sea waves caused by rapid vertical movements of the sea floor due to earthquakes, or by submarine or coastal landslip

3.1.41 vessel
craft that travels on water, including coastal and sea-going ships, inland and sea-going barges, workboats, tugs, ferries, trawlers and fishing vessels, small recreational or pleasure craft

NOTE Small vessels are considered as those of less than 24 m length.

3.1.42 wave frequency
inverse of wave period

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

NOTE A train of waves of single period travelling in still water propagate at a velocity less than the phase velocity of the individual waves. Waves are created at the rear of the train, move through the train and die out at the wave front.

3.1.44 wave height
height of a wave crest above the preceding wave trough

3.1.45 wave length
horizontal distance between two successive crests or troughs in a wave record

3.1.46 wave phase velocity
speed at which a wave propagates (sometimes referred to as wave celerity or velocity of wave propagation)

3.1.47 wave period
time for two successive wave crests to pass a fixed point

3.1.48 wave spectrum
measure of the amount of energy associated with the fluctuation of the sea surface elevation per unit frequency band and per unit directional sector

NOTE 1 The wave frequency spectrum (integrated over all directions) is often described by use of some parametric form such as the Pierson–Moskowitz or JONSWAP wave spectrum.

NOTE 2 The area under the wave spectrum is the zeroth spectral moment m_0, which is a measure of the total energy in the sea state and can be used to calculate significant wave height.

3.1.49 wave steepness
wave height divided by wave length, H/L

3.2 Symbols

A	cross-sectional area of a member (m²)
A_d	design value of accidental action
A_{Ed}	design value of seismic action
A_{Ek}	characteristic value of the seismic action for the reference return period
A_L	longitudinal projected area of a vessel above the waterline (m²)
A_n	area of structural member normal to flow
a	dimensionless coefficient used in JONSWAP wave spectrum
a_{gR}	reference peak ground acceleration
a_{g2}	design ground acceleration on type A ground for the controlled and repairable damage criterion
B_Q	adjustment factor of load model 2
b	distance between adjacent wave orthogonals (m)
b_o	wave ray separation in deep water
C_{CL}	depth correction factor for longitudinal current forces
C_{CT}	depth correction factor for transverse current drag forces
C_D	dimensionless time-averaged drag coefficient for steady flow
C_I	inertia force coefficient
C_{LC}	longitudinal current drag force coefficient
C_{LW}	longitudinal wind force coefficient
C_{TC}	transverse current drag force coefficient, forward or aft
C_{TW}	transverse wind force coefficient, forward or aft
c_g	wave group velocity (m/s)
d	still water depth (m)

d_c	maximum displacement under any applied cyclic loading of maximum value, P, and frequency of application, f_c
d_m	mean draught of a vessel
$d_{set,i}$	difference in settlements between individual foundations or parts of foundations compared to a reference level for a specific element
E	wave energy density
F_D	steady drag force (kN)
F_b	frictional force per unit area (N/m^2)
F_I	inertia force component
F_{oS}	factor of safety for fatigue analysis
F_W	total wave force normal to the axis of a member
F_{LC}	longitudinal current force (kN)
F_{LW}	longitudinal wind force (kN)
F_{TC}	transverse current force, forward or aft (kN)
F_{TW}	transverse wind force, forward or aft (kN)
f	wave frequency (Hz)
f_c	frequency of a forcing cyclic load
f_N	natural frequency of a structure
f_p	frequency at which the peak occurs in the spectrum
$G_{kj,sup}$	upper characteristic value of permanent action j
$G_{kj,inf}$	lower characteristic value of permanent action j
G_{set}	characteristic value of a permanent action caused by uneven settlements on the structure due to soil subsidence
g	acceleration due to gravity (m/s^2)
H	wave height (m)
H_L	limiting wave height in probability distributions (m)
H_b	breaking wave height (m)
H_{max}	maximum wave height (m)
H_n	nth height or height interval in a rank ordered set of n_x values
H_{m0}	spectral significant wave height (m)
H_o	deep water wave height (m)
H_s	significant wave height (m)
H'_o	equivalent deep water wave height (m)
$H_{1/250}$	mean height of the 0.4% highest waves (m)
h	water depth (m)
h_b	water depth associated with wave breaking point (m)
K	dimensionless constant related to flow-induced oscillation
K_b	bed friction factor
K_f	wave height reduction factor
K_p	percent reduction in wave height
K_r	wave refraction coefficient

K_s	wave shoaling coefficient
K_1	dimensionless constant related to flow-induced oscillation (rectangular section)
K_2	dimensionless constant related to flow-induced oscillation (lozenge section)
K_3	dimensionless constant related to flow-induced oscillation (square section with corner projections)
k	wave number
k_J	dimensionless coefficient used in JONSWAP wave spectrum
k_P	dimensionless coefficient used in Pierson-Moskowitz wave spectrum
k_s	stiffness of equivalent spring
L	wave length (m)
L'	overall length of a cylinder measured from the apparent fixity level to deck level (m)
L_{BP}	length between perpendiculars of a vessel (m)
L_F	fetch length (m)
L_o	wave length in deep water (m)
L_s	submerged length of a member (m)
l'	length from apparent fixity level to water level (m)
$I_D(\theta)$	wave diffraction intensity factor appropriate to the incident wave direction and harbour entrance width
I_r	Iribarren number
m_e	equivalent mass of a structure
m_L	mass per unit length of a cylinder, in kilograms per metre (kg/m)
m_0	zeroth spectral moment
$\bar{m}$	equivalent excited effective mass per unit length (kg/m)
N_i	number of waves in stress range *i* needed to cause failure
n	number between 1 and n_x (of any variable, e.g. years or events)
n_i	number of waves occurring during the design life in stress range *i*
n_T	total number of stress ranges
P	relevant representative value of a pre-stressing action
P_{DLR}	reference probability of exceedance of the seismic action in 50 years [or the design life of the structure if different] for the damage limitation requirement
P_{NCR}	reference probability of exceedance of the seismic action in 50 years [or the design life of the structure if different] for the no-collapse requirement
p	probability of a particular extreme condition occurring during design working life *n* years
p_n	probability of wave height H_n being equalled or exceeded
Q_{ca}	characteristic value of construction loads due to working personnel
Q_{cb}	characteristic value of construction loads due to storage of moveable items

Q_{cc} characteristic value of construction loads due to non-permanent equipment in position for use during execution

Q_{cd} characteristic value of construction loads due to moveable heavy machinery and equipment

Q_{ce} characteristic value of construction loads from accumulation of waste materials

Q_{cf} characteristic value of construction loads from parts of a structure in temporary states (under execution) before the final design actions take effect

$Q_{C,k}$ characteristic value of construction loads

Q_{fwk} characteristic value of the concentrated load (wheel load) on a footbridge

$Q_{k,i}$ characteristic value of the accompanying variable action *i*

$Q_{k,1}$ characteristic value of the leading variable action *1*

q proportion of critical damping, equal to between 0.01 and 0.05 for maritime structures

$S(f)$ one dimensional spectral density function

s_d tangent of seabed slope relative to horizontal

T wave period (s)

T_{DLR} reference return period for the damage limitation requirement

T_E mean energy period (s)

T_{NCR} reference return period for the no-collapse requirement

T_R return period of an extreme condition in years

T_S significant wave period (s)

T_m mean wave period (s)

$T_{m-1,0}$ spectral mean energy wave period (s)

T_p spectral peak wave period (s)

T_z zero-crossing wave period (s)

T_0 period or duration of observation

$T_{1/3}$ significant wave period (=T_s)

t time variable

U water particle velocity (m/s)

$\dot{U}$ water particle acceleration (m/s^2)

U_w wind speed 10 m above the sea surface

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

u horizontal component of water particle velocity (m/s)

$\dot{u}$ horizontal component of water particle acceleration (m/s^2)

V incident current velocity (m/s)

V_W design wind speed (m/s)

$V_{b,Q}$ value of basic wind velocity (m/s)

V_{crit} critical flow velocity (m/s)

V_c'	current velocity, averaged over the mean draught of the vessel, of the component of current in the direction under consideration, transverse or longitudinal
v	vertical component of water particle velocity (m/s)
v_c	velocity of wave propagation (m/s)
v_{co}	velocity of wave propagation in deep water (m/s)
v_{cg}	wave group velocity (m/s)
v_{cgo}	wave group velocity in deep water (m/s)
$\dot{v}$	vertical component of water particle acceleration (m/s²)
W_S	width or diameter of the submerged part of the structure or member (m)
W_s	pile diameter (m)
w_p	orbit width of water particles at the surface
x	horizontal distance from a defined datum (m)
Y_a	characteristic dimension of a submerged structural element
Y_b	characteristic dimension of a submerged structural element
Y_c	characteristic dimension of a submerged structural element
y	vertical distance from a defined datum (m)
$y(x)$	bending mode shape as a function of the ordinate, x, measured from the apparent fixity level
z	vertical distance from still water level (m)
α	angle of wind off bow, in degrees
α_c	angle of current relative to member axis
α_s	seabed slope angle
β	bed slope
β_0	dimensionless coefficient used in wave shoaling and breaking estimation
β_0*	dimensionless coefficient used in wave shoaling and breaking estimation
β_1	dimensionless coefficient used in wave shoaling and breaking estimation
β_1*	dimensionless coefficient used in wave shoaling and breaking estimation
β_{max}	dimensionless coefficient used in wave shoaling and breaking estimation
β_{max}*	dimensionless coefficient used in wave shoaling and breaking estimation
γ	peak enhancement factor (for wave spectra)
γ_{br}	depth-limited breaker index
$\gamma_{G,inf}$	partial factor for permanent action in calculating lower design values (see Table 1, Note A)
$\gamma_{G,sup}$	partial factor for permanent action in calculating upper design values (see Table 1, Note A)

$\gamma_{Gj,inf}$	partial factor for permanent action *j* in calculating lower design values
$\gamma_{Gj,sup}$	partial factor for permanent action *j* in calculating upper design values
γ_I	importance factor
γ_p	partial factor for prestressing actions
γ_Q	partial factor for variable actions
$\gamma_{Q,i}$	partial factor for variable action *i*
$\gamma_{Q,1}$	partial factor for the leading variable action
γ_2	performance factor for modifying the level 2 earthquake seismic action to be used in the controlled and repairable damage criterion
Δ	logarithmic decrement of structural damping
$\Delta_{dset,i}$	uncertainty value attached to the assessment of settlements of a foundation or part of a foundation for a specific element
δ_s	angular ray separation at the harbour entrance
ζ_v	vertical displacement of the water particle from its mean position
ζ_h	horizontal displacement of the water particle from its mean position
η	instantaneous water surface level (e.g. of a wave profile)
θ	angle between the plane across which energy is being transmitted and the direction of wave advance
μ	coefficient of friction between two faces in contact
ξ	surf similarity parameter
ρ	mass density of water (kg/m^3)
ρ_A	mass density of air (kg/m^3)
σ	standard deviation of the time series of water surface elevation over the duration of a measurement (min)
ψ	factor for the accompanying value of a variable action
ψ_0	factor for combination value of a variable action
$\Psi_{0,i}$	factor for the combination value of a specific variable action *i*
ψ_1	factor for frequent value of a variable action
$\Psi_{1,i}$	factor for the frequent value of a specific variable action *i*
$\Psi_{1,1}$	factor for the frequent value of the leading specific variable action
ψ_2	factor for quasi-permanent value of a variable action
$\Psi_{2,i}$	factor for the quasi permanent value of a specific variable action *i*
$\Psi_{2,1}$	factor for the quasi permanent value of the leading variable action
ω	angular frequency

3.3 Abbreviations

For the purposes of this part of BS 6349, the following abbreviations apply.

CFD	computational fluid dynamics
DGPS	differential global positioning system
DSOL	design stage operating limits
DWT	deadweight tonnage
EMS	environmental management system
GPS	global positioning system
HAZID	hazard identification study
IFD	intensity, frequency and duration
LAT	lowest astronomical tide
LiDAR	light detection and ranging
LNG	liquefied natural gas
LPG	liquefied petroleum gas
MBES	multi-beam echo sounder
MBL	minimum breaking load
MM	manoeuvrability margin
Ro-Ro	roll-on roll-off
SBES	single-beam echo sounder
SLS	serviceability limit state
SMP	shoreline management plan
SMS	safety management system
SWL	safe working load
TEU	twenty-foot equivalent unit
UKC	under-keel clearance
ULS	ultimate limit state
VLCC	very large crude carrier
VOS	voluntary observing ships

Section 2: Combinations of actions and structural design

4 Limit state design principles

4.1 General

The strength and stability of maritime structures should be verified taking into account the following ultimate limit states as defined in BS EN 1990:2002+A1:

- EQU: loss of static equilibrium of the structure or any part of it treated as a rigid body, where:
  - minor variations in the value or the spatial distribution of actions from a single source are significant; and
  - the strengths of construction materials or ground are generally not governing;
- STR: internal failure or excessive deformation of the structure or structural members, where the strength of construction materials of the structure governs;
- GEO: failure or excessive deformation of the ground where the strengths of soil or rock are significant in providing resistance;
- FAT: fatigue failure of the structure or structural members.

Ultimate limit states UPL (uplift/buoyancy) and HYD (hydraulic heave/gradients) as defined should be verified in accordance with BS EN 1997.

4.2 Partial factors and combination formulae

Combinations of actions should include permanent actions and variable actions. Internal actions such as pre-stress should be included where appropriate.

Partial factors and combinations of actions should be based upon BS EN 1990:2002+A1 and the recommendations given in this part of BS 6349. In particular, the following should be taken into account:

a) wave actions, which should be treated as variable actions in accordance with BS EN 1990:2002+A1;

b) current actions, which should be treated as variable actions in accordance with BS EN 1990:2002+A1;

c) actions associated with ship operations (actions due to berthing, mooring, ships propulsion and ships ramps), which should be treated as variable actions in accordance with BS EN 1990:2002+A1;

d) ship accidental impact actions, which should be treated as accidental actions in accordance with BS EN 1990:2002+A1;

e) actions associated with port operations (port vehicles, cranes and cargo), which should be treated as variable actions in accordance with BS EN 1990:2002+A1;

NOTE 1 The self-weight of fixed or mobile cranes may be treated as a permanent action, and the variable actions for the cranes are then treated as cargo actions or environmental actions.

f) combinations where wind loads on structures, buildings, linkspans, walkways or cranes, and wave loads on maritime structures, are present at the same time in a design situation, which should be assumed to be variable actions;

g) partial factors for the individual permanent actions and the variable actions, which should be taken from Table 1.

NOTE 2 If the combination includes the permanent actions and only one variable action, then the partial factors listed in Table 1 are the only factors required to derive the design actions that are used in the verification.

If the combination includes permanent actions and more than one variable action, then one of the variable actions should be designated as the leading variable action and the others should be assumed to be the accompanying variable actions. The permanent actions and leading variable action should be enhanced by the partial factors listed in Table 1, but the accompanying variable actions should be multiplied by the ψ factors, which are factors for the combination value ψ_0, frequent value ψ_1 or the quasi-permanent value ψ_2 of the variable actions.

NOTE 3 These represent the reduced probability that the maximum effects occur simultaneously when dealing with more than one statistically independent variable action. Where actions arise from the same cause, then they may be treated jointly as the leading or secondary variable action, as appropriate.

Where appropriate, the design should be assessed for a variety of design situations by taking different variable actions to be the leading variable action until the combination resulting in the most adverse structural response is identified.

NOTE 4 This means that a different variable is reduced by the ψ factor in each of the different design situations.

The formulae given in Table 2 should be used to determine the way in which the partial factors and ψ factors should be applied for combinations of permanent actions, pre-stress actions and variable actions.

Structures classified as highway, rail or foot bridges should be designed in accordance with BS EN 1990:2002+A1, Annex A2 and the National Annex to BS EN 1990:2002+A1.

NOTE 5 BS EN 1990:2002+A1, Annex A2 does not give guidance on partial safety factors for wave and current loading, vessel impact and other actions that might be applicable to bridges passing over water. It is suggested that the load combination factors given here are appropriate for use with BS EN 1990:2002+A1, Annex A2 for the design of bridges in these situations.

Table 1 Partial factors for actions *(1 of 3)*

Action type	Scenario	Item	Symbol (in accordance with BS EN 1990:2002+A1)	EQU (Set A)		STR/GEO (Set B)		STR/GEO (Set C)	
				Unfavourable	Favourable	Unfavourable	Favourable	Unfavourable	Favourable
Permanent actions A)		Steel self-weight	$\gamma_{G,sup}$ $\gamma_{G,inf}$	1.05	0.95	1.20	0.95	1.00	1.00
		Concrete self-weight	$\gamma_{G,sup}$ $\gamma_{G,inf}$	1.05	0.95	1.35	0.95	1.00	1.00
		Weight of soil and other materials	$\gamma_{G,sup}$ $\gamma_{G,inf}$	1.05	0.95	1.35	0.95	1.00	1.00
		Superimposed dead loads B)	$\gamma_{G,sup}$ $\gamma_{G,inf}$	1.05	0.95	1.20	0.95	1.00	1.00
		Geotechnical actions	$\gamma_{G,sup}$ $\gamma_{G,inf}$	1.05	0.95	1.35	0.95	1.00	1.00
		Ground water C)	$\gamma_{G,sup}$ $\gamma_{G,inf}$	1.05	0.95	1.35	0.95	1.00	1.00
		Uneven settlements	$\gamma_{G,sup}$	—	—	1.20 D)	—	1.00	—
		Prestressing	γ_P as defined in the relevant Eurocode or for the individual project						
Variable actions	Road traffic actions	Road vehicles (gr1a)	γ_Q	1.35	—	1.35	—	1.15	—
		Pedestrian loads (gr1a)	γ_Q	1.35	—	1.35	—	1.15	—
		Horizontal forces (gr2)	γ_Q	1.35	—	1.35	—	1.15	—
		Pedestrian loads (gr3)	γ_Q	1.35	—	1.35	—	1.15	—
	Equipment loads E)	Gantry crane	γ_Q	1.35	—	1.35	—	1.15	—
		Mobile harbour crane	γ_Q	1.35	—	1.35	—	1.15	—
		Construction crane	γ_Q	1.35	—	1.35	—	1.15	—
		Port vehicles	γ_Q	1.35	—	1.35	—	1.15	—
		Cargo handling equipment	γ_Q	1.35	—	1.35	—	1.15	—
	Cargo loads	Containers	γ_Q	1.50	—	1.50	—	[C1⟩ 1.30 ⟨C1]	—
		General cargo	γ_Q	1.50	—	1.50	—	[C1⟩ 1.30 ⟨C1]	—
		Bulk cargo	γ_Q	1.50	—	1.50	—	[C1⟩ 1.30 ⟨C1]	—
		Liquid products	γ_Q	1.05	—	1.05	—	1.00	—

Table 1 Partial factors for actions *(2 of 3)*

Action type	Scenario	Item	Symbol (in accordance with BS EN 1990:2002+A1)	EQU (Set A)		STR/GEO (Set B)		STR/GEO (Set C)	
				Unfavourable	Favourable	Unfavourable	Favourable	Unfavourable	Favourable
Variable actions	Environmental loads	Wind actions [F)]	γ_Q	1.50	—	1.50	—	1.30	—
		Operational wind (an environmental operating limit as defined in BS 6349-1-1:2013)	γ_Q	1.50	—	1.50	—	1.30	—
		Thermal actions [F)]	γ_Q	1.50	—	1.50	—	1.20	—
		Snow	γ_Q	1.50	—	1.50	—	1.30	—
		Ice	γ_Q	1.50	—	1.50	—	1.30	—
		Water currents	γ_Q	1.50	—	1.50	—	1.30	—
		Wave	γ_Q	1.50	—	1.50	—	1.30	—
		Operational wave (an environmental operating limit as defined in BS 6349-1-1:2013)	γ_Q	1.50	—	1.50	—	1.30	—
		Tidal lag or variable ground water [C)]	γ_Q	1.50	—	1.50	—	1.30	—
	Ship operational loads [G)]	Berthing (from characteristic energy) [H)]	γ_Q	1.35	—	1.35	—	1.15	—
		Berthing (from design energy) [H)]	γ_Q	1.20	—	1.20	—	1.10	—
		Mooring	γ_Q	See Table 8	—	See Table 8	—	See Table 8	—
		Ship ramps	γ_Q	1.50	—	1.50	—	1.30	—
		Ships propulsion	γ_Q	1.50	—	1.50	—	1.30	—
	Construction loads (execution)		γ_Q	1.50	—	1.50	—	1.30	—
	Geotechnical actions [I)]	Earth pressures	γ_Q	—	—	—	—	—	—

Table 1 **Partial factors for actions** *(3 of 3)*

Action type	Scenario	Item	Symbol (in accordance with BS EN 1990:2002+A1)	EQU (Set A)		STR/GEO (Set B)		STR/GEO (Set C)	
				Unfavourable	Favourable	Unfavourable	Favourable	Unfavourable	Favourable

NOTE There are different sets of partial factors for different situations, the main ones being:

- *EQU: the set for the assessment of overall equilibrium states, such as overturning stability where the strength of structural members or the ground do not govern;*
- *STR: the set governing the assessment of structural strength where the structural strength governs;*
- *GEO: the set governing the assessment of failure or excessive deformation of the ground where the soil or rock strength governs;*
- *FAT: the set for dealing with the fatigue failure of structure or structural members;*
- *UPL: the set for dealing with the loss of equilibrium of the structure or the ground due to uplift by water pressure (buoyancy);*
- *HYD: the set for dealing with hydraulic heave, internal erosion and piping in the ground caused by hydraulic gradients.*

EQU, STR, GEO and FAT are defined in BS EN 1990:2002+A1. UPL and HYD are defined in BS EN 1997.

A) The characteristic values of all permanent actions from one source may be multiplied by $\gamma_{G,sup}$ if the total resulting action effect from this source is unfavourable, and by $\gamma_{G,inf}$ if the total resulting action from this source is favourable.

B) Superimposed dead load represents features such as surfacing. For surfacing, the possibility of re-surfacing during the design life of the structure should be taken into account by including an additional thickness.

C) For actions dependent on the level of water, the design values may be directly assessed in accordance with BS EN 1997. Alternatively a safety margin may be applied to the characteristic water level. However, the limiting water levels might be constrained by the geometry of the structure. The characteristic values of water pressure from one source, e.g. tide level, are multiplied by a single value of γ_Q or γ_G as appropriate. Partial factors for such actions may be determined for the individual project.

D) For design situations where actions due to uneven settlement have unfavourable effects. This should be increased to 1.35 in the case of a non-linear analysis. For design situations where actions due to uneven settlements have favourable effects, these actions are not to be taken into account.

E) The partial factors given in this table for cranes combine the self-weight and the variable actions. These can be separated, with the self-weight being treated as a permanent action for a superimposed dead weight and the variable component being treated as a cargo load for general cargo and/or as one or more environmental loads. Where the self-weight component of the crane actions is not known, then the combined actions should be regarded as a permanent action for calculations that require seismic or quasi-permanent effects to be taken into account.

F) The values of γ_Q for environmental actions in Table 1 are based on the characteristic values having a return period equal to the design working life of the structure but not less than 50 years for persistent situations. For transient design situations, the return period should be determined for each project taking into account the guidance given in BS EN 1991-1-6 (typically 10 years for transient situations which last for less than 1 year).

G) For partial factors related to loads on mooring structures, see **29.2**, Table 8.

H) The values of γ_Q for fender actions have been chosen acknowledging that BS 6349-4:2014 recommends a much wider range of factors to be considered than hitherto, thus reducing the probability of exceedance of the actions significantly.

I) No values are given because the design value of earth pressure should be based on the design values of variable actions giving rise to the earth pressure.

Table 2 **Formulae for combinations of actions**

Design situation	Limit state	Permanent actions	Pre-stress	Leading accidental or seismic action	Leading variable action [A]	Accompanying variable actions [A]	Eq. no. [B]
Ultimate limit states							
Persistent and transient	Unfavourable	$\Sigma\gamma_{Gj,sup}G_{kj,sup}$	$+ \gamma_P P$	—	$+ \gamma_{Q,1}Q_{k,1}$	$+ \Sigma\gamma_{Q,i}\psi_{0,i}Q_{k,i}\cdots$	6.10
	Favourable	$\Sigma\gamma_{Gj,inf}G_{kj,inf}$	$+ \gamma_P P$	—	$+ \gamma_{Q,1}Q_{k,1}$	$+ \Sigma\gamma_{Q,i}\psi_{0,i}Q_{k,i}\cdots$	6.10
Accidental	Unfavourable	**Either:** $\Sigma G_{kj,sup}$	$+ P$	$+ A_d$	$+ \psi_{1,1}Q_{k,1}$	$+ \Sigma\psi_{2,i}Q_{k,i}$	6.11b
		or: $\Sigma G_{kj,sup}$	$+ P$	$+ A_d$	$+ \psi_{2,1}Q_{k,1}$	$+ \Sigma\psi_{2,i}Q_{k,i}$	6.11b
	Favourable	**Either:** $\Sigma G_{kj,inf}$	$+ P$	$+ A_d$	$+ \psi_{1,1}Q_{k,1}$ [C]	$+ \Sigma\psi_{2,i}Q_{k,i}$	6.11b
		or: $\Sigma G_{kj,inf}$	$+ P$	$+ A_d$	$+ \psi_{2,1}Q_{k,1}$ [C]	$+ \Sigma\psi_{2,i}Q_{k,i}$	6.11b
Seismic	Unfavourable	$\Sigma G_{kj,sup}$	$+ P$	$+ A_{Ed}$ [D]	—	$+ \Sigma\psi_{2,i}Q_{k,i}$	6.12b
	Favourable	$\Sigma G_{kj,inf}$	$+ P$	$+ A_{Ed}$ [D]	—	$+ \Sigma\psi_{2,i}Q_{k,i}$	6.12b
Accidental construction (execution)	Unfavourable	$\Sigma G_{kj,sup}$	$+ P$	$+ A_d$	—	$+ \Sigma\psi_{2,}Q_{C,k}$ [E]	A2.2
	Favourable	$\Sigma G_{kj,inf}$	$+ P$	$+ A_d$	—	$+ \Sigma\psi_{2,}Q_{C,k}$ [E]	A2.2
Serviceability limit states							
Characteristic	Unfavourable	$\Sigma G_{kj,sup}$	$+ P$	—	$+ Q_{k,1}$	$+ \Sigma\psi_{0,i}Q_{k,i}$	6.14b
	Favourable	$\Sigma G_{kj,inf}$	$+ P$	—	$+ Q_{k,1}$	$+ \Sigma\psi_{0,i}Q_{k,i}$	6.14b
Frequent	Unfavourable	$\Sigma G_{kj,sup}$	$+ P$	—	$+ \psi_{1,1}Q_{k,1}$	$+ \Sigma\psi_{2,i}Q_{k,i}$	6.15b
	Favourable	$\Sigma G_{kj,inf}$	$+ P$	—	$+ \psi_{1,1}Q_{k,1}$	$+ \Sigma\psi_{2,i}Q_{k,i}$	6.15b
Quasi-permanent	Unfavourable	$\Sigma G_{kj,sup}$	$+ P$	—	—	$+ \Sigma\psi_{2,i}Q_{k,i}$	6.16b
	Favourable	$\Sigma G_{kj,inf}$	$+ P$	—	—	$+ \Sigma\psi_{2,i}Q_{k,i}$	6.16b

Key

\+ To be combined with

Σ The combined effect of

[A] Variable actions are listed in Table 1. Any one of the variable actions in a design situation should be classified as "Leading", the others then being regarded as "Accompanying".

[B] Refers to equation numbers in BS EN 1990:2002+A1.

[C] The choice between $\psi_{1,1}$ or $\psi_{2,1}$ should be related to the relevant accidental design situation.

[D] $A_{Ed} = \gamma_I A_{Ek}$ where the seismic effects are as defined in BS EN 1998.

[E] $Q_{C,k}$ is the characteristic value of the construction loads as defined in BS EN 1991-1-6.

5 Assessment of actions

The values of actions should be taken from Section **4**.

Design situations and combinations should be chosen using the recommendations given in Clause **6** and Clause **7**. The actions covered in these clauses are not exhaustive, and any other critical actions which are likely to occur should also be analysed.

6 Design situations and combinations of actions

6.1 Design situations

6.1.1 Persistent design situations

Persistent design situations should be as defined in BS EN 1990:2002+A1.

Conditions of normal use for a marine facility appropriate to assessment of actions and combinations of actions in a persistent design situation should include both normal and extreme operating conditions as defined in BS 6349-1-1:2013.

NOTE 1 Examples of persistent design situations are:

a) environmental actions having a return period corresponding to the design working life of the structure (but generally not less than 50 years);

b) water levels having a return period corresponding to the design working life of the structure;

c) overdredging of seabed within specified tolerances;

d) deepening of the seabed due to scour;

e) increase in hydrostatic head due to drawdown in an impounded basin occurring during planned inspections at intervals not exceeding 1 year;

f) operational range of tidal water levels that occur in combination with loads from port operations;

g) foreseeable modifications to the structure, earthworks, paving, storage patterns, handling equipment or dredged depth;

h) combinations of environmental and port operation conditions where operations are limited to certain pre-defined environmental conditions. An example is where a ship cannot berth in the conditions of item a) above and so operations are restricted to lower wind speeds. In these circumstances the operational loads do not occur in conjunction with wind speeds having a return period corresponding to the design working life as in item a) above;

i) actions arising from ship berthing operations leading to the characteristic berthing energy as described in BS 6349-4:2014 (although the actions arising from berthing operations leading to the design berthing energy as defined in BS 6349-4:2014 might cause actions that are just as large);

j) actions due to containers, using diversification factors for stacks more than one container high;

k) actions caused by normal port traffic use;

l) combinations of wind and wave actions where either:

- *the wind is the extreme wind action including gust enhancements as defined in BS EN 1991-4 and is in the wave action is that arising from the significant and not the maximum wave height; or*

- *the wave is the wave action arising from the extreme maximum wave height and the wind is the extreme wind action but without the gust enhancements.*

*Item a) above is included in the list of persistent design situations since methods to define wind actions in BS EN 1991-4, and wave and current actions in Section **4**, are based on the analysis of extreme situations, and the partial factors chosen are appropriate to that situation.*

*NOTE 2 The appropriate return period for the environmental action can be considerably greater than the design working life (see Clause **15**).*

6.1.2 Transient design situations

Transient design situations should be as defined in BS EN 1990:2002+A1.

NOTE Examples of transient design situations are:

a) an increase in the hydrostatic head due to drawdown in an impounded basin occurring as a result of occasional inspections;

b) temporary actions during construction.

*Previous editions of BS 6349-1 listed many other types of action, but these are reclassified as accidental or seismic design situations (**6.1.3**, Note 1 gives some examples).*

6.1.3 Accidental design situations

Accidental design situations should be as defined in BS EN 1990:2002+A1.

Accidental design situations for marine facilities should include the accidental operating condition as defined in BS 6349-1-1:2013. Credible accidental design situations and consistent environmental conditions should be established by risk assessment as described in BS 6349-1-1:2013, Clause **22**.

Design situations involving accidental actions should be assessed in accordance with BS EN 1990:2002+A1, also taking into account the requirements specified in BS EN 1991-1-7.

NOTE 1 Examples of accidental design situations are:

a) an increase in the hydrostatic head due to drawdown in an impounded basin occurring as a result of accidental damage to impounding gates;

b) actions arising from uncontrolled ship berthing approaches;

c) combinations of actions arising from both the extreme wind including gust enhancements and the extreme maximum wave height.

NOTE 2 For some structures it is necessary to take into account the effect of very extreme environmental or operating loads to achieve a level of performance to avoid progressive or disproportionate failure. In such situations it might be necessary to treat environmental actions from events of return period 500 to 1 000 years as accidental design situations. A credible ship impact scenario with a structure supporting safety or production critical facilities might also be treated as an accidental design situation.

6.1.4 Design situations involving water pressures

The variation in water pressures and the water levels associated with them arising from tidal and meteorological conditions should be assessed. When using BS EN 1997-1:2004+A1 for the design of embedded retaining walls, gravity retaining walls or revetment slopes associated with quay walls or jetties, the permanent and variable components of water pressures should be identified so that appropriate partial factors can be applied. In addition, a decision should be taken as to whether it is appropriate to regard the components as coming from a single source [BS EN 1997-1:2004+A1, Note to **2.4.29**(P)] and an appropriate factor applied.

NOTE In general, the tidal lag component represents the variable part of the load, the remainder being the permanent part.

BS EN 1997-1:2004+A1, **2.4.6.1**(8) gives the option of applying a safety margin to the characteristic water level, which should take account of the anticipated tidal lag and effectiveness of any drainage system over the design life of the structure.

6.1.5 Design situations during execution

During execution (e.g. construction), the relevant design situations should be taken into account.

Where a maritime structure is brought into use in stages, the relevant design situations should be taken into account.

Where relevant, particular construction loads should be taken to act simultaneously with the appropriate operational and environmental loads.

NOTE 1 Where construction loads cannot occur simultaneously due to the implementation of control measures, they need not be taken into account in the relevant combination of actions.

NOTE 2 Snow loads and wind actions with long return periods need not be considered simultaneously with loads arising from construction activity Q_{ca} (i.e. loads due to working personnel).

NOTE 3 For an individual project it might be necessary to agree the requirements for snow loads and wind actions to be taken into account simultaneously with the other construction loads Q_{cb}, Q_{cc}, Q_{cd}, Q_{ce} and Q_{cf} as defined in BS EN 1991-1-6 (e.g. actions due to heavy equipment or cranes) during some transient situations.

Where relevant, the various parameters governing water and wave actions and components of thermal actions should be taken into account when identifying appropriate combinations with construction loads.

6.1.6 Design situations involving pre-stressing

The inclusion of prestressing actions in combination with other actions should be in accordance with BS EN 1992 to BS EN 1999.

6.1.7 Design situations involving differential settlements

Effects of uneven settlements should be taken into account if they are considered significant to the effects from direct actions.

NOTE 1 The individual project may specify limits on total settlement and differential settlement.

Uneven settlements on the structure due to soil subsidence should be classified as a permanent action, G_{set}, and included in combinations of actions for ultimate and serviceability limit state verifications of the structure. G_{set} should be represented by a set of values corresponding to differences (compared to a reference level) of settlements between individual foundations or parts of foundations, $d_{set,i}$ (where i is the number of the individual foundation or part of foundation).

NOTE 2 Settlements are mainly caused by permanent loads and backfill. Variable actions might have to be taken into account for some individual projects.

NOTE 3 Settlements vary monotonically (in the same direction) with time and need to be taken into account from the time they give rise to effects in the structure (i.e. after the structure, or part of it, becomes statically indeterminate). In addition, in the case of a concrete structure or a structure with concrete elements, there might be an interaction between the development of settlement and creep of concrete members.

The differences of settlements of individual foundations or parts of foundations, $d_{set,i}$, should be taken into account as best estimate predicted values in accordance with BS EN 1997 with due regard for the construction process of the structure.

NOTE 4 Methods for the assessment of settlements are given in BS EN 1997.

In the absence of control measures, the permanent action representing settlements should be determined as follows.

- $d_{set,i}$ should be assigned to all individual foundations or parts of foundations.
- Individual foundations or parts of foundation, selected in order to obtain the most unfavourable effect, should be subject to a settlement $d_{set,i} \pm \Delta_{dset,i}$, where $\Delta_{dset,i}$ takes account of uncertainties attached to the assessment of settlements.

6.1.8 Design situations involving snow loads

For an individual project, the potential for snow loads to be combined with other types of actions should be taken into account.

NOTE Snow loads need not be combined with port traffic unless agreed otherwise in particular geographical areas.

Snow loads should be combined with stored cargo loads unless agreed otherwise for the individual project.

6.1.9 Design situations involving wind and wave loads

Wind and wave loads acting in combination with operating loads should be consistent with environmental operating limits defined at the design stage for the operational situation under consideration.

NOTE 1 Operating wind and wave conditions are the maximum condition in which a particular operation is carried out (e.g. vessel berthing or crane operations). Operating values are defined for the individual project. Different operating values may be defined for different operations on a project.

NOTE 2 Depending upon the local climatic conditions, a different rule for wind and thermal actions may be defined for the individual project. The relevant National Annex might give guidance.

Wind, wave and current loading should be assumed to act simultaneously where the circumstances of the site allow them to act in the same direction.

*NOTE 3 In most cases, wind and locally generated wind waves are likely to act in the same direction. Wind and swell waves might not act in the same direction. See also the Notes to **6.1.2** and **6.1.3**.*

6.1.10 Design situations involving accidental (non-seismic) actions

NOTE 1 Where an action for an accidental design situation needs to be taken into account, no other accidental action need be taken into account in the same combination.

For an accidental design situation concerning impact from port traffic, mobile equipment or cranes, the loads due to the traffic, cranes, stored cargo and current acting on the structure should be taken into account where appropriate in the combinations as accompanying actions combined in accordance with Table 2.

As well as any variable loads as indicated in the combinations in Table 2, accidental berthing and mooring loads should be combined with the permanent component of loads from any port equipment or cranes on the structure. This is to allow for the situation where a crane might be located in a permanent parking place on the structure at the time that the accident takes place, and should therefore be regarded as part of the permanent loading, not the variable loading.

NOTE 2 Additional combinations of actions for other accidental design situations may be agreed for the individual project.

6.1.11 Design situations for geotechnical verification

Load combinations in design situations for geotechnical verifications should be the same as other ultimate limit state (ULS) verifications.

6.2 Combinations of actions

Actions should be combined using the formulae given in BS EN 1990:2002+A1 and in Table 2 of the present part of BS 6349 to verify ultimate and serviceability limit states for a full range of design situation appropriate to the works.

NOTE 1 Effects of actions that cannot occur simultaneously due to physical or functional reasons need not be considered together in combinations of actions.

NOTE 2 Combinations involving actions that are outside of the scope of BS EN 1991 and the present part of BS 6349 may be determined for the individual project taking account of the probability of simultaneous occurrence of different load components following the principles given in BS EN 1990:2002+A1, Annex C.

NOTE 3 For seismic actions, see Clause ***27****.*

The combination of actions given in the range of equations 6.9a to 6.12b in BS EN 1990:2002+A1 should be used when verifying ultimate limit states, with the exception of equations 6.10a and 6.10b, which should be excluded.

NOTE 4 Table 2 lists the combination of expressions in a systematic way.

NOTE 5 For fatigue verifications, see BS EN 1991 to BS EN 1999.

The combination of actions given in the range of equations 6.14a to 6.16b in BS EN 1990:2002+A1 should be used when verifying serviceability limit states.

NOTE 6 Additional recommendations are given in the relevant parts of BS 6349 for verifications regarding deformations and vibrations.

6.3 Combination factors

*NOTE 1 The combination factors, ψ, are used in the combination formulae. See **6.2** and Table 1.*

The ψ factors shown in Table 3 should be used for combinations of variable actions (see Table 2 for formulae).

NOTE 2 The ψ factors may be set for individual projects to reflect specific requirements provided that the principles of BS EN 1990:2002+A1, Annex C are adhered to.

6.4 Partial factors for actions

The partial factors listed in Table 1 should be used in the combination formulae set out in **6.2** and Table 2.

In applying Table 1 in cases where the limit state is very sensitive to variations in the magnitude of permanent actions, the upper and lower characteristic values of these actions should be taken and the most adverse case verified.

Static equilibrium (EQU) for marine structures should be verified using the design values of actions in set A.

Design of structural members (STR) where no geotechnical actions are involved should be verified using only the design values of actions in set B.

Design of structural members (STR) involving geotechnical actions and the resistance of ground (GEO) should be verified using one of the design approaches set out in BS EN 1997 and which, depending on the design approach used, utilizes design values of actions from either set B or set C or both.

Geotechnical design in general should be carried out using BS EN 1997 with set B or set C partial factors selected from Table 1 of the present part of BS 6349.

NOTE 1 BS EN 1997 has rules for the assessment of geotechnical stability and also the assessment of hydraulic and buoyant failure situations.

NOTE 2 The choice of design approach 1, 2 or 3 is given in the National Annex to BS EN 1990:2002+A1 where appropriate.

Load combinations for geotechnical verifications should be the same as other ULS verifications.

The γ_p values to be used for prestressing actions should be specified for the representative values of these actions in accordance with BS EN 1990:2002+A1 to BS EN 1999.

*NOTE 3 More detailed information on prestress is given in BS EN 1990:2002+A1, Annex A2, **A2.3.1**(8).*

Table 3 ψ **factors for load combinations in maritime structures** *(1 of 2)*

Action		ψ_0 **Factor for the combination value of a variable action** [A)]	ψ_1 **Factor for the frequent value of a variable action** [A)]	ψ_2 **Factor for the quasi-permanent value of a variable action** [A)]
Vehicular traffic loads	Road vehicles (gr1a) [B)]	0.75	0.75	0
	Road vehicles (gr1b)	0	0.75	0
	Pedestrian loads (gr1a) [B)]	0.40	0.40	0
	Horizontal forces (gr2) [B)]	0	0	0
	Pedestrian loads (gr3) [B)]	0	0.40	0
Pedestrian-only traffic loads [C)]	gr1	0.40	0.40	0
	Q_{fwk}	0	0	0
	gr2	0	0	0
Equipment loads	Gantry crane	0.75	0.75	0 [D)]
	Mobile harbour crane	0.75	0.75	0 [E)]
	Construction crane	0 [F)]	0	0
	Port vehicles	0.75	0.75	0
	Cargo handling equipment	0.75	0.75	0
Cargo loads [G)]	Containers in main storage stacks	0.90	0.80	0.70
	General cargo or containers on quay aprons [H)]	0.70	0.50	0.30
	Bulk cargo	1.00	0.90	0.80
	Liquid products	1.00	0.90	0.80
Environmental loads	Wind	0.50	0.20	0
	Operational wind	1.00	0	0
	Thermal actions	0.60	0.60	0.50
	Snow	0.80	0	0
	Ice	0.80	0.80	0
	Water currents	0.60	0.20	0
	Operational water currents [I)]	0.60	0.20	0
	Wave	0.60	0.20	0
	Operational wave	0.60	0.20	0
	Tidal lag [J)]	0.60	0.20	0
Operational loads	Berthing	0.75	0.75	0
	Mooring	0.50	0.20	0 [I)]
	Ship ramps	0.70	0.50	0
	Ships propulsion	0.75	0.75	0
Construction and installation loads		1.00	0	1.00

Table 3 **ψ factors for load combinations in maritime structures** *(2 of 2)*

Action	ψ_0 **Factor for the combination value of a variable action** A)	ψ_1 **Factor for the frequent value of a variable action** A)	ψ_2 **Factor for the quasi-permanent value of a variable action** A)

A) Definitions of ψ factors are given in BS EN 1990:2002+A1, **4.1.3**.

B) For road traffic and pedestrian loads, the factors for bridges should be adopted and are reproduced here. The abbreviations gr1a, gr1b, gr2 and gr3 refer to load models defined in BS EN 1991-2:2003, Table 4.4a

C) Pedestrian-only traffic loads are applicable to structures that provide access for foot traffic and light service vehicles only.

D) For seismic conditions with respect to gantry cranes, where no information on the seismic performance of the crane is available, a gantry crane should be placed in a location judged as likely to generate the critical actions and it should then normally be regarded as a permanent action, i.e. it is not regarded as a variable load. If there is information on the seismic performance of the crane this can be taken into account in allocating the crane actions to the quay wall or jetty. ASCE 61-14 [N1] provides specific guidance on this issue.

E) The operation of the mobile harbour crane should be taken into account. Its permanent load might need to be added to the permanent load of the overall structure in some locations as for a gantry crane.

F) A crane used for construction or maintenance might need to be combined with other loads if operations are permitted in the vicinity.

G) The factors for the cargo loads are based on the definition of frequent and quasi-permanent in BS EN 1990:2002+A1. In the case where unusually persistent loadings are expected, the provisions of BS EN 1990:2002+A1, Annex B and Annex C should be used to derive revised partial and combination factors.

H) If the quay apron is used for main container storage stacking, the factors appropriate to "Containers in main storage stacks" should be used.

I) Where currents contribute significantly to the mooring loads, the mooring load due to the quasi-permanent current should be used.

J) Tidal lag for the combination and frequent load should be that for a mean spring tide. For the quasi-permanent load, the tidal lag due to a mean tide should be used.

7 Serviceability and other specific limit states

For serviceability limit states, load combinations should be selected using the combination formulae in Table 2 and design values of actions selected accordingly, unless differently specified in BS EN 1991 to BS EN 1999.

The serviceability criteria should be defined in relation to the serviceability requirements. Deformations should be calculated in accordance with BS EN 1992 to BS EN 1999 by using appropriate combinations of actions according to BS EN 1990:2002+A1, expressions 6.14a to 6.16b, taking into account the serviceability requirements and the distinction between reversible and irreversible limit states.

NOTE It is not generally necessary to assess structures for deformation limits except to control drainage and other services and deformation of sensitive equipment such as gantry or ship-to-shore (STS) cranes.

Walkways and similar structures which might be sensitive to vibration should be verified using the rules for footbridges in accordance with BS EN 1990:2002+A1, **A2.4.3**.

Section 3: Wave and water-level conditions

*COMMENTARY ON SECTION **3***

It is necessary to obtain and verify information on relevant wave and water-level conditions for the planning, design, construction and operation of maritime works.

The information required can range from the persistence of relatively calm intervals for the operation of construction plant or safe vessel operations, to seasonal variations in the general characteristics for operational purposes, to predictions of normal and extreme environmental load events which structures need to be able to withstand during their design life. In nearly all cases, the availability of long-term wave information (over several decades) is required.

*Wave data (measured or estimated), as discussed in Clause **11**, is generally more readily available at offshore (deep-water) locations than at nearshore sites. For maritime works in the coastal zone, it is therefore almost always necessary to transform offshore wave conditions to nearshore locations, taking account of transitional and shallow water effects, as discussed in Clause **12**. It is desirable and in some cases essential to validate the transformation with short-term measured data and/or site observations at the site of interest. Water-level data tends to be more readily available at inshore locations and thus the coincident timing of wave conditions and water-levels at the location of interest needs to be borne in mind (relative to the original measured location and any transformation process).*

This section gives guidance on derivation of wave and water-level conditions for the planning, design, construction and operation of maritime works.

8 General recommendations on deriving wave and water-level conditions

When deriving characteristic wave and water-level conditions for planning or design of maritime works, the designer should be aware of the use to which they are to be put. Examples of issues that should be addressed include the following.

a) Where wave actions (e.g. impact, slam or uplift) or hydraulic responses (e.g. run-up, overtopping or transmission) are of interest for design of structures such as breakwaters, seawalls, quays or jetties/piers, extreme (low probability) wave and water-level conditions at or near the structure toe should be derived.

b) Where actions on piles (singular or arrays) or other relatively slender structural elements are of interest, wave spectra or statistics (to take account of the range of wave frequency and hence fatigue loading) should also be taken into account.

c) In situations where large vessels are to be moored inshore or within harbours that are in relatively exposed locations, the potential for long waves to be experienced at the site should be assessed.

NOTE 1 This is because long period wave motions associated with wave grouping, e.g. where a series of high waves follows a series of low waves, are frequently responsible for the largest mooring loads.

d) Where the wave and water-level conditions are to be used for the design of a beach, or beach-control structure, the designer should derive long-term wave statistics or time-series records (seasonal, annual and decadal coverage) as well as short-term storm event information.

e) In all cases, the designer should also take into account the frequency (and directional) distribution of the wave energy spectrum, as longer period (swell) waves can often contain greater energy (and/or arrive from a different direction) than shorter period higher waves within the wave spectrum.

NOTE 2 Bi-modal or bi-directional sea-states can have more onerous consequences for some design situations, e.g. vessel motions and mooring loads, wave run-up/overtopping/transmission or beach (or dynamic structure) response.

f) In many cases, incident wave conditions are influenced by wave-structure interactions (e.g. reflection or diffraction) and the designer should take account of the extent to which the method of deriving the wave conditions and/or the design methodology considers or represents such effects.

g) For certain design applications which rely on empirical relationships (typically seawalls or breakwaters), estimates should be made of irregular wave conditions based on statistical or spectral parameters such as height, period and direction, but an appreciation of the wave form (e.g. non-breaking, plunging, surging) is important to allow greater insight as to the likely structural performance or response.

NOTE 3 Some structure types are suited to adoption of a response-based approach where the response of the structure (or element) is assessed against the full range of environmental actions expected (either statistically or in time-series form). In this way, the probability of a certain response can be assessed without the need for simplified treatment (and description in probabilistic terms) of the input parameters which define the actions.

9 Climate change

9.1 General

During planning or design of maritime works, the designer should take account of the fact that climate change is widely predicted to cause sea-level rise and changes in storm intensity and direction (potentially affecting coastal surge/wave and pluvial/fluvial events). The designer should take into account region-specific guidance where available, as well as the probability level associated with published or predicted changes, e.g. due to inherent uncertainties in future emissions scenarios and climate modelling generally. These factors should be assessed in relation to the known purpose and design working life of the structure, its vulnerability and the consequences of damage, as well as the potential for future adaptation of the structure (to accommodate future changes to predicted or published values).

Where appropriate (see Note 1), a sensitivity/vulnerability assessment should be conducted to take account of inherent uncertainties in climate change predictions, allowing for confidence intervals assigned to estimates, lower/higher bound probability estimates or alternative emissions scenarios.

NOTE 1 A valid alternative approach, where sufficient understanding of inherent uncertainties is not possible, is to adopt a more severe design event (with lower probability of occurrence).

The designer should take into account the evolving nature of published estimates due to the complexities of climate change science.

NOTE 2 For example, the Intergovernmental Panel on Climate Change (IPCC) provides an updated assessment every five to seven years, on which many regional authorities base local guidance. Many researchers also propose alternative estimates to those produced by the IPCC and in particular there has been considerable speculation over the likely timing of the melting of the Greenland Ice Sheet and the West Antarctic Ice Sheet, which are significant potential contributors to sea-level rise.

NOTE 3 In the UK, a commonly adopted source of projections is UK Climate Protections (http://ukclimateprojections.defra.gov.uk/ [1]) (UKCP) and this may be used to obtain information on relative sea level rise, changes in extreme water levels and climate-driven changes in offshore wave conditions at different points around the UK. UKCP may also be used to obtain predictions of changes in other climate-related parameters, where these might be required for a particular project, e.g. temperature, rainfall and salinity.

NOTE 4 It is considered good practice to put in place measures that are robust across a range of probability levels. That range depends on the purpose of the structure, the potential risks/consequences, and the costs and benefits of allowing for different levels of uncertainty.

9.2 Sea-level rise

Changes in mean sea-level relative to land levels over the design working life, and through decommissioning where required/appropriate, should be taken into account by the designer.

NOTE Sea-level rise is principally caused by thermal expansion of the water and melt of mountain glaciers/land-based ice increasing its volume (eustatic rise), and tectonic and post-glacial movements of land (isostatic rise). Predictions for relative mean sea-level rise in a particular region therefore vary from the global average due to geographical variations in temperature and salinity and geological processes affecting vertical land movement.

The designer should take the contributing factors into account when reviewing and selecting appropriate values to be adopted in the assessment of actions and environmental conditions affecting the structure.

Where appropriate, the designer should make an allowance for associated increases to wave conditions whereby increased water-depths allow generation of larger waves or reduce wave attenuation effects due to nearshore wave processes or depth-limited breaking.

9.3 Increased storminess

The potential for changes in storm intensity and frequency relevant to the site of interest should be taken into account by the designer when reviewing and selecting values to be adopted for the assessment of actions and environmental conditions affecting the structure. Where national guidelines exist, these should be followed; otherwise the approach used should be explained to the client or end user as appropriate.

NOTE 1 Changes to storm intensity and frequency (associated with climate change) can lead to increased magnitude storm surges (positive or negative) when compared to estimates based on historical events. Further changes can be associated with higher energy wave conditions (swell and/or locally generated waves) and potentially differing directions of wave travel when compared to historical records, measurements or statistics (or estimates derived from such).

NOTE 2 Guidance relevant to UK conditions is given in the Environment Agency publication Adapting to climate change: Advice for flood and coastal erosion risk management authorities *[1].*

9.4 Other climate change issues

Where relevant to the works being planned or designed, the designer should take into account potential climate change effects on parameters other than water-level and wave conditions. Where national guidelines exist, these should be followed; otherwise the approach used should be explained to the client or end user as appropriate.

1) Last accessed 17 June 2016.

NOTE 1 Such parameters can include. but are not limited to. salinity, water temperature, water acidity, oxygen depletion, thermal water circulation/currents, evaporation, precipitation, freshwater flows, air temperature, wind speeds and directions.

NOTE 2 Guidance relevant to UK conditions is given in the Environment Agency publication Adapting to climate change: Advice for flood and coastal erosion risk management authorities *[1].*

10 Wave characteristics

10.1 General

The assessment of wave conditions should be performed with an appreciation that a real sea is often a complex and irregular surface comprising many waves of different heights, periods and directions superimposed on one another. The designer should choose a method of simplifying or representing the real wave conditions which is appropriate to the situation being assessed and the stage of design. The method used should take into account whether the waves are in deep, nearshore (transitional) or shallow water and the likely spectral shape and wave form of the sea state.

*NOTE 1 In most design situations, waves are described using a single representative wave height, period and direction (in a particular water depth). Representative (statistical or spectral) wave parameters for a particular event (or group of events) can be estimated using a variety of techniques, as described in Clause **11** and Clause **12**, or alternatively can be derived directly from measured wave data, any of which yield parameters such as significant wave height H_s (=$H_{1/3}$), mean period T_m, from a statistical approach and spectral significant wave height H_{m0}, peak wave period T_p from a spectral approach. CIRIA publication C683 [2] provides an introduction to the concepts of statistically described wave conditions (**4.2.4.4**) and spectral descriptions (**4.2.4.5**). Design approaches based on these parameters are commonly used in design. It is sometimes necessary to convert one set of parameters to another. Guidance on this can be found in CIRIA publication C683 [2], **4.2.4.5**.*

Situations should be assessed on a case-by-case basis.

NOTE 2 For example, design approaches for some structure types require the maximum wave height as an input parameter, and generally rely on a simplified ratio between significant and maximum wave height based around an assumed statistical distribution.

Diffraction and refraction are sensitive to the characteristics of the whole wave spectrum and where possible should be estimated taking into account the component contributions.

NOTE 3 Circumstances where the cumulative effect of waves is important can be dealt with by means of a probability distribution of wave parameters. Other design situations which can be sensitive to wave groups can benefit from physical modelling using irregular waves.

10.2 Spectral description

For an accurate analysis of wave conditions, where possible the designer should use a spectral approach. If required, regular wave parameters for design can be derived from spectra; however, for more accurate analysis, the component waves of the spectra should be used in computations and/or model testing.

NOTE 1 Energy in the sea is carried by a large number of individual waves of different heights, periods (or frequencies) and directions, which when superimposed create an irregular surface profile. Conversely, the irregular profile of the real sea can be broken down into a number of component waves. The distribution of energy of these component waves (which is proportional to the square of their wave height), plotted against frequency and direction, is called the "directional wave spectrum". When plotted against the frequency alone it is called the "frequency" or "one-dimensional" wave spectrum.

NOTE 2 A wave spectrum at a particular site can be obtained either through measurement and analysis of sea surface profiles or by adopting an appropriate standard one-dimensional (non-directional) spectrum (although actual spectra deviate from these standard forms).

NOTE 3 Semi-empirical equations exist to represent the standard one-dimensional (non-directional) energy density wave spectra of various sea states. The two most widely used are the Pierson–Moskowitz (P–M) spectrum, which can be used to represent a fully developed sea in deep water, and the JONSWAP (JOint North Sea WAve Project) spectrum, which can be used to represent fetch-limited sea-states in deep water (i.e. growing seas). Details of these can be found in CIRIA publication C683 [2]. Alternative spectra exist for shallow water areas where wave decay occurs through depth limited breaking (see Random seas and design of maritime structures *[3]).*

NOTE 4 Swell wave energy tends to be distinguishable within a wave spectrum due to a sharper peak than locally generated wind-wave energy, owing to velocity dispersion over a long distance once the waves have left the generating area (the low frequency wave components propagate faster than the high frequency wave components). For engineering applications, the swell component of the spectrum may be approximated by the JONSWAP spectra with the peak enhancement factor chosen depending on the distance travelled (see Random seas and design of maritime structures *[3]). When swell co-exists with locally generated wind seas, two (or possibly more) peaks occur at the frequencies corresponding to the representative periods of swell and (locally generated) wind waves, and is often referred to as a "bi-modal" sea. The spectrum of the resultant sea state can be estimated by linearly superimposing the spectra of wind waves and swell. In this case, or where the spectra is very flat as a result of heavy wave breaking, it is not easy to establish T_p, and the mean energy period, T_E (= T_{-10}, the averaged period weighted by the energy spectrum) may be used for the design of structures.*

Swell energy in bi-modal sea-states can often have a direction that differs significantly from the wind-sea. The influence of this bi-directionality on the design situation should be taken into account, e.g. in the optimization of berth orientation and mooring design.

10.3 Non-linear wave theories

*COMMENTARY ON **10.3***

Waves, especially those in deep water, are commonly idealized as linear (sinusoidal) waves for a first approximation to the wave motion, and linear theory adequately describes many phenomena associated with the wave motion, such as dispersion, refraction and diffraction. Further guidance on linear wave theories can be found in Annex A.

A summary of wave characteristics (velocities, accelerations, pressures, etc.) defined by linear wave theory for early stage engineering approximations in deep, nearshore and shallow water is given in Annex B. However, even in deep water, linear wave theory only provides an approximation. Non-linear wave theories better describe wave breaking, shoaling, reflection, transmission and mass transport.

*A useful introduction to non-linear wave theories can be found in CIRIA publication C683 [2], **4.2.4.3**. More detailed descriptions can be found in* Nonlinear wave theories *[4]. References to other wave theories can be found in USACE EM 1110-2-1100 (Part II) [5].*

Non-linear analytical wave theories which may be used include Stokes fifth order theory for waves which are not very long relative to the water depth (deep water, short waves), and Cnoidal fifth order theory (which may be enhanced by Aitken transformations) where the waves are long relative to the water depth (shallow water, long waves). Stokes' fifth order theory is not to be used in situations where the Ursell number is greater than 10. Cnoidal fifth order (plus Aitken) may be used when the Ursell number is greater than 10.

However, the numerical Fourier solution method (based on a Fourier analysis of the stream function flow) is superior to the above analytical theories as it is accurate to within 1% of the highest waves and is valid for all finite wavelengths, whatever the water depth. This approach is suitable for practical applications.

If it is necessary to solve a wave of infinite wavelength (for example long waves resulting from tsunamis or waves resulting from large displacements of water such as landslides), solitary wave theory can be used, which entails using cnoidal theory in that limit. Further information can be found in USACE EM 1110-2-1100 Part II) [5].

Linear wave theory may be used for computational reasons such as in spectral fatigue calculations and for large volume body calculations (i.e. diffraction regime).

Where a spectral approach is not considered practical, the designer should select a wave theory which is suitable for the potential wave-forms to be encountered and the response of the structure or element being assessed.

The most simple representation of wave-form is that of a linear (sinusoidal) wave. As wave steepness (H/L) or relative wave height (H/d) increases, however, the wave-form deviates from linear/sinusoidal representations, as is the case, for example, in the nearshore zone or when waves meet an opposing current. In such cases, linear wave theory becomes less valid and a non-linear wave theory should be used to better represent the sharper crests and longer troughs of real waves. Non-linear wave theories, ideally Fourier theory, should therefore be used where applicable and the designer should take into account the fact that the velocity of any currents might need to be combined with the water particle velocity caused by waves in order to ascertain loading/actions on structures (typically slender structures or elements).

10.4 Wave-forms and motions

Where relevant, the designer should take account of water particle orbital velocities and accelerations due to waves (and currents) to assess actions on structures, particularly on submerged structural elements.

The designer should take into account the fact that although particle velocities at the surface are greatest in short-crested/steep waves (relatively high/short period), a longer period wave can result in greater velocities at depth because the velocities decay less rapidly.

Application of wave data to the design of maritime structures should take appropriate account of the form of motion.

NOTE 1 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. Non-linear waves have a different profile to linear waves; the wave crests become more peaked and are higher above the still water level and the troughs become flatter.

NOTE 2 There is a difference between "phase velocity" (the speed at which a wave propagates), "orbital velocity" (the water particle velocity at various locations as a wave passes, which is required for assessing loading on structures) and the "group velocity", which is the energy a train of waves travels and is less than the phase velocity of an individual wave.

10.5 Nearshore wave processes

When a wave group approaches a land-mass, reef, shoal or island (or any area where the water depth reduces such that the waves can no longer be described by offshore or deep-water descriptions), the designer should assess how the wave characteristics are altered by the influence of the seabed, leading to changes in the velocity, length, height and direction of the waves. Nearshore effects should generally be taken into account when the depth reduces to less than one half the deep-water wave-length. The most common effects of refraction, shoaling and wave breaking should be taken into account, with other effects such as bottom friction, diffraction, reflection, wind-growth (or decay), wave-wave and wave-current interaction included where appropriate.

NOTE 1 Under linear wave theory, the period of the waves can be assumed to be constant as waves propagate from deep-water through the nearshore zone to shallow water.

NOTE 2 Further information on nearshore wave processes and assessment methods suitable for concept design is given in Annex B.

The designer should assess variations in water level (due to tide, surge or wind/wave set-up/set-down) and bathymetry (due to accretion or scour) in order to understand the range of possible water depths applicable in the vicinity of the structure and the potential for depth limited wave breaking to be a controlling effect on design wave conditions.

*NOTE 3 For some nearshore structures, the design wave height can be limited by assessing wave breaking, as described in **10.6** and Annex B.*

10.6 Wave breaking

For some structures located in nearshore or shallow water areas, the design wave conditions and/or structure performance and response are governed or heavily influenced by wave breaking. The designer should therefore reach an understanding regarding the nature and form of wave breaking as well as the resulting design wave parameters, and the extent to which these parameters accurately represent the wave conditions.

In shallow water the designer should assess the breaker type using the surf similarity parameter (which allows a description of the wave characteristic) as this can be important in terms of wave-structure interaction.

In cases where wave breaking is a dominant action on the structure, numerical or physical models should be employed to fully understand the potential effects.

NOTE Further information on wave breaking and assessment methods suitable for concept design are given in Annex B.

10.7 Wave-current interaction

The effect of tidal, longshore and fluvial or estuarine currents on wave propagation and wave parameters should be assessed. In particular, waves and currents should be taken into account when assessing flow velocities near the bed, e.g. for scour protection, or on relatively slender structures such as singular piles, pile arrays or submerged pipes.

NOTE 1 This might require assessment of joint probabilities to obtain appropriate design conditions.

NOTE 2 The interaction of waves with currents can increase or decrease wave height and steepness and also cause refraction. Currents near the mouths of estuaries can increase wave heights significantly.

11 Offshore wave climate

11.1 General

If wave data (of sufficient quality and length of record) is not available at the site location (which generally it is not), offshore wave data sources should be researched and appropriate data obtained and reviewed, prior to transforming the data to the nearshore (or inshore) site of interest as described in Clause **12**. The relative water-depths of the offshore data location and site of interest should be assessed in relation to the range of wave conditions contained in the record and therefore the potential for any nearshore wave processes to influence the wave data.

Account should be taken of the expected wave spectrum at the site of interest in terms of relative distribution of locally generated waves and swell waves. In relatively enclosed water bodies it might be sufficient to apply wind-wave hindcasting techniques (empirical or numerical). At sites with potential exposure to large sea or ocean generated waves, care should be taken to ensure that the effect of swell waves is accounted for (potentially alongside locally generated waves). The designer should therefore determine the suitability of the wave data sources available in this respect.

Published values are sometimes available, depending on the region of interest (e.g. in BS EN ISO 19901-1). The designer should assess the reliability of any such values for use in design, and seek alternatives where appropriate.

In the absence of reliable published values being available, offshore wave data should be obtained from one or more of the following sources, each of which has different characteristics:

- synthetic wave data (based on meteorological data and wave estimation/prediction techniques);

  NOTE 1 Synthetic offshore waves can be estimated in a relatively straightforward manner using recorded wind data together with empirical hindcast techniques taking account of the wind speed, direction, duration and generation area (fetch length). A more sophisticated approach involves application of open source or commercially developed large scale numerical models. Such models typically provide continental or global scale coverage and can be applied in hindcast mode to estimate wave time-series over preceding years or decades, or also applied in forecast mode to provide predictions over several days or weeks ahead. A typical example of an open source model of this type is WaveWatch III, although other similar models are available.

- measured wave data (either in-situ from direct recorders such as wave buoys or pressure transducers, or remote sensing, e.g. photography, radar, laser and acoustic altimetry);

  NOTE 2 Wave measurements are often the most accurate source but are specific to the location and period of operation of the measurement device (often a relatively short period, typically less than ten to twenty years at the time of publication of this part of BS 6349).

- visually observed wave data [typically from a voluntary observing ships (VOS) system].

  NOTE 3 Visual wave measurements to some degree suffer from the errors and subjectivity of the observers. Wave climate information based on a VOS system typically covers a wide area and relatively long period of time, but may be based on sporadic frequency or location of observations.

NOTE 4 Synthetic numerical model data is most commonly adopted and is widely available in the form of continental or global scale models, often based on satellite-measured wind hindcasts, e.g. UK Met Office European and Global wave models or similar products from other commercial sources. The physics represented by such numerical models need to be sufficiently understood to confirm the model's ability to simulate the anticipated processes in the region of interest.

The limitations of each source and method of collecting, analysing and processing wave data should be understood, including measurement accuracy of a particular device in a particular circumstance, duration of measurements and simplifications made by analysis software.

It is common that measured nearshore data of relatively short duration is used to calibrate numerical models applied for offshore to nearshore wave transformations. In cases where such models are applied to transform extreme storm events for design purposes, the calibration should ideally be based on measured data including at least one storm event of some significance, although this is not always achievable.

NOTE 5 In certain situations, such as early stage assessment of sites, offshore wave data can be obtained from the use of meteorological data in conjunction with wave prediction charts. Further information on this is given in Annex C.

11.2 Synthetic wave data

11.2.1 Satellite-based offshore wave models

NOTE 1 Several proprietary and open-source numerical models exist which simulate deep-water wave growth and propagation at relatively large scales (regional seas, oceans or global coverage). These models rely on satellite-measured wind data and have been calibrated to varying degrees against available wind and wave measurements.

Before use, the relative advantages and limitations of each available model should be assessed in order to select the most suitable, in terms of representation of key physical processes.

NOTE 2 For example, certain models do not resolve the wind-field associated with cyclone events to a sufficient scale to allow reasonable estimation of the associated wave climate, which could be of critical importance for projects in tropical latitudes.

Synthetic wave data sources should be reviewed by the designer and assessed for suitability in terms of the wave parameters necessary for the intended purpose. In particular the designer should determine the extent to which the wave data sources provide a realistic representation of swell waves.

In applying this data to a specific site, the designer should determine the need for wave transformation modelling (to transform the source data to the site of interest to account for nearshore wave processes) and the extent to which local wave records/measurements should be obtained or commissioned in order to calibrate or validate the model output.

11.2.2 Empirical wave hindcasts

Where the wave climate at a site of interest is expected to be dominated by locally generated waves (with an absence of swell wave energy) and more reliable wave data sources are not available, an empirical hindcast technique should be adopted by the designer to provide an approximate estimate of wave conditions.

NOTE 1 Such an approach is likely to be restricted to inland or enclosed lakes or seas. There might be other cases where such an approach is justified, e.g. where the design wave parameters or structure performance/response are known (with a high degree of confidence) not to be sensitive to the contribution of swell wave energy.

Before proceeding with this approach, the designer should confirm that a suitable length and quality of wind record is available for input/boundary condition purposes to provide a reliable basis on which to estimate wave conditions. The selected methodology should be appropriate to the physical processes within the region of interest.

NOTE 2 Further information regarding empirical wave hindcasts including suitable methodologies is given in Annex C.

NOTE 3 In many locations around the British Isles, the most severe wave conditions are usually associated with locally generated wind waves, and the wave estimation techniques described in Annex C 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 maritime works. 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 locally generated storm waves.

Wherever possible, an appreciation of the wave spectra should be gained including the relative contributions of swell and locally generated wave energy within the spectra.

11.3 Measured wave data

Where available and if the quality and reliability of the data can be assured, measured wave data is the preferred source of offshore wave data and the designer should obtain and make use of such data. In many situations, however, the period over which measured data is available is generally insufficient to allow an understanding of the long-term wave climate, particularly in relation to extrapolating extreme storm events for design purposes.

Where measured data is not available offshore, but exists at a nearshore or inshore location within the region of interest, this should be obtained and used to calibrate or validate wave transformations from offshore to nearshore.

Wave measurements from oceanographic instruments, e.g. waverider buoys or pressure transducers, are the most accurate means of recording the wave climate, but are generally limited to a few sites and of relatively short duration, often with record gaps or data anomalies due to instrument malfunction. Wave measurements based on remote sensing are comparatively less accurate and tend to cover very short (and intermittent) durations only at any given location. The designer should research the availability of existing measured wave data before commissioning new measurements.

NOTE 1 The British Oceanographic Data Centre maintains a global inventory of marine data including waves. Around the coast of the UK, in England and Wales the Environment Agency has established five regional coastal observatories that record and collate wave data for their region. This data can be accessed via the website of the Channel Coastal Observatory. National data centres for the dissemination of wave data exist in other European countries.

NOTE 2 If the time available for the installation and operation of a wave recorder, and the subsequent analysis of the data, is insufficient, then the limited wave records for which there is time may be supplemented with published synthetic data or visually observed data. This sets the short term records in a longer term context and thus allows the longer term published data to be calibrated for the specific site of interest.

11.4 Visually observed wave data

Visually observed data depicts combined (resultant) locally generated and swell waves. The existence of visually observed data is reliant on ships being in the area. In evaluating the reliability of this data the designer should take account of the number of observations on which the statistics are based. Where other wave data is available or can be derived using numerical models, visually observed data should be used only for comparison purposes or to complement shorter-term data-sets.

NOTE 1 Visually observed wave statistics, based on a number of years of visual wave observations made by VOS, giving wave heights, periods and directions, are usually available for each month of the year and for parts of most seas and oceans worldwide. 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. Various comparisons have been made which generally tend to show that VOS data can over-estimate wave-heights.

NOTE 2 Busy shipping lanes are particularly well served, while there might be a lack of data in more remote areas. Further, visual data might be absent for particular extreme events (such as hurricanes) when vessels tend to leave the affected area or take shelter.

11.5 Extrapolation of offshore wave data

NOTE 1 When a suitable length and quality of record of offshore wave data has been obtained from one or a number of the previous sources or methods, it is often necessary to extrapolate these data to obtain design event or extreme operating conditions appropriate to the design requirements. Extrapolation techniques, often referred to as extreme values analysis techniques, are described in Clause ***15****.*

In most cases an extreme values analysis should be conducted on offshore wave data records, the results of which (individual design events) should then be transformed to the site of interest, taking account of nearshore wave processes and wave-structure interactions. Where appropriate, however, firstly the full offshore wave record should be transformed to a nearshore location, and then an extreme values analysis should be performed.

NOTE 2 For example, this might be more appropriate in situations where nearshore wave processes significantly modify important parameters such as storm duration or direction, or where coincident timing of wave conditions and water-levels is important (for some dependent/joint probability analyses).

12 Wave transformation

12.1 General

Deep-water waves should be transformed to calculate how they change as they propagate from the areas in which they were generated to the locations of interest, taking account of additional wave growth within the domain of interest. Nearshore wave processes should be taken into account when the water depth reduces to less than one half the deep-water wave length.

Numerical models should be used to calculate transformations to wave spectra. However, the available numerical models are based on simplified governing equations and boundary conditions and employ various numerical schemes, imposing different restrictions on their practical application. Consequently, designers should always be aware of the advantages, disadvantages and limitations of the various theories and of the models used to describe wave transformation.

NOTE 1 Changes can be triggered by variations in wind speed and direction over the domain of interest, coupled with swell wave energy entering the domain and hence resultant wave spectra, the underlying bathymetry, the presence of currents, the existence of natural or man-made structures and other features. Shoaling, refraction, diffraction, wave-current interaction, reflection, wave breaking, wave set-up and a range of other processes could all occur.

NOTE 2 As ocean waves are themselves variable in time and space, prediction of their transformations by these combined processes is complex and subject to uncertainties. As wind-waves consist of components having a wide range of frequencies and directions, and those with long periods propagate faster than those with short periods, the original storm waves can become widely dispersed, with the longer waves being affected quite differently to the shorter waves.

NOTE 3 The appropriateness of any individual modelling procedure depends on the relative importance of the various physical processes involved and on the level of detail required at the specific site. The development and validation of numerical models is rapidly changing, and no central authority keeps track of those changes. For practical purposes, there are two overall classes of irregular wave transformation models in common use.

- *Phase-averaged models are for slowly varying wave conditions and are generally used for transforming waves from offshore to nearshore over large distances in large areas. They are based on calculating quantities such as spectral energy densities rather than the time-varying details of the water surface elevation. They are generally not suitable for modelling wave steepening and breaking in the nearshore area, interaction with structures or diffraction.*
- *Phase-resolving models are for rapidly varying wave conditions (i.e. for those with significant variations over distances of the order of one wavelength) such as diffraction around structures, regions with excessive wave breaking and non-linear wave-wave interactions. They are currently confined to applications involving relatively small sea areas (typically a few square kilometres) due to the computation time and computer memory requirements. They include models which employ the Boussinesq equations and where appropriate elliptic or parabolic solutions to the mild-slope equation.*

NOTE 4 The use of computational fluid dynamics (CFD) approaches is becoming increasingly common in modelling waves and wave-structure interactions, which allow a more accurate representation of the physical processes, but owing to intensive computing requirements can only currently be applied to very small areas.

No existing wave transformation model, phase-resolving or otherwise, involves a comprehensive description of all of the many processes involved. The simplifying assumptions that all numerical wave models are based on should be taken into account by the designer when interpreting model output. As a general rule, the designer should validate the transformation with short-term measured data and/or observations at the site of interest, as well as physical model tests or alternative numerical approaches as appropriate to the risks involved.

NOTE 5 An example of an occasion when this might not be necessary is an outline feasibility assessment for high-level planning purposes.

In the case of very shallow foreshores where wave breaking and re-forming occurs, narrow-peaked bi-modal wave spectra can develop, the characteristics of which are not represented by many approaches. If necessary, the designer should use a phase-resolving model based on Boussinesq equations or physical modelling to better understand the resulting wave actions.

NOTE 6 Generally, numerical models valid for realistic shorelines are appropriate. However, at initial stages in the development of a project, it might be appropriate to use simple empirical methods to represent the main physical processes. Further information on these methods is given in Annex C.

12.2 Channel effects

The designer should assess how a deep draught navigation channel (or sudden natural change in bathymetry) affects the wave field, as wave refraction, reflection/scattering and transmission caused by interactions of waves with the side slopes of the channel can have a significant impact on the wave climate.

Numerical models which make the assumption of slowly varying bathymetry do not represent a deep channel (or sudden natural step) well, and should not be relied upon in this respect to make an accurate assessment of the wave climate for design.

NOTE Phase-averaged models are likely to be inappropriate due to their inability to incorporate wave scattering. Boussinesq models can be used to assess the implications and optimise the design. The design can then be confirmed through three-dimensional physical modelling.

12.3 Geological and bathymetric features

The designer should collect information on geological/bathymetric features, both local to any development and in neighbouring areas, as wave conditions are influenced by the bathymetry of the sea-bed and by the configuration of the adjacent coast (e.g. in reflecting wave activity). Inshore wave measurements or observations should be taken to obtain reasonably accurate predictions where the bathymetry is complex.

NOTE For example, with sand banks or reefs the behaviour of waves is highly non-linear due to effects such as wave breaking.

The designer should take account of the fact that that erosion and accretion of the sea-bed and coastline can impose significant changes in bathymetric features, both seasonally and over the lifetime of any project.

The rate of change in the bathymetry depends on the balance between erosion and accretion, which themselves depend upon the waves, currents and sediment budget. The complicated interaction between waves, currents and bathymetry introduces considerable uncertainty into the predictions of any changes. For this reason, the trends and extremes in relevant historical data relating to the sea-bed and shoreline should be identified, and the reasons for their existence should be understood (e.g. mobile sandwaves are present in some areas of the North Sea). Where sand waves occur, their height and mobility should be assessed when designing a structure in their vicinity.

13 Long waves

13.1 General

*COMMENTARY ON **13.1***

Long-period waves with periods typically of the order of 25 s to several minutes have been measured and observed at many sites around the world. Velocities and run-up from long waves affect nearshore sediment transport, beach morphology, harbour oscillation, overtopping/stability and energy transmission through breakwater structures. Local morphology, structure configuration and basin/harbour shape and size can amplify or dampen the effects of the incident or reflected long waves.

Large vessels at their moorings can have natural periods of oscillation that are similar to those of long waves. The result is that the moored vessels can move in resonance with long waves and, because the damping of such slow movements is small, large mooring loads can develop.

There are various possible causes of long waves, including:

- *moving pressure fronts;*

- *wave grouping effects;*
- *tsunamis.*

The designer should assess the potential for long wave conditions to develop at the site and hence the likely risk of long waves inducing harbour resonance/seiching, excessive mooring loads or other adverse impacts. If the risk of long waves is present, new facility layouts should be planned accordingly and/or operational measures taken to reduce the risks in new or existing facilities.

Where necessary, wave measurements should be performed at the site of interest, with subsequent specialist analysis and interpretation to confirm the presence and nature of any long wave energy.

NOTE 1 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. If the damping of such motion is small the disturbance will still be present some time after the pressure front has passed.

NOTE 2 A depression in the mean water level occurs beneath a 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 is known as set-down beneath wave groups, also referred to as bound long waves.

NOTE 3 When the set-down reaches the coastline, the primary wave system (wind-waves and swell) is normally dissipated, but because of its long wave character, the energy in the set-down (bound long waves) is unlikely to be completely dissipated and long waves of similar period are likely to propagate back out to sea (referred to as free long waves). This can lead to a partial standing wave pattern known as surf beat.

NOTE 4 When long waves approach with an oblique angle of incidence, they can produce variations in set-up and wave run-up (of primary waves) and hence cause greater damage to coastal structures.

13.2 Tsunamis

*COMMENTARY ON **13.2***

Tsunami waves are usually caused by earthquakes, landslides, slumps or volcanic activity and tend to travel very quickly (hundreds of kilometres per hour) with periods typically in the order of 10 min to 60 min. Although their height in deep water can be relatively small, they can increase in amplitude substantially upon reaching the shore due to wave shoaling. Resonance in bays, wave refraction and reflection from steep bathymetry can further amplify the tsunami height.

In regions where the risk of tsunami waves is relatively high, and/or in the planning and design of sensitive facilities where the consequences of damage or failure could be severe, the tsunami-induced hazard and associated risk should be established. A statistically based tsunami event should be defined using a probabilistic approach accounting for historical and projected future seismic activity in the region of interest, likely fault mechanism and displacement.

Effective and economically feasible counter-measures should be identified to mitigate the consequences. These should include, where appropriate:

a) measures to promote safety through warning systems, evacuation procedures and facilities;

b) measures to preserve operational continuity through appropriate site selection and location of critical infrastructure;

c) structural counter-measures.

Where it is appropriate to design structures to withstand tsunamis, the designer should estimate the incident tsunami scenario (time evolution of water level/run up/inundation and associated velocity and pressure field) and use methods for design against the corresponding hydrodynamic loadings. An appropriate hydrodynamic model should be employed where the consequences of the damage or failure could be severe.

NOTE Further guidance on design considerations for tsunami effects on maritime works can be found in PIANC Report No 112 [6] and PIANC Report No 122 [7].

14 Water levels

14.1 General

The designer should select an appropriate water level (or range of water levels) for the design of maritime structures.

NOTE 1 The water level at the site of interest can be influenced by a range of factors, primarily including astronomical tides and meteorological surges, but also potentially by fluvial/estuarial influences or set-up/set-down due to winds, waves or currents, and in the long-term by climate change effects.

Where appropriate, wave-structure interactions should also be taken into account.

The designer should take into account the influence of water levels in changing water depths and causing associated alterations in the wave and current patterns and, consequently, in the patterns of erosion and accretion. Over long (decadal) timescales, the mean sea-level rise should be taken into account, and in certain short-term (several days) cases, water-level changes due to storm surges (positive or negative) and wave set-up at the shoreline might be relevant.

The designer should assess the potential degree of correlation (dependence) between water level and wave condition events when selecting design event parameters. The most critical case of combined wave parameters and water level should be identified for the structure element being designed, taking account of the combined (dependent) probability of occurrence of the events.

Where available, long-term water-level measurements suitable for the site of interest should be used to inform planning and design of maritime works. Such records should be carefully reviewed for quality and continuity, and an assessment should be made of the need to separate tide and surge components to better understand water-level characteristics at the site.

NOTE 2 The United Kingdom Hydrographic Office (UKHO) provides a wealth of information relating to astronomical tide levels, tide predictions and water-level measurements both in the UK and globally. A further source is the British Oceanographic Data Centre (BODC) which provides access to water-level measurements from a range of sites across the globe based on the Global Sea Level Observing System (GLOSS).

14.2 Storm surge

Storm surges can raise or lower the water level significantly above or below astronomical tide levels, and the designer should therefore make every effort to quantify the magnitude of storm surge likely under a range of design event probabilities (return periods). In locations where a large tidal range and minimal surge is encountered, the design is unlikely to be sensitive to the effect of storm surges, but where storm surges are large (either in isolation or relative to the tidal range), more in-depth understanding should be obtained by the designer.

NOTE 1 The combination of negative surges and low astronomical tides can be significant in the design of certain works, e.g. port entrance channels, toe/scour protection and pump intakes. Significant currents can be generated by tides, by wind, by breaking waves, and by differences in water levels induced by surge, and these can in turn modify local wave conditions.

NOTE 2 In England and Wales, specific guidance relating to extreme water levels is available from the Environment Agency, although such guidance is frequently updated. Elsewhere, information can be obtained from regional authorities and academic bodies, and site-specific studies can be performed.

15 Design event probability and extreme values analysis

15.1 Design event probability

The probability of an event should be characterized by its return period, T_R, a statistical definition, being the period that (on average) separates two occurrences of equal or greater magnitude.

The designer should assess and select the return period event(s) appropriate to the structure being developed. Where the consequence of exceedance would be very serious, e.g. in terms of risk to life or severe operational impact, then a design event with a long return period (low probability of occurrence) should be used, whereas in other circumstances where the consequences are not so severe, it might be more appropriate to adopt a shorter return period event for design purposes. The residual risk to the structure or facility should also be taken into account, in terms of the consequence of an event with longer return period occurring (relative to the selected design event).

Where a structure or facility response is sensitive to the combination of more than two variables/design parameters, e.g. where increased water-levels allow larger wave conditions due to depth-limitation in shallow water, the designer should take into account the cases of independent and dependent (joint) probabilities.

NOTE 1 The relationships between design working life, return period and the probability of an event being exceeded are shown in Figure 1.

In general, the return period of the design event exceeds the design working life. However, the design event can be exceeded in any given year by a higher magnitude event (lower probability/longer return period), and the consequences of this should be taken into account.

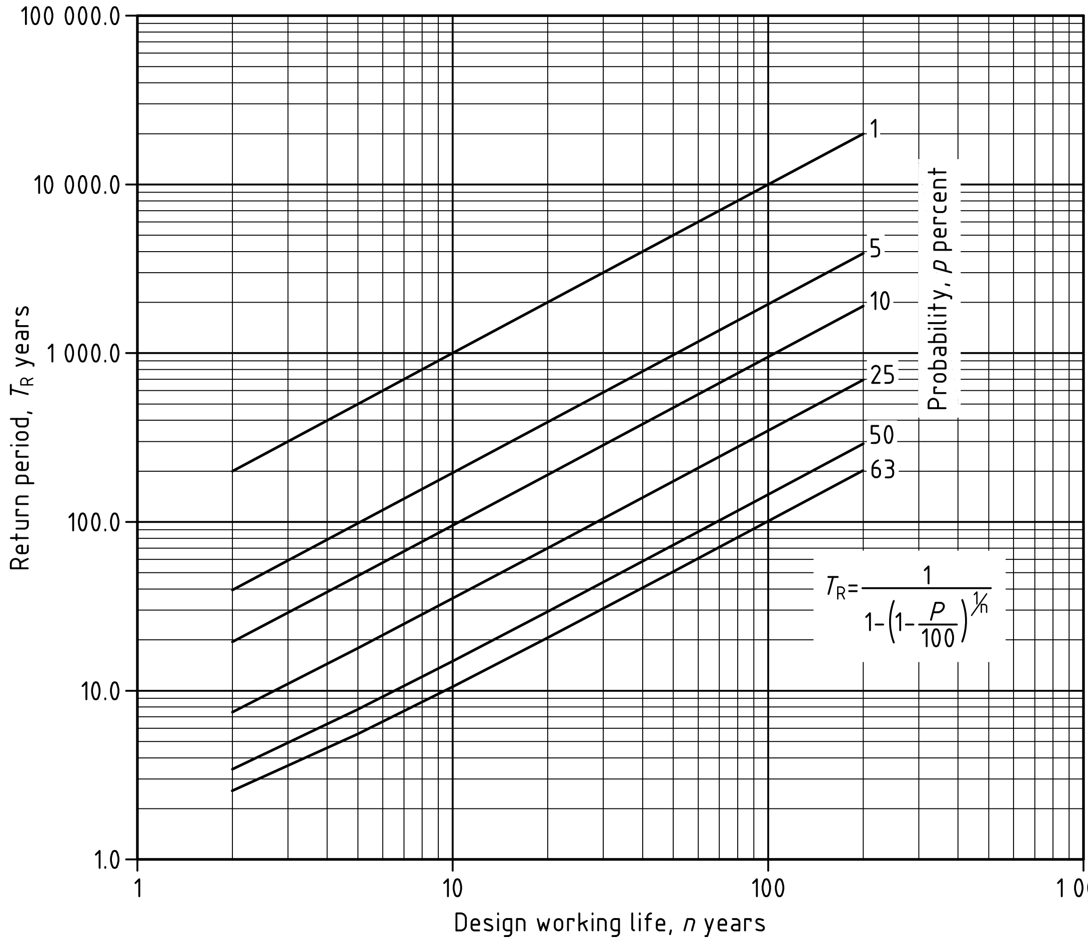
NOTE 2 For an event with a return period of 100 years, there is a 1% probability of occurrence in any one year, even the year following a previous occurrence, and approximately an 18% chance of occurrence in a 20-year period. For an event with a return period of T_R, there is an approximately 63% probability of occurrence within T_R years. In this way it is possible to establish an acceptable level of risk of the design event occurring within a given number of years (the design working life or preferred maintenance interval). For example, if it is established that over a 20-year period it is acceptable to tolerate a 10% probability of occurrence of an event that leads to significant disruption to facilities operations, then the necessary return period from Figure 1 is 200 years.

The degree of redundancy in the structure should also be assessed in conjunction with a sound understanding of likely response or failure modes.

NOTE 3 The consequences of a design event occurring or being exceeded differ substantially. Structures which are designed as “dynamically stable”, such as rubble-mound breakwaters, tend to experience gradual damage. In contrast, structures designed as “statically stable”, such as vertical-faced seawalls, can undergo a more catastrophic failure when the design condition is exceeded.

NOTE 4 Some structure types are suited to adoption of a response-based approach, where the response of the structure or element is assessed against the full range of environmental actions expected, either statistically or in time-series form. In this way, the probability of a certain response can be assessed without the need for simplified treatment (and description in probabilistic terms) of the input parameters which define the actions.

Figure 1 **Relationship between design working life, return period and probability of an event exceeding the normal average**



NOTE T_R is the return period of a particular extreme condition in years. p is the probability of a particular extreme condition occurring during design working life n years.

15.2 Independent extremes analysis

*COMMENTARY ON **15.2***

A widely adopted approach to estimating design event values (e.g. for wave heights or water-levels) with long return periods/low probability of occurrence is to perform an independent extreme values analysis. The basis of such analyses is to assign a probability density function to peak events within an existing data set in order to statistically extrapolate extreme events.

The designer should have sufficient understanding of the approach to be adopted and any inherent assumptions or limitations in the results (e.g. representing what may be a relatively long duration storm event as a statistically derived short duration peak value).

The designer should select an appropriate probability density function that provides a good fit to the data. The confidence interval of the extrapolated values should be determined and the design should take account of the sensitivity to the range of the extrapolated values.

The designer should also take into account the influence of climate change on extreme events during the design life of the structure.

NOTE 1 Further guidance relating to extreme value analysis, probability density functions and joint probability of wave and water-level conditions is given in CIRIA publication C683 [2]. Information relating to probability density functions for application to independent analyses is given in Annex D.

NOTE 2 In the case of wave height estimates, in most cases it is more appropriate to perform extreme values analysis using an offshore data set due to differences introduced to wave populations through nearshore wave transformation processes. In addition, and subject to the wave characteristics at the site of interest, it is likely to be necessary to separate the wave data into directional sectors which are then treated separately for probability density function fitting. That said, each case needs to be assessed on its own merit and a suitable course of action determined by appropriately qualified and experienced specialists.

15.3 Dependent (joint probability) extremes analysis

Because the combination of certain parameters, particularly wave height and water level, are important in the design of many maritime structures, the designer should establish from the available data whether large wave heights and high water levels tend to be dependent or independent of one another.

NOTE 1 The degree of dependence varies according to each site and can also be influenced or partially masked by seasonal or tidal effects.

In the case of partial or strong dependence (or correlation) it is possible for various combinations of water level and wave conditions to lead to similar degrees of structure response, and the designer should take account of these combinations. Different structure responses are maximized by different combinations of water level and wave height (sometimes by wave period as well). It is therefore not obvious which single combination of these parameters will constitute the most onerous design condition, and the designer should assess a number of different combinations for each of the responses analysed.

NOTE 2 In the absence of sufficient data for establishing whether or not high water levels and large wave heights (or other parameters) are correlated, full dependence can be assumed but this produces very conservative estimates. For UK waters, guidance can be found in R&D Technical Report FD2308/TR2 [8]. For other regions, it is advisable to seek local guidance, and in the absence of such guidance, the intuitive/correlation factor approach set out in CIRIA publication C683 [2], ***4.2.5.3*** *can be used.*

16 Wave structure interaction

16.1 Effects of breakwaters and walls on sea states

Maritime structures such as breakwaters, seawalls and quays tend to reflect incident wave energy. In some cases this can lead to agitated wave patterns and implications, which the designer should take into account.

The designer should determine ways of reducing wave reflection through layout design or choice of structure type, or otherwise through modification to existing structures or operational arrangements.

NOTE 1 The degree of reflection depends on several factors including incident wave conditions (and direction), structure slope, geometry and permeability. In some cases, the resulting wave agitation can lead to unfavourable conditions for ships, increase wave overtopping or induce scour at structure foundations.

NOTE 2 Methods for estimation (and reduction of) wave reflection from maritime structures are given in BS 6349-7. Further information is given in CIRIA publication C683 [2].

16.2 Harbour response

*COMMENTARY ON **16.2***

A general requirement in the design of harbours is the ability to estimate the degree of shelter that results from any given layout. The response of harbours is influenced by the incident wave energy (including any long waves), wave refraction over a varying seabed 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 seiching, can cause ships to range at their berths, thereby developing high mooring loads and leading to lines and fenders being broken in severe cases.

The designer should plan harbour layouts to reduce the risk of resonance, and should select structure types carefully so as to reduce reflections, e.g. using rubble slopes wherever possible, as these better dissipate the wave energy.

NOTE 1 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.

NOTE 2 Resonant wave lengths are fixed by the dimensions of the harbour, but the resonant wave period varies with the state of the tide.

NOTE 3 For typical large ship harbours, the longest resonant wave lengths are of the order of kilometres with periods of the order of minutes.

*NOTE 4 Possible sources of excitation are moving pressure fronts, set-down beneath wave groups, surf beats, edge waves and tsunamis (see Clause **13**). 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 vessels and pleasure craft. Seiching can also be caused by wake/pressure waves from passing ships.*

17 Numerical and physical models

17.1 Numerical models

*COMMENTARY ON **17.1***

Numerical models can be employed by the designer to assess a range of issues relating to planning and design of maritime works. These range from hydrodynamic, water quality and sediment transport models, often used to determine environmental impacts of developments, through wave transformation models to derive inshore extreme conditions and wave agitation models to configure harbour layouts and estimate berth availability. CFD approaches are increasingly being applied to localized wave-structure assessments.

The designer should have a clear understanding of the results required from numerical model studies and how these will be used in design or planning, as well as an appreciation of the inherent limitations or uncertainties associated with numerical model studies.

Specialist advice should be sought to properly plan and apply numerical models to assess actions and processes relating to maritime works.

*NOTE Further information relating to wave transformation models is given in Clause **12**. CIRIA publication C683 [2] also provides guidance.*

17.2 Physical models

*COMMENTARY ON **17.2***

This subclause covers the use of physical models for assessing loads, stability of structures and harbour design. Guidance on the use of physical modelling for ship movements and mooring loads is given in BS 6349-1-1:2013.

For most applications, guidance for specifying model tests is given in Guidelines for wave modelling in flumes and basins – Hydraulic model testing in waves *[9].*

For many applications, numerical modelling of flows or non-breaking waves is often sufficient for initial design purposes. Advances in CFD codes and computational power have allowed wave/structure modelling to become more sophisticated, including non-impulsive loadings on walls or similar elements, and some forms of overtopping/transmission.

Designers might however need to use scaled physical models to refine and validate designs when empirical design equations or numerical modelling do not give adequate confidence, particularly for hydraulic performance or structural responses under breaking (impulsive) waves. In some cases, hybrid models might be required where a numerical model may be used to provide input conditions at the boundaries of a physical model, or vice versa. All model testing (physical, numerical and hybrid) is based on simplifications of reality and therefore needs to be used with an understanding of those simplifications and of the inherent limitations of the use of the model test results.

Physical model tests might be particularly appropriate to:

- *assess stability and movement of armour on rubble mound breakwaters or seawalls;*
- *assess loads and/or movement of caissons and floating structures;*
- *identify wave breaking where complicated bathymetry in front of a structure causes significant variations in sea state;*
- *quantify wave interactions with the structure such as overtopping, run up, reflections, toe scour, wave breaking, and forces and pressures on structural elements;*
- *assess wave penetration into harbours where wave diffraction cannot be modelled numerically with sufficient confidence;*
- *assess ship movement and mooring forces.*

Physical modelling cannot on its own be relied on to model sediment transport (those processes cannot be directly scaled in the same way as wave forces), although in carefully selected cases useful assessments of sediment movements might be possible.

All physical model testing should be conducted, supervised and interpreted by experienced and qualified personnel and carried out in facilities with the appropriate specialized equipment. The designer should clearly state the objectives of the modelling so the laboratory can advise on an appropriate modelling approach, measurement methods, and scales. The designer should request information from the modelling laboratory on the direction and magnitude of potential uncertainties/errors due to model and scale effects and measurement techniques. The implications of these uncertainties should be taken into account by the designer.

When the structure is three-dimensional (3D), or when wave action is significantly oblique to the structure, a 3D (basin) model should be used.

NOTE 1 A two-dimensional (2D) model in a wave flume can be useful for rapid development of a cross-section design under normal wave attack or to quantify the comparative effects of different structure variations.

The real wave field should be accurately represented in the model. A spectral distribution of irregular waves should be applied, including bi-modal spectra where the contribution of swell might be significant. The wavemaker and model boundaries should absorb reflected waves so that the overall contribution of reflections at the face of the wavemaker is less than 5%.

NOTE 2 Correct generation of wave groups and second order wave spectra are particularly useful in studies of wave overtopping and/or floating body motions.

The duration of tests should be adequate to represent the range of waves anticipated in the wave field. When the model involves complicated bathymetry or structures, wave heights in the model should be calibrated against measured waves on site where possible. A structure should be tested for wave conditions up to and exceeding the design wave condition. Overload testing should be included to test reserve capacity, especially where the key responses are expected to be sudden or the consequences severe. Where appropriate, repeat tests should be run to improve confidence in the anticipated structure performance.

NOTE 3 Repeating tests with the same wave conditions can lead to differences in overtopping volumes and unit stability in rubble mound structures, owing to small movements in the structure or differences in the applied wave conditions.

Loads and pressures from breaking waves vary spatially and can also reach high peaks over short durations. The ability to measure these loads accurately in a scaled model depends on the sampling rate of the measurement device, the area covered by the device(s) and scaling effects of short duration loads. The choice of measurement device(s) should take into account the likely dynamic response of the structure as well as the nature of the impacting waves and the technology available. The designer should seek advice from the testing laboratory on the accuracy and uncertainties associated with the measurement devices used so this can be taken into account in the design.

Section 4: Actions, loads and hydraulic responses

*COMMENTARY ON SECTION **4***

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 and combinations of actions is given in Section **2**.*

18 General recommendations for actions, loads and hydraulic responses

18.1 Basic loads

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 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 assessed separately in determining what provisions are to be made.

The loading design criteria adopted should be clearly stated and recorded in the facility operating manual. 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.

18.2 Dynamic response

*COMMENTARY ON **18.2***

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.

*NOTE An approximate method for estimating dynamic amplification is given in Clause **34** and guidance on the particular problem of vortex shedding is given in **E.2**.*

Table 4 **Typical frequencies of environmental forces**

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

18.3 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 **3**).

NOTE 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, e.g. 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 taken into account with respect to the wave loading spectra, but the techniques described in Clause **29** should be used to deal with the coupled motions of the ship and non-linear behaviour of the mooring lines).

18.4 Fatigue

Structural members subjected to fluctuating loads can suffer from fatigue failure, and this should be taken into account in the design.

*NOTE For maritime structures, problems due to fatigue are most likely to arise in steel members subjected to wave loading. (See **25.3** for fatigue analysis.)*

19 Soil pressures

NOTE Guidance on the calculation of soil pressures is given in BS 6349-1-3.

For the purposes of calculating soil pressures:

a) live loading on surfaces should be determined as described in Clause **31** and Clause **32**;

b) extreme water levels should be derived as described in Clause **23**;

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.

20 Winds

Wind actions should be determined in accordance with BS EN 1991-1-4.

NOTE 1 The characteristic values for wind velocity given in BS EN 1991-1-4 have annual probabilities of exceedance of 0.02, which is equivalent to a mean return period of 50 years. The fundamental value of the basic wind velocity, $V_{b,Q}$, is the characteristic 10 min mean wind velocity, irrespective of wind direction and time of year, at 10 m above ground level in open country terrain with low vegetation such as grass and isolated obstacles with separations of at least 20 obstacle heights.

The basic wind velocity should be factored for height, terrain roughness, direction and return period.

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

*NOTE 2 Recommendations and general guidance on wind speeds to be used with design situations that include actions on/from ships are given in BS 6349-1-1:2013 and Clause **29**.*

NOTE 3 Design wind speeds for specific load cases might be limited by operational practices. For instance, crane operations might be suspended at specified wind velocities. In this case it would not be necessary to take the maximum operational loads in combination with the characteristic wind speed for a long mean return period.

NOTE 4 In cases where wind loading is critical, values of aerodynamic force coefficients might need to be obtained from wind tunnel tests.

21 Snow and ice

*COMMENTARY ON CLAUSE **21***

For the coastal areas around the British Isles, accumulated snow is unlikely to affect the design of heavier maritime structures significantly.

Ice loading can occur on maritime structures from the following mechanisms:

a) as imposed loads from the formation of ice on the superstructure or structural element. This can occur with changing water levels or with splash from waves particularly in low salinity environments. The loads can be significant in the case of ice forming on relatively small elements such as solid walkways and fenders;

b) as imposed loads from ice flows;

c) as ice pressure from a build-up of ice adjacent to the structure or, for example, by bow waves from passing vessels;

d) as ice pressure caused by thermal expansion of ice.

In the recent past, loading from floating sea ice has not been a problem around the British Isles and need not be considered for structures whose design life is of the order of 50 years.

Snow loads should be taken into account in the design of ancillary structures such as cargo sheds, port buildings and cargo handling installations, for which the appropriate imposed roof loadings given in BS EN 1991-1-3 should be used.

The designer should take into account the risk of icing of the superstructure and small structural elements and the thermal expansion of small pockets of trapped ice in the design of maritime structures, and should design for ice loads if the risk is significant within the working design life.

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, the guidance in the National Annex to BS EN 1991-1-3, or National Annex to other equivalent European implementation of EN 1991-1-3, should be followed.

NOTE Guidance is given in Recommendations of the Committee for Waterfront Structures, Harbours and Waterways *[10];* Planning and design of ports and marine terminals *[11] and* Ice engineering design of marine piling and piers *[12].*

22 Thermal actions

Design for thermal actions on maritime structures should conform to BS EN 1991-1-5.

Design allowances for temperature changes in suspended decks should meet the requirements specified for temperature changes in bridges in BS EN 1991-1-5.

For a maritime structure that is over or adjacent to a body of water, an allowance should be made to the temperature difference component for the cooling or the warming effect of the water or ice. Temperature measurements as described in BS 6349-1-1:2013 should be carried out, where practicable, to evaluate this effect.

Temperature difference statistical data should be collected from similar structures that have similar structural form and similar temperature profile.

Where temperature difference statistical data due to the cooling or warming effect of the water body or ice is not available, or where it is not practicable to collect such statistical data, the air temperature due to the unfavourable effect of the water body or ice should be taken as the water or ice temperature. The air temperature due to the favourable effect of the water body or ice should be taken as 60% of the benefit gained by the difference in air and water or ice temperatures.

23 Water-level variations

Maritime works should be designed to withstand safely the effects of the extreme range of still water-level from extreme low water to extreme high water expected during the design working life of the structure. These extremes should be established in relation to the purpose of the structure and the accepted probability of occurrence (see Clause **15**), but should normally have a return period of not less than 50 years for permanent works.

NOTE 1 Extreme water levels, which can be caused by a combination of astronomical tides, positive or negative surges, seiches, wind or wave set-up and freshwater flows (see Clause **14***), are required for the evaluation of:*

a) wave overtopping/over-flow;

b) hydrostatic pressures, including buoyancy effects;

c) soil pressures on quay walls;

d) lines of action of mooring and berthing actions;

e) forces from other floating objects and wave actions.

The effects of climate change should also be taken into account, including increases in mean sea level and increased storminess leading to changes in the frequency and magnitude of storm surges and storm wave conditions (see Clause **9**).

NOTE 2 Reduced safety factors are appropriate in relation to mooring and berthing forces, forces from other floating objects and wave forces, when assessed in conjunction with extreme water levels.

NOTE 3 Some aspects of design are most sensitive to low water levels (e.g. scour at the toe) and some are most sensitive to high water levels (e.g. overtopping assessment of seawalls, breakwater crest design and wave slam assessment on the underside of quay decks).

24 Current actions

24.1 General

For design purposes the current velocity should be established primarily in relation to the purpose of the structure and the accepted probability of occurrence (see Clause **15**). For most structures, the design current velocity is the maximum value expected at the site during the design life (considering climate change influence) and should normally have a return period of not less than 50 years for permanent works.

Current velocities and directions used for design purposes should be based on direct field measurements (over an appropriate duration) or calibrated (and validated) numerical model studies, and may also involve an extreme values analysis. In the absence of such information, currents can be estimated from hydrographic charts/reports or using empirical approaches, but should be treated with caution and are likely to be suitable for concept design purposes only.

The distribution of current velocities and directions through the water column should also be taken into account, e.g. where surface velocities and directions might differ substantially from bed velocities and directions.

NOTE 1 Actions imposed directly by tidal and/or fluvial, wave-induced, wind- or density-driven currents on maritime structures can be classified as:

- *"drag" or "in line" forces, parallel to the flow direction; or*
- *"cross-flow" forces, perpendicular to the flow direction.*

Drag forces are principally steady and the oscillatory component is significant only when its frequency approaches the natural frequency of a 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.

*NOTE 2 Current actions that are applied to vessels can then be applied to port structures via the mooring system. Recommendations for the evaluation of these actions are given in Clause **29**.*

NOTE 3 For operational conditions, currents are usually taken at mean spring tides.

24.2 Steady drag force

NOTE 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 provided in Annex E.

The values used for the drag coefficient and area of member normal to flow (C_D and A_n) should be determined taking due account of the effect of marine growth on cross-sectional dimensions. For marine growth in UK coastal waters, guidance can be found in Annex F. In other regions, local guidance should be sought or, in the absence of such information, the guidance in ISO 21650:2007, Annex G should be followed.

Where the incident current velocity is non-uniform or the structural member is gradually tapered, the total force and the line of action should be determined by integration. Where the structure is fully submerged and end effects can be significant, or where floating or of significantly non-uniform shape, model tests should be conducted to measure the drag force.

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. Inertial forces should also be taken into account where necessary in such situations (see Clause **25**).

24.3 Flow-induced oscillations

*COMMENTARY ON **24.3***

A slender structure situated in a current can experience fluctuating forces, both in-line and cross-flow due to the shedding of vortices downstream of the structure. The frequencies of the fluctuating forces are directly related to the frequency of the vortex shedding. When the structure 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 structural element or that of the whole structure. This is due to a feedback system sometimes referred to as "locking on". If the inherent damping of the structure is sufficient to suppress the motion developing, the locking on does not occur.

The designer should take into account flow-induced oscillation/vibration due to vortex shedding and make allowance for suitable resistance in terms of strength and fatigue. Piled structures are particularly vulnerable to this type of oscillation during construction and the designer should determine whether restraint to the pile heads is required immediately after driving to prevent the possibility of oscillation in the cantilever mode.

NOTE Flow-induced oscillation can be assessed using a direct numerical solution such as CFD modelling where practical. For circular section piles (or tubes/cylinders of other types), the expressions given in Annex E can be used to estimate critical velocities and mass damping coefficients. Non-circular sections are also subject to flow-induced oscillations but at higher critical flow velocities, and once initiated can occur with greater amplitude. Non-circular sections can be checked using the expressions provided in Annex E for circular sections, using the maximum dimension normal to the direction of motion (in place of the cylinder diameter).

If the actual flow velocity is close to the calculated critical flow, the designer should obtain specialist advice, and model tests should be used where practical.

24.4 Scour due to vessels

Ships' propulsion systems are capable of producing very high flow rates at seabed level, so the scour effects from vessel propulsions systems should be assessed when designing port structures. The following parameters should be taken into account:

- geotechnical parameters and nature of the seabed;
- seabed level and tidal range;
- vessel draught;
- vessel propulsion type and power;
- vessel berthing and departure procedures (whether under own power or tug-assisted);
- the requirement for the use or trial of the ship's propulsion systems whilst on the berth;
- type and layout of quay structure;
- location and power of any bow thrusters.

Where the berth configuration includes a return wall or piles adjacent to the ship's propulsion system, the effect of these should be taken into account.

NOTE 1 The nature of the seabed in conjunction with the ship's propulsion system dictates whether scour protection measures are required. Scour protection might also be necessary to reduce the amount of seabed material entering the ship's mechanical systems, and/or to restrict impingement corrosion of the structure.

NOTE 2 Where vessels have jet propulsion systems, these produce very high flow rates (up to 20 m^3/s) with complex turbulent flows which can directly affect the seabed. For these vessels, particular care is needed in the design and specification of any scour protection system; generally specialist knowledge or modelling is required.

NOTE 3 Further guidance can be found in PIANC Report No. 180 [13].

25 Wave actions

25.1 General

Design wave forces should be derived from the design wave parameters defined in **25.2**, either by calculation, as described in this clause, or by physical model tests.

NOTE 1 The nature and magnitude of wave actions depends on the wave height, period and spectral form, as well as the hydrodynamic regime and the dimensions and physical properties of the structure. The nature of the wave–structure interaction can be broadly classified in simple terms by determining 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:

- *for $W_s/L < 0.2$, Morison's equation applies (see **25.4**);*
- *for $0.2 < W_s/L < 1$, diffraction theory applies;*
- *for $W_s/L > 1$, reflection applies (see **25.6**).*

Diffraction theory is likely to be of limited application to the maritime structures covered by this part of BS 6349.

*NOTE 2 In many cases linear wave theory provides a sufficient approximation, but in shallow water, where the depth to deep water wave length ratio, d/L_o, is less than 0.1, as the wave form starts to deviate significantly from sinusoidal, it might be necessary to use either spectral, solitary or cnoidal wave theory for greater accuracy. Details of these theories are given in Section **3**.*

*NOTE 3 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 **25.3**.*

*NOTE 4 Where the structure is fully submerged and reflection effects can be considered negligible, the recommendations given in **25.5** may be used for sub-sea elements such as pipelines.*

*NOTE 5 In the case of horizontal structures or members above the water level, such as suspended decks and beams, where wave uplift and/or wave slam actions might occur, the recommendations given in **25.7** may be used.*

*NOTE 6 For crest structures above the water level, such as those typically found on breakwaters and seawalls, where horizontal wave impact in combination with hydraulic uplift pressures might occur, the recommendations given in **25.8** may be used.*

*NOTE 7 For floating structures such as lightly-loaded pontoons or heavy-duty pontoons with a limited degree of wave attenuation, where wave impact and uplift pressures act in combination, the recommendations given in **25.9** may be used.*

NOTE 8 For the purpose of most structural analyses, the resistance or response of the structure (or element) is assessed per unit length (on plan). In most cases, when addressing overall stability, dynamic response and some design detailing aspects, an understanding is required of the plan-length over which the wave action applies within any one time period. The wave action over the plan-length varies according to the predominant direction of wave approach, the geometry of the structure, the directional spread around the mean wave direction and the energy density/frequency distribution (wave period/wave length). In practical terms, for an assumed case of an infinite length structure perpendicular to the wave direction, the plan length of structure over which the wave action applies within a short time period might be expected to be at a minimum for a short-crested sea-state, especially in a highly reflective environment (with alternating pattern of application), up to a maximum for a long-crested swell sea where reflection is minimal.

25.2 Design wave parameters

COMMENTARY ON ***25.2***

The required design wave parameters depend on the analytical approach being adopted. Many empirical approaches are based on the significant or maximum wave height, H_s or H_{max}, and the mean or spectral peak period, T_m or T_p, respectively. An increasing number of analytical approaches (both empirical and numerical) are now based on spectral wave parameters such as the spectral significant wave height, H_{m0}, and spectral wave period, $T_{m-1,0}$.

For analysis of structures having static or quasi-static response characteristics to wave loading, in most cases a limited number of wave, water-level and current conditions, focusing on operational, extreme or accidental design scenarios, should be taken into account. Where the dynamic response of the structure to wave action is significant, account should also be taken of the possible range of wave periods, directions and associated (maximum or significant) wave heights that would result in the greatest dynamic magnification.

Design wave parameters should be obtained by the methods described in Section **3**, taking due account of the local conditions relevant to the site or structure. In all cases, the design wave parameters should be derived for a range of probability-based events appropriate to the intended design working life and maintenance philosophy of the structure.

25.3 Fatigue analysis

For fatigue analysis, an assessment should be made of the number of waves likely to occur during the design working life of the structure, taking into account ranges of height, period and direction.

Steel structures that are subject to fatigue should be designed in accordance with BS EN 1993-9.

NOTE Guidance on fatigue assessment for offshore steel and concrete structures is given in BS EN ISO 19902 and BS EN ISO 19903. The applicability of methods for offshore situations in coastal waters needs to be assessed.

25.4 Wave action on vertical or inclined cylindrical structures

In assessing the width, diameter or cross-sectional area of the structure or member, allowance should be made for the build-up of marine growth on the structure (see **24.2**).

Expressions for the water particle velocity, acceleration and instantaneous water level (U, $\dot{U}$ and η respectively) should be derived from appropriate wave theory (see **25.1**). For vertical members, only horizontal particle velocities and accelerations need be evaluated. For inclined members, vertical components should also be taken into account. For closely grouped members, the inertia force coefficient, C_I, can increase and should be determined from physical model tests.

*NOTE 1 For non-breaking wave situations where the structure or member presents a relatively narrow (slender) obstruction to the passage of waves (see **25.1**), 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.*

NOTE 2 Any current velocities being added vectorially to the wave particle velocities are normally 90° out of phase for pure waves, but this changes in the presence of currents.

NOTE 3 Further guidance for estimation of wave actions on vertical slender cylindrical structures is given in Annex E.

NOTE 4 Wave actions on large diameter cylindrical elements can be calculated using wave diffraction theories, CFD approaches or physical model tests.

25.5 Wave action on sub-sea elements

*COMMENTARY ON **25.5***

*Wave action on sub-sea elements in this context is assumed to mean objects or structures installed on the seabed as either gravity-based structures or structures embedded into the seabed with limited overall height, e.g. piles would be assessed in accordance with **25.4**. Wave action applied is assumed to be from non-breaking waves. For situations in shallow or tidal areas where shoaling and breaking waves occur, it is advisable to obtain specialist advice on the site-specific conditions to account for the effects of such loading.*

Wave loading should be assessed on the exposed part of the structure that will be subject to the combined actions of waves and the associated current.

The seabed acts as a boundary layer and this effect should be taken into account. A significant effect of the boundary layer is to modify the effective shape to assess the appropriate hydrodynamic coefficients for drag, lift and inertia, and thus a full examination of the object profile should be made to derive the appropriate coefficients.

NOTE 1 The above seabed part of the structure may be examined to re-define the profile accounting for the seabed as a boundary layer, and this can often be done by assuming a mirror image of the object below the seabed line. This resultant profile can be used to assess the applicable hydrodynamic coefficients. For example, a semi-circular profile placed on the seabed has similar hydrodynamic coefficients to a cylindrical object in free water space.

NOTE 2 DNV-RP-H103 [14] gives methods of derivation for the correct drag lift and inertia coefficients to establish the correct coefficients for various shapes and profiles.

NOTE 3 Morison's equation may be used to establish the loads from both water particle velocity and accelerations.

As peak wave velocity and accelerations are out of phase for small structures, the combined loads of wave velocity and accelerations through each part of the wave phase, including the associated current loading, should be assessed to establish the maximum loading.

Wave velocities and accelerations should be derived from the appropriate theory as outlined in **25.1**, but for shallow water higher order wave theories such as Stokes V and the higher stream function theories might be more appropriate as these are a function of wave height, water depth and the square of the wave period. Velocities and accelerations should be assessed at the appropriate water depths and applied to the structure accordingly.

NOTE 4 For low structures, e.g. less than 3 m with a relatively small vertical profile, the velocity and accelerations calculated at the top of the structure may be applied to the whole structure to obtain the overall forces.

NOTE 5 An alternative approach for assessing wave and current velocities over pipelines is given in DNV-RP-F109 [15], which derives velocities for cylindrical objects and takes account of additional factors such as seabed surface roughness, permeability and wave to current ratios.

Depending on the water depth and local conditions, an allowance should be made for the effect of an increased profile caused by marine growth on the structure.

25.6 Wave action on walls and breakwaters

25.6.1 General

The designer should take into account wave action on walls (seawalls or quay walls) and breakwaters, and in doing so should qualitatively assess the nature of the wave action as well as quantifying the magnitude and direction of design wave parameters. The design should take into account hydraulic pressures, hydraulic responses, uplift and buoyancy effects, as well as geotechnical and climatic actions affecting the structure.

NOTE 1 The magnitude of wave action depends not only on the hydraulic conditions incident on the structure (water-depth, wave height, wave period and wave direction) but also on the geometrical and dynamic characteristics of the structure. These can affect wave–structure interaction in the immediate vicinity of the structure, i.e. by inducing wave breaking, and hence influence the effective load experienced by the structure.

Based on the range of probability-based events to be assessed to address operational, extreme and accidental risks, the designer should distinguish between three different wave condition regimes, namely non-breaking (reflecting), breaking and broken waves, and select a means of analysis suitable in such conditions. The designer should also take into account secondary actions due to hydraulic responses (see Clause **26**).

NOTE 2 Further guidance and recommendations for assessing wave actions on walls and breakwaters can be found in BS 6349-7, MAST III/PROVERBS Probabilistic design tools for vertical breakwaters *[16], CIRIA publication C683 [2] and ISO 21650:2007.*

25.6.2 Wave action on vertical and near vertical faced structures

Where appropriate, generally in the case of non-slender vertical or near vertical faced structures, the designer should further assess whether wave loads are classified as pulsating or impacting (impulsive), and hence whether quasi-static or dynamic analysis techniques are appropriate.

The designer should assess positive pressures on the structure due to incident waves (toward the structure, often referred to as shoreward) and negative pressures on the structure due to reflecting waves (away from the structure, often referred to as seaward) and in doing so should assess the relative position and influence of wave crests and troughs that could generate a worst case.

NOTE 1 In the case of structures that are sensitive to negative pressures, particular care is required and the guidance is given in Wave forces on vertical and composite walls *[17] and* Seaward wave loading on vertical coastal structures *[18].*

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 and short durations. The designer should therefore take into account the peak intensity and duration of the impulsive pressures acting on the structure.

NOTE 2 Where structures are founded on a porous or non-cohesive foundation or soil, such as caissons founded upon a rubble mound, the stability of the structure foundation requires particular attention due to the action of scour or partial liquefaction due to wave-induced pore pressure and soil deformation.

25.6.3 Wave action on sloping faced structures

When dealing with non-slender sloping structures subject to breaking waves (or that might cause waves to break), the designer should assess the breaker type as well as the magnitude and direction of design wave parameters.

The stability and integrity of the cover layer (armour) under wave action should be assessed, including toe structures, seaward slope, crest (including any crest structures) and leeward slope.

Impact and destabilizing loads due to wave run-up and overtopping should be taken into account, as well as direct loading due to incident wave action. Hydraulic actions within the structure due to porous flows should also be taken into account.

For all design assessments, the designer should use empirical, numerical or physical model techniques as appropriate to resolve the physical process to a level of accuracy suited to the design stage and relative risks of the project.

NOTE Particular care is necessary in the case of structures subject to breaking waves because of the difficulties of accurately modelling or estimating the wave action and loadings that sometimes occur in the prototype.

25.7 Wave action on horizontal structures

For horizontal structures or members that could be affected by wave action, the designer should take account of all hydraulic loads acting upon the structure.

For horizontal structures exposed to a high degree of wave action, such as bridge decks, jetties or platforms in cyclone prone regions, or where the operational risks or consequences of failure are severe, physical model tests should be used where practicable to support detailed design.

NOTE 1 Further guidance on the type of hydraulic loads that can act upon a horizontal structure or member is given in Annex E.

NOTE 2 For concept design purposes, further guidance can be found in Piers, jetties and related structures exposed to waves – guidelines for hydraulic loadings *[19] and* Wave-in-deck loads on exposed jetties *[20].*

NOTE 3 Owing to the impulsive nature of the loading, the dynamic response of the member can be particularly significant.

25.8 Wave action on crest structures

*COMMENTARY ON **25.8***

Crest structures are often adopted on breakwaters and seawalls and can be simple structures, whose only function is to provide an access roadway for inspection and maintenance, or more substantial structures incorporating a wave wall to reduce overtopping and/or incorporating landside features required for services or operational activities.

Crest structures are most commonly designed as gravity structures, primarily relying on their self-weight to resist sliding and/or overturning actions imposed by wave impact and other hydraulic actions. Hydraulic actions are normally considered in terms of horizontal and uplift pressures, where horizontal pressures tend to be associated with impulsive wave action (high peak values of very short duration) and uplift pressures with pulsating hydraulic loads due to flows and water-level fluctuations within the structure.

Guidance on the analysis of wave actions and associated stability assessment of crest structures is given in CIRIA publication C683 [2].

Stability assessments tend to be based on empirical approaches, leading to the hydraulic actions and resulting structural response being treated simultaneously. Wave actions depend on the degree of wave run-up, geometry of the crest structure, the level of any armour layer present in front of the structure and the porosity of any underlayers or core material. The empirical methods given in the following publications are commonly adopted:

a) Hydraulics Research Report SR 146 [21];

b) PIANC Bulletin No. 102 [22];

c) Coastal Engineering No. 37 [23];

d) Wave forces and overtopping on crown walls of rubble mound breakwaters – An experimental study [24].

These methods are based on scaled hydraulic physical model studies and should therefore be used for concept design within their respective ranges of applicability only. Where appropriate, it is advisable for all the methods to be applied by the designer to provide an indication of the potential variability of results.

The designer should take into account the most onerous combination of horizontal and vertical (uplift) hydraulic actions on the crest structure, with an associated acceptable probability of occurrence and consequence.

It is preferable to carry out specific hydraulic model tests in the case of important structures/high-risk situations. In addition to the general recommendations in this subclause regarding physical model tests, when assessing crest structures the testing should include, as applicable:

- appropriate scaling of the crest structure for geometry and/or stability depending on whether the structure is to be moveable (for the purpose of testing) or artificially fixed (for the purpose of testing) to allow pressure/force measurements to be made;
- taking account of 3D effects in the prototype, e.g. due to oblique wave attack, and hence appropriateness of 2D and/or 3D testing, and undertaking comparisons with available empirical methods (see the Commentary on the present subclause) which are based on 2D flume tests only;
- subjecting the model structure to wave conditions exceeding those used for the extreme design case to determine the reserve of stability (if moveable for the purpose of testing).

Where the seaward face of the crest structure projects above the crest elevation of the armour, or where the crest structure offers protection to important facilities or installations, there should be a relatively high reserve of stability. In such cases, the upper part of the wave wall should be designed to fail before the main structure moves, if the implications of this are deemed acceptable.

The designer should take into account the increased wave actions that could arise from loss or displacement of armour and to check that the reserve of stability under this condition is sufficient.

Unless pressure/force measurements clearly show otherwise (based on sufficient test data and/or repeat tests), when calculating stability of the crest structure against overturning, full uplift should be assumed under the entire width of the base.

The designer should also take account of the potential for local slip failures in the top of rubble mound structures (where crest structures are included), and downward hydraulic pressures due to overtopping, particularly in terms of damage to the crest structure and adjacent structures or facilities.

25.9 Wave action on floating bodies

The assessment of wave action on floating bodies and their moorings should take into account the expected range of hydraulic actions from wave and current conditions for operational, design and accidental design scenarios for a range of water depths.

NOTE 1 For floating structures, a modified form of Morison's equation (see Dynamics of fixed marine structures *[25]) may be used to assess the wave actions for concept design. Numerical hydrodynamic modelling or physical model tests may be carried out for detailed design.*

NOTE 2 Further guidance and recommendations are expected to be given in the new version of BS 6349-6, which is being revised at the time of publication of BS 6349-1-2.

26 Hydraulic responses

The designer should take account of potential hydraulic responses of maritime structures when exposed to wave, water level or current actions.

NOTE 1 These secondary responses can generate actions which require quantification and appropriate measures to be put in place to reduce associated risks. Hydraulic responses might include:

- *wave transmission (through porous structures such as rubble mound breakwaters or floating bodies) which can lead to destabilization of rear-side armour (on rubble mounds) or incident wave action on nearby structures or shorelines;*
- *wave reflection from breakwaters, seawalls, quays, jetties, floating bodies or submerged obstructions;*
- *wave run-up and overtopping leading to direct damage due to impacting pressures or indirectly due to flooding;*
- *deformation of reshaping structures leading to modified design conditions;*
- *scour of structure toe/foundations that can lead to damage or undermining.*

When assessing hydraulic responses, the designer should determine whether such actions are inherently addressed within the design methodology being adopted or whether separate treatment is required.

NOTE 2 The characteristics of hydraulic responses vary widely depending on the structure type and conditions to which it is exposed. Methods to assess hydraulic responses of walls and breakwaters are covered in BS 6349-7 and further information is available in CIRIA publication C683 [2].

27 Earthquakes

*COMMENTARY ON CLAUSE **27***

The assessment of seismic actions for maritime structures is carried out using the following process.

- *Assess the characteristics of the structure, assign an importance class, identify the reference peak ground accelerations and assess the implications of the performance requirements (**27.2**).*
- *Assess the seismic action (**27.3**).*
- *Select the design approach (**27.4**).*

It is expected that future parts or revisions of BS 6349 will include guidance on the seismic design and detailing of maritime structures.

27.1 General

The design of maritime structures to resist earthquake actions should be carried out in accordance with BS EN 1998, except where recommended otherwise below.

NOTE 1 Additional guidance is given in PIANC Report of MARCOM Working Group 34, Seismic design guidelines for maritime structures *[26] and ASCE 61-14 [N1].*

NOTE 2 It is not possible to separate entirely the assessment of seismic actions from the design decisions leading to choice of concept and details.

27.2 Structural performance

27.2.1 Structural categorization

NOTE 1 Additional considerations might apply to petrochemical, LNG, LPG and other similarly hazardous industrial activities.

Maritime structures should be assigned an importance class, as shown in Table 5, based on an appropriate risk assessment.

The assignment of the importance class should be made following a discussion with the end user and any appropriate regulatory authorities where the risks and implications of using each importance class are explained and an agreed assignment made following an assessment of the risks.

NOTE 2 The performance factor γ_2 has been specified as that which, when applied to the reference return period T_{NCR} for earthquake level L2, results in the same return period for importance classes III and IV as that required by ASCE 61-14 [N1] for the controlled and repairable damage criteria for structures assigned a "high" design classification (475 years).

For importance class II, the value of γ_2 should correspond to the return period required by ASCE 61-14 [N1] for the controlled and repairable damage criteria for structures assigned a "moderate" design classification (224 years).

*NOTE 3 The value of γ_2 can be estimated in accordance with BS EN 1998-1:2004+A1, **2.1**(4) using an appropriate value for the exponent k. If the value of k is not known, a value of γ_2 of 0.78 may be adopted for importance class II (this corresponds to a value of 3 for the exponent k).*

27.2.2 Reference peak ground accelerations

Two levels of reference peak ground acceleration for soil type A, a_{gR}, as shown in Table 6, should be selected in order to carry out the performance requirements for maritime structures.

Table 5 **Importance classes for maritime structures**

Importance class	Importance factor, γ_I	Performance factor, γ_2	Maritime structure classification
IV	1.4	1.0	Critical structures that, if damaged, devastate economic or social activities or could lead to extensive loss of human life Structures the failure of which will lead to severe environmental consequences Critical structures that handle hazardous materials
III	1.3	1.0	Critical structures with potential for appreciable reconstruction cost and which if damaged could lead to loss of human life Key structures that are required to be serviceable for recovery from earthquake disaster Structure that, if damaged, is difficult to restore Structures the failure of which will lead to significant environmental consequences
II	1.0	Refer to Note 3	Ordinary structures not belonging to the other categories
I	0.85	Not applicable	Small easily restorable structures

Table 6 **Characteristics of the reference peak ground acceleration, a_{gR}, at the site of a maritime structure**

Earthquake level	Probability of exceedance in 50 years	Reference return period
L1	P_{DLR} = 41%	T_{DLR} = 95 years
L2	P_{NCR} = 10%	T_{NCR} = 475 years

Where it is known that the peak ground acceleration, a_g, is less than 0.04g for the level L2, seismic design is not normally required except for structures with importance class IV (see Table 5).

NOTE 1 In regions of very low seismicity, such as the UK, the seismic profile is different to that in regions of high seismicity. In certain circumstances this requires a larger reference return period.

NOTE 2 For countries having a National Annex to EN 1998, the reference return period is given in the appropriate National Annex.

27.2.3 Seismic performance

Maritime structures should be designed to meet the performance requirements given in BS EN 1998-1:2004+A1, **2.1**(1) as follows:

- earthquake level L1: damage limitation requirement;
- earthquake level L2: no collapse requirement.

In addition, a controlled and repairable damage requirement should be checked at level L2. For the controlled and repairable damage requirement the structure should achieve the following performance:

a) the structure responds in a controlled and ductile manner, experiencing limited inelastic deformations at locations where repair is possible;

b) the required repairs result in a loss of serviceability for no more than several months; and

c) there is no loss of containment of materials in a manner that would pose a public hazard.

NOTE 1 The definitions of the "damage limitation" and "no collapse" requirements are given in BS EN 1998-1.

NOTE 2 The controlled and repairable damage requirement is taken from ASCE 61-14 [N1], which includes additional guidance. Further guidance is included in PIANC Report of MARCOM Working Group 34, Seismic design guidelines for maritime structures *[26].*

The reference peak ground acceleration on type A ground a_{gR} and the importance factor γ_I should be used in conjunction with BS EN 1998-1, to define the design ground acceleration on type A ground, a_g.

For the controlled and repairable damage requirement, the peak ground acceleration on type A ground, a_g, should be replaced with $a_{g2} = \gamma_2 a_{gR}$, where γ_2 is as listed in Table 5 and a_{gR} is the reference peak ground acceleration for earthquake level L2.

NOTE 3 Facilities for the conveyance of hazardous substances might need supplementary checks as defined by the Seveso III Directive [27].

Structural analysis to assess the performance of the structure should replicate the situation during the design earthquake. The performance assessment should take account of the effects of seismic actions during the course of the design earthquake.

27.3 Assessing seismic excitations

27.3.1 Seismic action

The reference peak ground acceleration, a_{gR}, for the bedrock (type A soil in accordance with BS EN 1998-1:2004+A1, Table 3.1) should be determined for the site of each maritime structure to be designed for L1 and L2 earthquakes.

NOTE 1 a_{gR} and spectral shape can be found from the following sources.

- *For a country having a Eurocode National Annex, the requirements are set out in the appropriate National Annex.*
- *For other countries, guidance can often be found in local standards. If specific local standards are unavailable, it might be possible to obtain advice from the local government or local academic institutes with a department of seismology or geology.*
- *Specialist institutes are able to provide peak ground accelerations in most parts of the world.*
- *Site-specific seismic studies can be carried out by a competent specialist.*

The response to the seismic action should be determined using the methods given in BS EN 1998-1, BS EN 1998-2 and BS EN 1998-5 or, where appropriate, ASCE 61-14 [N1].

NOTE 2 The design approach used in ASCE 61-14 [N1] is significantly different to that used in BS EN 1998. The designer needs to decide which approach to take and use it consistently.

27.3.2 Vertical accelerations

The vertical component of seismic accelerations should be taken into account where the conditions indicate that it will have significant impacts.

NOTE Vertical accelerations are particularly important when at least one of the following applies:

- *where the horizontal acceleration exceeds 0.30g;*
- *where there are rail-mounted cranes or tanks and the horizontal acceleration exceeds 0.15g;*
- *where relative movement of adjacent structures could cause impact, e.g. where the fundamental periods differ by a factor of 1.15 or where the vertical acceleration contributes to rotation and also the horizontal acceleration exceeds 0.20g;*
- *in continuous sea walls of sufficient length in which vertical bending is significant where the horizontal acceleration exceeds 0.25g;*
- *where the ratio of the spectral magnification for the vertical acceleration to that of the horizontal acceleration exceeds 2.5 and the horizontal acceleration exceeds 0.15g or the ratio of the spectral magnification for the vertical acceleration to that of the horizontal acceleration exceeds 2.0 when the horizontal acceleration exceeds 0.20g;*
- *where there are hazardous substances.*

27.3.3 Spatial variability of excitation

For structures with a structurally continuous deck with lengths greater than the dominant wavelength of the earthquake as determined in BS EN 1998-2:2005+A2, the excitation can vary over the length of the structure and can affect design in the longitudinal direction. The designer should take such spatial variability into account.

NOTE 1 BS EN 1998-2:2005+A2, Annex D gives guidance on the spatial variability of earthquake ground motion, the model and appropriate methods of analysis.

NOTE 2 The UK National Annex to BS EN 1998-2:2005 states that spatial variability of ground motion need not be considered for bridges with continuous decks where the supports are founded on approximately uniform soils of types A, B or C.

NOTE 3 The UK National Annex to BS EN 1998-2:2005 states that spatial variability of excitation needs to be taken into account for a continuous deck for:

- *soil type D if the length exceeds 200 m;*
- *soil type E of the length exceeds 333 m.*

NOTE 4 Variation in the excitation along a wall does not normally affect the design of the wall cross-section and is unlikely to cause torsions in long thin structures. It has a significant effect on the design of transverse joints.

Simplified methods for taking into account spatial variation are given in BS EN 1998-2:2005+A2, **3.3**(4) to (7) and should be used where appropriate in maritime structures.

27.4 Seismic design approach and detailing

The design and detailing of the maritime structures should be chosen taking into account the opportunities for mitigating the effects of seismic actions, by introducing ductility into the design where possible.

NOTE It is expected that future parts or revisions of BS 6349 will include guidance on such design and detailing.

28 Berthing actions

*COMMENTARY ON CLAUSE **28***

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 stiffness and energy absorption capacity of fendering and berthing structure.

28.1 General

Fendering systems should be designed in accordance with BS 6349-4:2014, taking into account:

- the operational factors for the location and facility;
- the types of vessel and operations proposed;
- the form of the berth;
- the prevailing environmental conditions.

28.2 Operational factors

Berthing actions as the basis for structural design of marine facilities should take into account the operating philosophy established with the operator at the planning and design phases, and the DSOL in accordance with the recommendations in BS 6349-1-1:2013, Clause **22**.

Conditions of normal use for marine facilities appropriate to assessment of berthing actions in a persistent design situation should include normal operating conditions as defined in BS 6349-1-1:2013.

Berth design and planning for operations should in all cases seek to reduce risk of an accident (e.g. in layout, terminal and port operating procedures, and safety equipment such as berth monitoring systems, navigation aids, etc.) and to then to mitigate any residual risk to acceptable levels (by protective measures/systems, additional operational procedures). If, despite this approach, site and operationally specific risk assessment indicate a residual risk of collision, then credible accidental scenarios should be established, and accidental impact actions assessed. Such loads should then be taken into account as an accidental design situation.

28.3 Actions from fenders

Berthing actions from fendering systems during vessel berthing should be assessed in accordance with the recommendations of BS 6349-4:2014 for both characteristic and design berthing energy, based upon the fender type and properties, taking into account the effects of variation of fender mechanical properties and reaction characteristics with temperature, berthing velocity, angle of impact and manufacturing tolerances. The allowances for such effects should be established by reference to the fender manufacturer’s test or performance data and the prevailing conditions for the facility.

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 berthing and movement.

The design friction load acting parallel to the berthing face should be taken as μ times the fender reaction, where μ is the coefficient of friction between the two faces in contact. In the absence of a more detailed assessment, the frictional load should be assumed to act in the most adverse direction in the plane of the berthing face.

29 Mooring and breasting actions

29.1 General

Actions from moored vessels, both on mooring points and on fenders and breasting structures, should be assessed using one of the calculation methods described in BS 6349-4:2014, Clause **9** as appropriate to the vessel and berth type and the prevailing environmental conditions, taking into account the following.

- Method 1 (elastic static mooring analysis using a computer program) should be used to assess actions where wave or swell penetration or passing ship effects do not cause significant mooring line loads due to ship dynamic response.
- Methods 2 and 3, together with Method 1 with simplified hand calculation approach, may only be used to provide preliminary estimates of actions for vessels less than 20 000 t loaded displacement.
- Method 4 (fully dynamic numerical mooring analysis using a computer simulation program) should be used to assess actions where wave or swell penetration or passing ship effects cause significant mooring line loads due to ship dynamic response. Where necessary, numerical simulation should be supplemented by physical modelling.

*NOTE 1 The operational aspects of mooring are discussed in BS 6349-1-1:2013, Clause **20**, which gives guidance on methods for assessment of acceptable conditions for moored vessels including the use of numerical and physical models.*

NOTE 2 In the absence of other site-specific assessments for large tankers, bulk carriers and container ships, it is typically found that significant mooring line loads due to ship dynamic response are unlikely to occur:

- *in the following range of wave conditions (the range of wave height for each period depends upon ship type and wave direction):*
  - H_s *<0.50 to 1.0 m and peak periods <6 s;*
  - H_s *<0.25 to 0.50 m and peak periods <12 s;*
  - H_s *<0.10 to 0.20 m and peak periods >12 s; and/or*
- *in shipping channels for which the following conditions apply with respect to passing ships:*
  - *hull to hull separation distance of at least 4 times the passing ship's beam, at speeds of 6 knots or less; or*
  - *hull to hull separation distance of at least 2 times the passing ship's beam, at speeds of 4 knots or less.*

*NOTE 3 Recommendations for estimation of wind and current forces on moored vessels are given in **29.3**.*

29.2 Operational factors

Mooring actions as the basis for structural design of marine facilities should take into account the operating philosophy established with the operator at the planning and design phases, and the DSOL according to the recommendations in BS 6349-1-1:2013, Clause **22**.

Conditions of normal use for marine facilities appropriate to assessment of mooring actions in a persistent design situation should include normal operating conditions as defined in BS 6349-1-1:2013. Extreme operating conditions should also be taken into account if it is proposed that vessels are required to remain moored in such conditions.

NOTE 1 For many large ships, and large oil and gas carriers, bulk carriers and container ships, it is unlikely that the ship would stay at a berth under events of 50-year to 100-year return periods. The exception might be very protected harbours in regions with relatively benign environmental conditions. In most cases, however, the capacity of the mooring equipment on the ship would be insufficient for holding the vessels in extreme storms. In such cases the extreme operating condition would be for a vacant berth. In such cases it is often recommended to establish maximum credible sets of wind speed, wave height and current velocity consistent with the capacity of the mooring equipment of the design vessels, and to use this as a potential extreme operating situation for the occupied berth.

As recommended in BS 6349-4:2014, **9.3**, mooring equipment on the berth should be designed for a persistent design situation to have a safe working load (SWL) equal to or greater than the calculated loads.

Actions from mooring line overload, whether from operator error, mooring equipment malfunction, or collision with a moored vessel, should be treated as accidental design situations. Credible accidental design situations may be established by risk assessment for the specific operations proposed and the prevailing environmental conditions.

For quick release hooks, each hook should have a rated SWL of not less than the minimum breaking load (MBL) of the largest capacity line anticipated to be used for mooring of the design vessels, with the assumption (consistent with recommended mooring practice) that each hook will receive only a single mooring line.

For mooring bollards, each bollard should have a rated SWL of not less than the minimum breaking load (MBL) of the largest capacity line anticipated to be used for mooring of the design vessels. The designer should make an assessment of the number of lines likely to be connected to a single bollard.

The design mooring load on the mooring point structure should be assessed with respect to the likely joint probability of maximum line forces, or upon limiting loads from vessel's mooring equipment with appropriate partial factors as indicated in **29.4**.

NOTE 2 The requirement for the mooring equipment SWL to exceed individual line MBL is based upon the practice that, in the event of accidental overload of moorings, mooring system components need to fail progressively such than the breaking load of the mooring line is less than the rated SWL of the mooring equipment (hook, etc.); its fixings to the mooring point structure; and the design strength and stability of the structure itself.

*NOTE 3 In the design of any failure device pursuant to BS 6349-2:2010, **9.5** (such as break off foundation bolts) incorporated into a mooring point to limit the maximum load on mooring point, attention is also drawn to the related provisions of BS 6349-4:2014 regarding the mode and type of mooring failure so that risks to personnel, the vessel and quay structure are minimized.*

*NOTE 4 There is an increasingly large variety of synthetic mooring line materials available to ship operators and furthermore there can be a lack of available data on lines for new types or classes of ships (e.g. very large ore carriers). Design based upon rated MBL of mooring lines requires high confidence in the definition of line characteristics at the design stage, which in turn emphasises the importance of obtaining reliable data from ship owners, designers or operators as set out in BS 6349-1-1:2013, Clause **18**.*

29.3 Evaluation of wind and current forces

If a detailed assessment of mooring loads as recommended in BS 6349-4:2014 is not possible, then bollards for vessels up to 20 000 t loaded displacement should be provided along a continuous quays at intervals of 0.15 times the L_{OA} of the smallest most likely vessel to use the quay. The load capacity should be as given in Table 7, which allows for more than one rope to be attached to each bollard.

Table 7 **Nominal bollard loadings for vessels up to 20 000 t displacement**

Vessel loaded displacement t	**Bollard loading** kN
Up to 2 000	100
Up to 10 000	300
Up to 20 000	600

For vessels larger than 20 000 t loaded displacement, specific calculations should be carried out to determine the probable maximum mooring loads, taking into account:

- the number, patterns, characteristics and pre-tensions of the mooring lines;
- prevailing environmental conditions;
- proposed operations and DSOL for environmental conditions for the moored vessel.

For sheltered harbour environments where wave or swell penetration or passing ship effects do not cause significant ship response, mooring loads may be estimated by static mooring analysis based on the prevailing wind and current conditions using Method 1 described in BS 6349-4:2014 (see **29.1**).

Wind speeds for the evaluation of wind forces acting indirectly on structures from moored vessels should be assessed in accordance with BS 6349-1-1:2013, taking into account the size and response period of the vessel as noted in BS 6349-1-1:2013, **7.2**.

Current velocities for the evaluation of current forces acting indirectly on structures from moored vessels should be assessed in accordance with BS 6349-1-1:2013, using the depth-averaged current over the draught of the moored vessel based upon the current-velocity depth profile for the location as noted in BS 6349-1-1:2013, Clause **9**.

In the absence of other data, for large ships and where wave and current loading are expected to be significant, wind and current forces should be established by the testing of scale models.

*NOTE 1 The method of calculation of wind and current forces based upon the charts and empirical formulae included in Annex G may be used as a guide to the magnitude of wind and current forces on ships for concept design purposes. Subclause **G.4** also contains further guidance on wind speed averaging periods and spectra appropriate to vessel response.*

For VLCCs and other oil and product tankers down to 16 000 DWT, and for gas carriers in the range 75 000 m^3 to 125 000 m^3, wind and current drag coefficients should be assessed based upon the guidance contained in OCIMF MEG 3 [N2], Appendix A.

Comprehensive details and characteristics of the proposed design ships should be obtained as set out in BS 6349-1-1:2013, Clause **18** and used for detailed designs.

NOTE 2 Typical values for the lengths, draughts and lateral areas of bulk carriers and container vessels are given in BS 6349-1-1:2013 and in Annex G. These figures can be taken as guides for preliminary design in the absence of specific vessel data.

NOTE 3 Recommendations for the evaluation of wind, wave and current forces from moored wall-sided box shaped floating structures, such as pontoons, are expected to be included in the revision of BS 6349-6, which is currently in preparation.

NOTE 4 Wind and current forces vary considerably, depending on both type and size of vessel. In particular, the wind forces upon container vessels and other high-sided ships are influenced greatly by the particular design of different ships and the extent of cargo loaded on the deck. Very large tankers show marked variations in longitudinal force depending upon the design of the bow.

NOTE 5 Attention is drawn to the different empirical formulations of wind and current forces as explained in Annex G. Designers need to ensure that drag coefficients used are consistent with the formulation adopted.

29.4 Actions on mooring and breasting structures

As noted in **29.2**, mooring structures for large ships with multiple mooring points should be designed based upon the capacity of the mooring systems used to moor vessels to the structures. The overall mooring point capacity should be sufficiently robust to accommodate the dynamic nature of environmental response of vessels, including when under the influence of passing ships.

Credible accidental actions can arise from human error, individual mooring line failure, equipment malfunction or other such circumstances. The objective of the berth design and planning for operations should in all cases be to minimize the risk of an accident (e.g. by appropriate design of layout, terminal and port operating procedures, and by providing safety equipment such as berth monitoring systems and navigation aids) and then to mitigate any residual risk to acceptable levels (e.g. by the provision of protective measures/systems, additional operational procedures or contingency plans).

In the case of mooring structures designed to support more than one item of mooring equipment, and especially for large ships such as oil and gas tankers, container ships and bulk carriers, designers should assess the maximum credible total load on mooring points based on:

- risk assessment based on the specific operations proposed and the prevailing environmental conditions;
- capacity of the mooring equipment on the vessel;
- whether the mooring loads represent persistent or accidental design situations.

Table 8, to be read in conjunction with Table 1, shows different methods of assessment of loads on mooring structures for large ships and the corresponding values of the partial factor for actions, γ_Q, that should be adopted, taking into account the operating conditions and prevailing environment. All operating and accidental conditions should be assessed.

NOTE 1 For assessment of credible maximum mooring point loads for vessels moored using quick release hooks in exposed environments (when mooring line forces due to dynamic ship response are significant, e.g. due to wave exposure or passing ship effects), a practical approach to the maximum force on the supporting mooring point for the accidental operating condition that has been proposed for two, three and four hook assemblies is shown in Table 9.

NOTE 2 Annex H gives additional background guidance on assessment of design situations and loads, and in particular information on the selection of appropriate partial load factors in this clause.

Table 8 **Basis of loads on mooring structures with partial factors for actions**

Operating condition	Method of assessment of loads	Partial factor for actions, γ_Q EQU,Set A and STR/GEO Set B	Partial factor for actions, γ_Q STR/GEO Set C
Normal operating condition (berth occupied, and cargo handling and other operations ongoing)	Probable maximum calculated loads for specified normal environmental conditions (in combination with breasting loads and/or other consistent operating loads where applicable)	1.5	1.3
Extreme operating condition (berth occupied)	Probable maximum calculated loads for specified extreme operating environmental conditions (in combination with breasting loads where applicable)	1.5	1.3
Extreme operating condition (berth occupied)	Extreme operating condition – load is based on rated capacity [A] of vessel's mooring equipment when this is known at the design stage and when appropriate to operating requirements	1.3	1.15
Accidental condition (berth occupied)	Assessment of the maximum credible total load on mooring points for accidental scenarios based on: • risk assessment for specific operations proposed and the prevailing environmental conditions; and • the limiting capacity [B] of the vessel's mooring equipment and lines used to moor the vessel and berth mooring equipment multiplied by not less than 1.18	—	—

[A] For example, when the future berth operator prescribes oil and gas carriers conforming to OCIMF MEG 3 [N2] the rated capacity of mooring equipment and fittings is usually defined as the SWL which typically incorporates a safety margin over the yield. For winches, the brake holding load is set to render at 60% of MBL of the vessel lines and the rated capacity would be the design holding load equivalent to 80% of MBL of the vessel lines.

[B] The limiting capacity of a vessel's mooring equipment and mooring lines may be taken as the MBL of the mooring lines. The limiting capacity of the berth mooring equipment may be taken as the SWL. The factor of 1.18 allows for typical margins between nameplate or design limiting capacities of lines, winches and fittings and upper yield strength.

Table 9 **Accidental loads for multiple hooks**

Number of mooring hooks per mooring point	Total accidental mooring point load as multiple of rated hook SWL (or rated MBL of vessel's mooring line, where appropriate)
2	2.1
3	2.8
4	3.5

30 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, and should therefore be taken into account by the designer.

Although the total static vertical load is limited to the docking displacement of the vessel, when determining the application and distribution of that load, the designer should take account of the operational criteria and the relative strengths and stiffnesses of both the structure and the expected vessels.

NOTE Further guidance on the selection of design loadings for these types of structure is given in BS 6349-3.

31 Cargo storage

31.1 General

The requirements for cargo storage should be provided by the port or terminal operator or in consultation with the port or terminal operator.

NOTE Cargo may be stored in the open or within sheltered structures such as silos, tanks or sheds.

Any procedures and instructions established by the operator to define procedures, environmental operating limits and other such matters to ensure safe and efficient operation of the maritime works and facilities in the operation and maintenance phase should be listed in the facility operating manual as described in BS 6349-1-1:2013.

In all cases, the persistent and transient actions on the sub-structure should be calculated taking into account the actions from the storage structure, the material stored, the cargo handling equipment 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.

31.2 Dry bulk stacks

For open stacks of bulk materials, the actions from the stacked material should be calculated based 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.

NOTE 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 I.

31.3 Other commodities

For other storage areas, the actions imposed should be calculated based on the height of stacking and effective density of the commodities as packaged, including space between stacks.

NOTE 1 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.

NOTE 2 In the absence of more specific information, the typical values of stacking height given in Table 10 may be adopted for concept design. Typical values of effective stacked densities for some common commodities are given in Annex I. If better information is not available, the loading from general cargo can be taken as 30 kN/m² in operational areas or 50 kN/m² in stacking areas.

Table 10 **Typical stacking heights**

Cargo type	**Open storage stacking height** m	**Sheltered storage stacking height** m
General palletized cargo	2	3 to 5
Timber or timber products	3	6 to 7
Metal products	2	3
Fish	2	2.5
Vegetables and fruit	2	4

31.4 Containers

*COMMENTARY ON **31.4***

Containers are supported by corner castings that are 178 mm × 162 mm in plan dimension and which transmit the load from the container and any containers stacked above to the ground. These corner castings give highly concentrated loads that have to be taken locally by the pavement. When designing for global loads, e.g. in settlement calculations, it is appropriate to treat the loads from containers as an uniformly distributed load. Containers are usually stacked in separate areas for empty and full containers, and the height and density of the stacks depends on the stacking equipment.

Typically average container gross weights are less than half their maximum gross weight per twenty-foot equivalent unit (TEU). It is unlikely that containers will be stacked by weight and hence it is unlikely that all of the containers within a stack will be the maximum weight. The designer should establish from the container terminal operator, if possible, the distribution of loaded containers to establish statistically the local (corner casting) and global actions.

If statistical data is not available for concentrated loads then Table 18 of the Interpave document *The structural design of heavy duty pavements for ports and other industries* [28] may be used. This table should be corrected for an increase in maximum container weights since its publication.

If statistical data is not available for global loading, the distribution of loads with stacking height given in Table 18 of the Interpave document *The structural design of heavy duty pavements for ports and other industries* [28] may be used based on a gross weight of 320 kN for a 20 ft. full container. The calculation of the uniformly distributed load should allow for the stacking density that varies between stacks formed using different types of stacking equipment.

31.5 Other loads

An allowance of an additional transient dynamic load, equal to the maximum unit load handled, should be made for setting down impacts where cranes operate.

When storing commodities that are above or below ambient temperatures, the effect of temperature on the ground or structure should be taken into account.

Storage areas for dangerous and/or leaking cargoes should allow for containment or other protective measures.

32 Cargo handling and transport systems

32.1 General

*COMMENTARY ON **32.1***

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 port vehicles operating on their tyres within the confines of the port, with or without lifting capacity;

f) rubber-tyred port vehicles operating on their supporting pads or legs within the confines of the port, with or without lifting capacity;

g) tracked cranes.

Any procedures and instructions established by the operator to define the cargo handling and transport systems, including operating procedures, environmental operating limits and other such matters to ensure safe and efficient operation of the maritime works and facilities in the operation and maintenance phase, should be listed in the facility operating manual as described in BS 6349-1-1:2013.

The actions imposed on structures should be taken into account in both vertical and horizontal directions.

When designing the superstructure in working areas, the effects of collision impacts should be taken into account.

The persistent design situation should take account of the fact that the operations of cranes are usually halted at high wind speeds, and while a crane is handling cargo, the wind speed acting on the crane can be limited accordingly. All environmental limits should be provided by the equipment designer together with the applicable actions. For maximum wind conditions for the transient design condition, account should be taken of any special measures for stowage of the crane.

32.2 Fixed and rail-mounted equipment

For fixed and rail-mounted cargo handling equipment, actions should be calculated for the equipment to be installed for the permanent and imposed loads. Both vertical and horizontal actions should be taken into account. Imposed loads 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 treated as an accidental design situation and should be calculated using a relative speed at impact of 1.0 m/s.

32.3 Ship to shore container cranes

Because the size and loads from container cranes are likely to increase within the working design life of the structures, allowance for increasing loads from ship to shore container cranes should be made by the designer in consultation with the terminal operator and owner.

The design of structures supporting container cranes should include the actions from container cranes including self-weight, operational loads including dynamic loads, environmental loads including wind, and jacking loads for erecting and maintenance.

The combination load cases should include for operational design situations when the wind speed is limited and non-operational design situations that include the design wind speed. Wind directions should include from the front, rear, side and diagonally from each direction.

NOTE 1 Corners of container cranes can be subject to uplift in non-operational conditions. In the case of uplift, cranes are fitted with holding down links that are linked into connectors cast into the supporting structure.

NOTE 2 Brakes on container cranes are not usually designed to prevent the movement of the cranes along the crane rails under non-operational wind loads. This motion is usually prevented by pins attached to the cranes that engage into connectors cast into the supporting structure. The connectors transfer the horizontal wind loads acting along the direction of the rail into the supporting structure.

NOTE 3 It is usual to fix the locations where the jacking of cranes takes place by providing jacking plates on the surface of the supporting structure.

NOTE 4 Based on recent increases in crane sizes, a load factor of 1.5 can be used to allow for possible future increases in equipment specification over those current at the publication date of this standard. This load factor does not include the partial factors used in the design.

NOTE 5 Figure 2 gives typical dimensions for rail-mounted ship to shore container cranes current at the publication date of this part of BS 6349.

NOTE 6 Typically under the in-service condition the maximum loads are applied by the seaward two legs. In the out-of-service condition and under storm conditions, the maximum loads are typically applied by the landward legs.

NOTE 7 Wheel loads can be limited by increasing the number of wheels in each bogie subject to any restrictions on the overall dimension between buffer faces.

32.4 Conveyors, pipelines and loading arms/hoses

Actions from conveyors, pipelines and loading arms/hoses should be calculated for each installation, taking account of permanent loads and factors that apply to the imposed loads, including material densities, rates of flow, changes of direction, temperature effects and the spacing and nature of the support framework.

32.5 Rail traffic

Actions from rail traffic in ports should be taken from BS EN 1991-2:2003.

Within ports, rail traffic might be within segregated or non-segregated routes and within rail sidings, and the following factors should be taken into account.

a) Train speeds are restricted when compared to main line installations.

b) Train vehicle movements might have to take place in mixed working environments where non-railway activity is taking place. The design situations should therefore allow for mixed use.

c) Where rail is segregated there might be a higher number of crossing points with roadways and highways. Internal port crossings are unlikely to have full barrier protection, which results in low speeds.

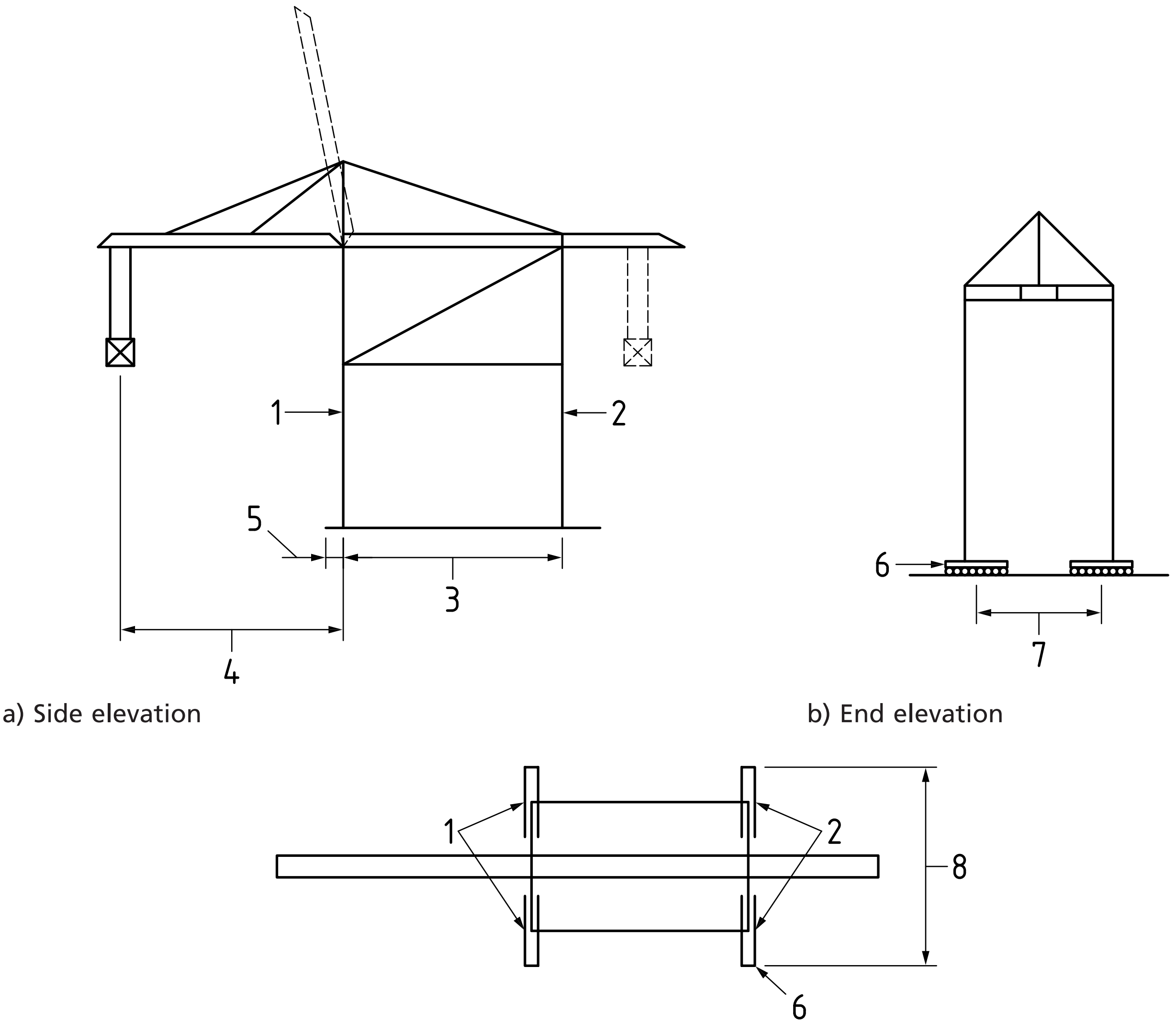
d) Curve radii might be tighter than on main lines owing to space constraints within port sites, leading to lower speeds and greater risk of noise generation. This might increase dynamic effects and lateral loads.

e) For constrained layouts the breaking up of trains in reception areas might be required. This might require additional equipment to shunt wagons into place, increasing the duration of operations and possibly leading to increased standing static loads in rail yards and more frequent dynamic effects and lateral loads.

f) Rail wagons provide dynamic effects from setting down impacts.

g) Accidental design situations might include derailment damage, which might impact the surface infrastructure that is used for other activity when trains are not present.

Figure 2 **Typical container crane dimensions**



a) Side elevation

b) End elevation

c) Plan

Key

1 Front portal leg

2 Rear portal leg

3 Rail gauge (typically 15 m to 35 m)

4 Outreach from front rail (typically 28 m to 72 m)

5 Front rail to vessel (typically 5 m to 7 m)

6 Multiple wheel bogie (six to ten wheels, typically eight wheels)

7 Distance between centre of the wheel groups (typically 14.5 m to 17 m)

8 Distance between buffers (typically 27 m)

32.6 Road traffic

Loads from road traffic in ports should be taken from BS EN 1991-2:2003.

Rubber-tyred port road vehicles can impose considerably higher loads or local load intensities than highway traffic. Load model 1 (LM1) from BS EN 1991-2:2003 should be factored accordingly.

For non-typical road traffic such as specific port equipment with high axle loads, load model 2 (LM2) from BS EN 1991-2:2003 should be modified to give a representative value for the adjustment factor B_Q.

For non-typical road vehicles such as specific port equipment with evenly distributed axle loads, load model 3 (LM3) from BS EN 1991-2:2003 should be used where possible by selecting a suitable class of special vehicle to match the vehicle's width, total weight, number of axles, axle spacing and axle load.

For other port equipment that does not conform to the load models in BS EN 1991-2:2003, the designer should model the loads given in the equipment's specification. Outside the UK, heavier traffic loads might be permitted or encountered and account should be taken of local conditions.

32.7 Rubber-tyred port vehicles

32.7.1 General

Where specific equipment is used in the port, loads should be established from the manufacturer's data. If a class of equipment is proposed, the designer should research potential equipment from a range of manufacturers and establish the most likely loads from the manufacturers' data.

NOTE For concept designs, and where data is not available, the values of equivalent uniformly distributed loading given in Table 11 may be used for various common port transport systems.

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

Vehicle	**Payload capacity** t	**Maximum laden mass** t	**Equivalent uniformly distributed load** kN/m²
Fork lift trucks	3	7 to 9	12
	5	10 to 15	15
	10	25 to 40	20
	20	45 to 55	25
	25	65 to 85	30
	40	110 to 120	40
Straddle carriers (for containers)	30	50	12
	40	90	15
	50	120	25
Tractor/roll trailer systems	20	—	10
	40	—	15
	80	—	20

32.7.2 Fork lift truck loading

Where a range of dimensions of the potential equipment is established by the designer, the value in the range that gives the most severe case for the relevant structural element should be adopted.

NOTE 1 For concept designs, and where data is not available, Table 12 may be used for nominal loads for various ranges of fork lift trucks.

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 12, except where the capacity is less than 5 t, in which case solid rubber tyres might be used and the contact area may be assumed to be a rectangle. The length of the rectangle parallel to the axle may be taken as 150 mm.

NOTE 2 For heavy fork lift trucks it might be feasible to reduce the load intensities by increasing the number of wheels per axle from four to six.

Table 12 **Fork lift truck wheel loading: container handling duties**

Payload	Length of container	Maximum front axle loads	No. of wheels on front axle
t	ft.	kN	
28	20 (part full)	700	4
32	20	740	4
42	40	940	4

32.7.3 Reach stackers

*COMMENTARY ON **32.7.3***

Reach stackers have very high front axle loads when lifting, moving with a load and setting down. Bespoke cargo reach stackers have been developed to handle containers, logs, paper rolls, steel, pipes, intermodal wagons and other heavy loads. They have been used in rail terminals to load rail wagons, utilizing their ability to load/unload onto the adjacent and second rails. They have also been used to load small barges.

Pavements that might be loaded by reach stackers should be designed for the very high axle loads that will exceed the typical capacity of general port paving.

In container terminals, loading from reach stackers should be allowed for within blocks of containers that would normally be handled by other terminal equipment such as rubber-tyred gantry cranes.

NOTE 1 For concept designs, and where data is not available, the typical reach stacker loads given in Table 13 may be used.

Table 13 **Reach stacker axle loads**

Lift capacity	Self-weight	Laden front axle load
t	t	kN
10	40	400
20 to 25	60	600
40	70	900 to 1 000
45	80 to 110	1 000 to 1 200

NOTE 2 Reach stackers usually have four wheels on the front axle and two wheels on the rear axle.

32.7.4 Side loaders

NOTE Side loaders are less commonly used than fork lift trucks and reach stackers, especially for heavy loads.

Side loaders impose wheel, outrigger or jack loadings and provision should be made for these loads.

32.7.5 Straddle carriers

*COMMENTARY ON **32.7.5***

Stacking straddle carriers are usually used to lift, transport and stack containers up to three high. Transfer straddle carriers are also used to lift, transport and put down containers and are particularly used in automated rail-mounted gantry crane terminals.

Straddle carriers typically do not fall into load model 3 (LM3) loading under BS EN 1991-2:2003, and hence the designer should design pavements and decks using specific wheel loads.

NOTE In the absence of specific information, for stacking straddle carrier container operations, laden carriers may be considered as equivalent to:

a) 50 t to 60 t self-weight with six wheels arranged in two parallel lines, each imposing 165 kN; or

b) 60 t to 72 t self-weight with eight wheels arranged in two parallel lines, each imposing 150 kN.

For transfer straddle carrier container operations, laden carriers may be assumed to be equivalent to 50 t self-weight with six wheels, each imposing 145 kN, arranged in two parallel lines.

For special purpose straddle carriers, information on wheel loads should be obtained for the particular machines.

32.7.6 Mobile cranes

Load model 3 (LM3) or load model 1 (LM1) from BS EN 1991-2:2003 should be used when mobile cranes are moving between operations. Provision should also be made for the outrigger reactions and bearing pressures that might be imposed, relative to the maximum size of crane expected.

NOTE 1 For concept designs, and where data is not available, the typical mobile harbour crane loads given in Table 14 may be used.

NOTE 2 Mobile cranes are rated according to their load moment capacity or their maximum lift capacity at short radius.

The reactions on the structure should be taken as acting on two outriggers simultaneously at the outrigger spacing given in Table 14. The other outrigger loads may be calculated as sharing the sum of the maximum lift plus the machine self-weight, less the outrigger loads already calculated.

NOTE 3 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/m² can develop unless restrictions are imposed.

Table 14 **Mobile crane outrigger reactions**

Maximum lift capacity range	**Self-weight of crane**	**Outrigger spacing**	**Maximum static outrigger reaction**
t	t	m	kN
up to 40	100	9.0	800
40 to 60	200	10.0	1 350
60 to 80	300	11.0	2 000
80 to 125	450	12.5	2 900
150 to 200	600	14.0	4 000
200 to 300	800	15.0	5 500

32.7.7 Roll trailer loading

*COMMENTARY ON **32.7.7***

Roll trailers are made up of a combination of the tractor and trailer. Container terminals might also have specialized container carriers, which might be automated.

In the absence of data from specific roll trailers, the plan dimensions shown in Figure 3 and the loads shown in Table 15 for various capacity trailers of mass up to 80 t may be used

Tractor wheel loads should be assumed to be uniformly distributed over a circular or square area with an effective pressure of 700 kN/m².

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.

Figure 3 **Dimensions of roll trailer vehicle**

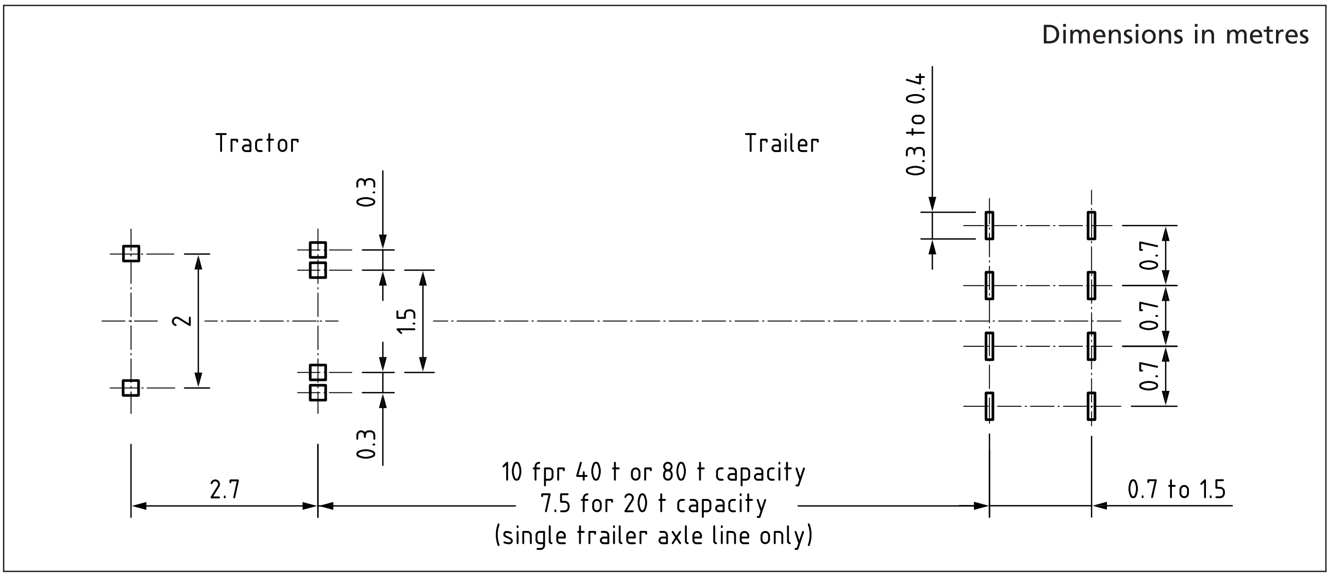


Table 15 **Roll trailer loading: axle loads and effective wheel pressures**

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	N/mm²
20	70	115	1	150	1
40	100	280	2	150	1
80	110	350	2	290	1

32.7.8 Rubber-tyred gantry cranes

*COMMENTARY ON **32.7.8***

Rubber-tyred gantry cranes are typically used in container and rail terminals. They normally have eight or sixteen wheels depending on the requirements for the terminal layouts and whether they are supported by specific structures or on the general pavement.

Typically rubber-tyred gantry cranes travel unladen and can move between operating locations.

Large gantry cranes for container handling can impose individual wheel loads of up to 400 kN for eight-wheel cranes and 200 kN for sixteen-wheel cranes with contact pressures of 950 kN/m².

Rubber-tyred gantry cranes might be electrically powered and/or automated, and in each case this might increase the self-weight of the cranes.

The designer should take into account self-weight, dynamic loading and horizontal loading, including wind loads and [C1) braking actions (C1].

Owing to the wide range of equipment available, details should be obtained from the terminal or port operator of the particular equipment and associated wheel load calculations, taking into account the load capacity of the cranes, their span and lift height. Where rubber-tyred gantry cranes change direction, account should be taken of the actions that the wheels have on the pavement due to rotation.

32.7.9 Tracked cranes

*COMMENTARY ON **32.7.9***

Tracked cranes are not regularly used in terminal and ports as they can cause local damage to pavements and decks.

If cranes have been specified to be used within a terminal then the track widths might be increased to reduce track pressure.

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.

If tracked cranes are to be used then the terminal or port operator should be advised that mats should be used to protect the surface of asphalt concrete pavements. Other types of surfacing might also be at risk of damage and if appropriate the surfacing should be protected. This should be included in the facility operation manual described in BS 6349-1-1:2013.

NOTE For concept designs, and where data is not available, the values in Table 16 may be used.

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/m²	kN/m²
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	60	3.0	4.5	0.90	78	300

33 Channelized loading in pavements and decks

In assessing the effect of vehicular loading on pavements and decks, allowance should be made for the effects of channelization.

The designer should calculate the number of vehicle movements along each vehicle route for the design working life of the pavement or deck. This calculation should allow for an increase in port productivity, which should be discussed with the terminal or port operator.

NOTE 1 Where information from a terminal or port operator is not available then an allowance of 50% on the number of vehicle movements may be made for a 20 year period.

In situations where vehicles are restricted to narrow traffic lanes and repeatedly travel along the same path, the number of design vehicle movements should be increased by a factor of three to allow for increased repetitions and damage.

In more severe cases where there is no room for vehicle wander, e.g. straddle carriers travelling through container stacks and auto-guided vehicles which precisely follow the same wheel paths, the number of design vehicle movements should be increased by a factor of five.

NOTE 2 Damage can take the form of structural failure and/or surface rutting. Channelization affects the choice of surfacing materials.

The designer should take into account the potential for failure caused by severe surface rutting.

NOTE 3 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, for the following reasons.

a) The axle loads involved can be considerably greater than the range of loads for which the conversion has been established. This can lead to a significant underestimate of design loads.

b) Slow moving heavy vehicles performing turning manoeuvres are considerably more damaging than loads imposed by road-legal vehicles moving at higher speeds.

c) The spacing of the wheels and the contact pressures imposed can differ significantly from those associated with highway traffic.

d) 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.

Wheel loads should be ascertained for the specific equipment. The pavement designer should take into account unladen vehicle movements and laden vehicle movements transporting a design critical load.

NOTE 4 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 may be taken to be in units equal to the heaviest unit load. Alternatively, a more precise spectrum may be used if sufficient information on traffic patterns is available.

34 Movements, dynamic response and vibrations

*COMMENTARY ON CLAUSE **34***

Many maritime structures are significantly different to 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.

Maritime structures are frequently classified into two general groups: rigid and flexible. Rigid structures are those that resist horizontal loading by mainly direct compression and/or tension. This group includes filled earth structures such as quay walls and structures incorporating opposing raking 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 or from wave effects are likely to be significant. Flexible structures have larger deflections under impulsive loading and, as a consequence, the effect is reduced.

Recommendations regarding the acceptability of movements and vibrations for specific types of maritime works or structures are provided in other parts of BS 6349.

34.1 Assessment of movements and vibrations

Movement and vibrations of maritime structures should be assessed taking into account the following types of effects causing response and actions:

a) cyclic effects;

b) impulsive effects;

c) random effects;

d) static and long-term cyclic effects.

Actions, dynamic and static response should be assessed using an approach which is appropriate to the response and structure type.

Apart from static and long-term cyclic effects, hydraulic and operational effects causing actions on maritime structures are dynamic in character, and therefore action and movements should be assessed taking into account the magnitude of the effect, its variation with time, and the response of the structure to that particular effect.

*NOTE 1 Recommendations in terms of dynamic response to environmental effects are given in **18.2** and **24.3**.*

NOTE 2 Annex F provides an approximate method of assessment of the displacement of both flexible and raked pile rigid structures. This method can be used for concept design purposes and is generally likely to be conservative, especially for flexible structures in open sea or in water more than 30 m deep.

Where rotating machinery is supported, data on expected frequency and energy levels should be obtained from the manufacturer and the effect on the structure should be assessed.

NOTE 3 Heavy rotating machinery can have some effect on structural elements if significant amounts of energy are present at the natural frequencies of the structure. In most cases, rotating machinery gives rise to variations at 25 Hz and 50 Hz.

34.2 Impulsive loads

The potential effects of impulsive loads should be assessed, including:

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.

NOTE Impulsive loads cause the displacement to rise to a maximum and thereafter to decay cyclically about the original position at rest. Guidance on the design of fenders from berthing forces is given in BS 6349-4:2014.

34.3 Static and long-term cyclic loads

Certain cyclic loads have such long periods that they act on the structure as static loads. As a minimum, the following loads of a static or quasi-static nature should be taken into account:

a) dead load of structure;

b) earth pressure;

c) superimposed live load;

d) current loading;

e) tidal change loads;

f) time-averaged wind loading.

NOTE Normal methods of static analysis can be used to calculate the movements resulting from static or quasi-static loads.

34.4 Expansion and contraction

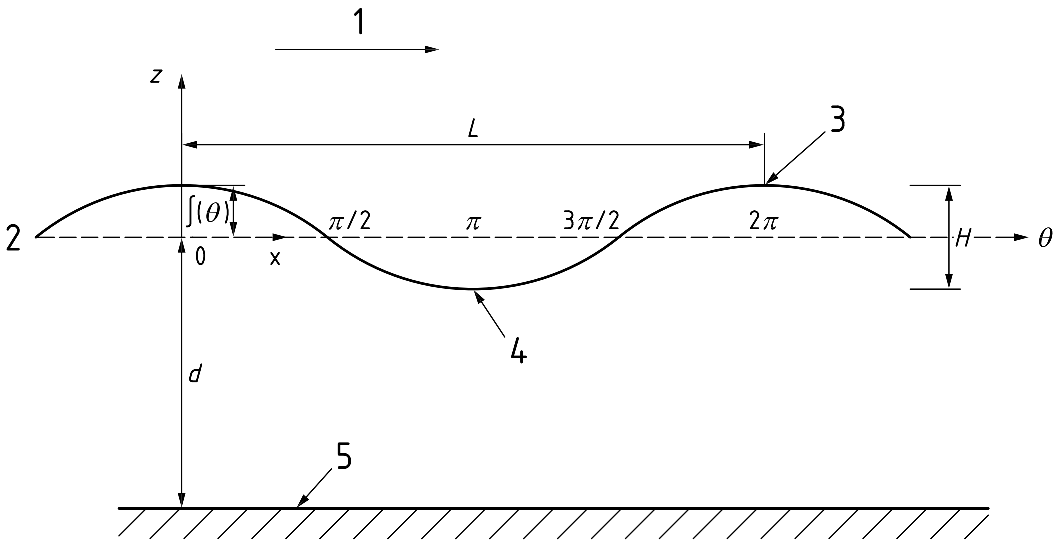
Maritime structures should allow for expansion and contraction movements of the structure, which occur both as short-term daily movements and as long-term annual movements.

Annex A (informative) Linear ("first order" or "sinusoidal") wave theory

The assumptions and limitations of linear wave theory can be found in numerous textbooks. Some of the key wave characteristics described by linear wave theory are given in Figure A.1 and Table A.1.

In linear wave theory the period is assumed to be independent of water depth (assumed constant with varying water depth, wave height and direction).

Figure A.1 **Linear wave theory – definition diagram**



$\theta = kx - \omega t$

$\omega = \dfrac{2\pi}{T}$

$k = \dfrac{2\pi}{L}$

where:

ω is the angular frequency

k is the wave number

Key

1	Direction of propagation	*H*	Wave height
2	Still water level (SWL) (z = 0)	*L*	Wave length
3	Crest	*d*	water depth
4	Trough	*t*	Time
5	Bottom (z = $-d$)	*T*	Wave period

Table A.1 **Linear wave theory – useful equations** *(1 of 2)*

Estimation of:	**Appropriate formulae**		
	Shallow water, as defined by $\frac{d}{L} < \frac{1}{20}$	**Transitional water, as defined by** $\frac{1}{20} < \frac{d}{L} < \frac{1}{2}$	**Deep water, as defined by** $\frac{d}{L} > \frac{1}{2}$
Wave profile	$\eta = \frac{H}{2}\cos\left[\frac{2\pi x}{L} - \frac{2\pi t}{T}\right] = \frac{H}{2}\cos\theta$		
Wave velocity	$v_c = \frac{L}{T} = \sqrt{gd}$	$v_c = \frac{L}{T} = \frac{gT}{2\pi}\tanh\left(\frac{2\pi d}{L}\right)$	$v_c = v_{co} = \frac{L}{T} = \frac{gT}{2\pi}$
Wavelength	$L = T\sqrt{gd} = v_c T$	$L = \frac{gT^2}{2\pi}\tanh\left(\frac{2\pi d}{L}\right)$	$L = L_o = \frac{gT^2}{2\pi} = v_{co}T$
Group velocity	$v_{cg} = v_c = \sqrt{gd}$	$v_{cg} = nv_c = \frac{1}{2}\left[1 + \frac{4\pi d/L}{\sinh(4\pi d/L)}\right]v_c$	$v_{cgo} = \frac{1}{2}v_{co} = \frac{gT}{4\pi}$
Water particle velocity			
Horizontal	$u = \frac{H}{2}\sqrt{\frac{g}{d}}\cos\theta$	$u = \frac{H}{2}\frac{gT}{L}\frac{\cosh[2\pi(z+d)/L]}{\cosh(2\pi d/L)}\cos\theta$	$u = \frac{\pi H}{T}\exp\left(\frac{2\pi z}{L}\right)\cos\theta$
Vertical	$v = \frac{H\pi}{T}\left(1 + \frac{z}{d}\right)\sin\theta$	$v = \frac{H}{2}\frac{gT}{L}\frac{\sinh[2\pi(z+d)/L]}{\cosh(2\pi d/L)}\sin\theta$	$v = \frac{\pi H}{T}\exp\left(\frac{2\pi z}{L}\right)\sin\theta$

Table A.1 **Linear wave theory – useful equations** *(2 of 2)*

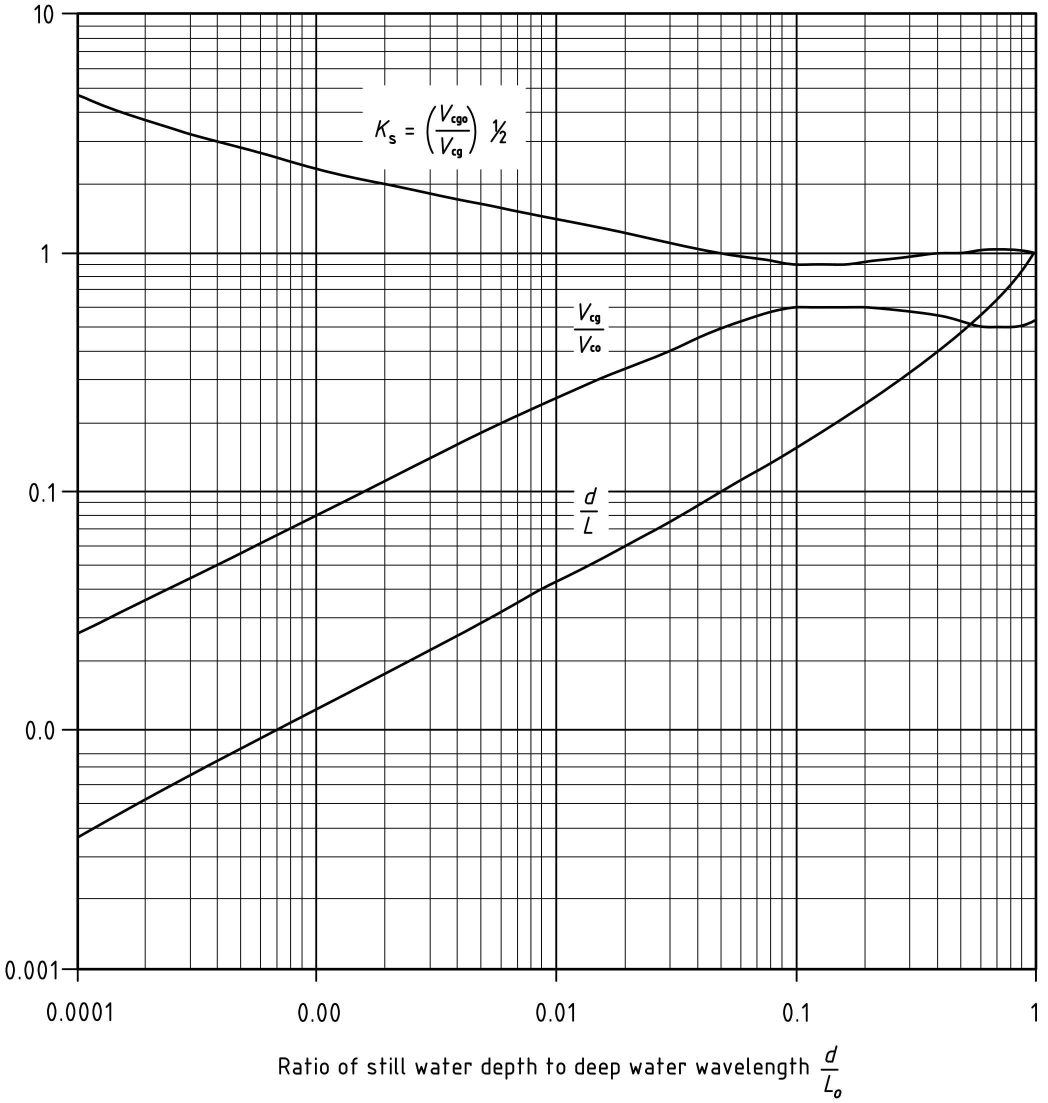
Estimation of:	**Appropriate formulae**		
	Shallow water, as defined by $\frac{d}{L} < \frac{1}{20}$	**Transitional water, as defined by** $\frac{1}{20} < \frac{d}{L} < \frac{1}{2}$	**Deep water, as defined by** $\frac{d}{L} > \frac{1}{2}$
Water particle accelerations			
Horizontal	$\dot{u} = \frac{H\pi}{T}\sqrt{\frac{g}{d}}\sin\theta$	$\dot{u} = \frac{g\pi H}{L}\frac{cosh\left[2\pi(z+d)/L\right]}{cosh(2\pi d/L)}\sin\theta$	$\dot{u} = 2H\left(\frac{\pi}{T}\right)^2 \exp\left(\frac{2\pi z}{L}\right)\sin\theta$
Vertical	$\dot{v} = -2H\left(\frac{\pi}{\tau}\right)^2\left(1+\frac{z}{d}\right)\cos\theta$ $\dot{v} = -2H\left(\frac{\pi}{\zeta}\right)^2\left(1+\frac{z}{d}\right)\cos\theta$	$\dot{v} = -\frac{g\pi H}{L}\frac{sinh\left[2\pi(z+d)/L\right]}{cosh(2\pi d/L)}\cos\theta$	$\dot{v} = -2H\left(\frac{\pi}{T}\right)^2 \exp\left(\frac{2\pi z}{L}\right)\cos\theta$
Water particle displacements			
Horizontal	$\zeta_h = -\frac{HT}{4\pi}\sqrt{\frac{g}{d}}\sin\theta$	$\zeta_h = -\frac{H}{2}\frac{\cosh\left[2\pi(z+d)/L\right]}{sinh(2\pi d/L)}\sin\theta$	$\zeta_h = -\frac{H}{2}\exp\left(\frac{2\pi z}{L}\right)\sin\theta$
Vertical	$\zeta_v = \frac{H}{2}\left(1+\frac{z}{d}\right)\cos\theta$	$\zeta_v = \frac{H}{2}\frac{\sinh\left[2\pi(z+d)/L\right]}{sinh(2\pi d/L)}\cos\theta$	$\zeta_v = \frac{H}{2}\exp\left(\frac{2\pi z}{L}\right)\cos\theta$
Subsurface pressure	$p = \rho g(\eta - z)$	$p = \rho g\eta\frac{cosh\left[2\pi(z+d)/L\right]}{cosh(2\pi d/L)} - \rho gz$	$p = \rho g\eta exp\left(\frac{2\pi z}{L}\right) - \rho gz$

Annex B (informative) Nearshore wave processes

B.1 General

Figure B.1 provides graphs for the estimation of wave shoaling and wave height in the surf zone based on uniform seabed slopes.

Figure B.1 **Wave shoaling and estimation of wave height in the surf zone** *(1 of 5)*



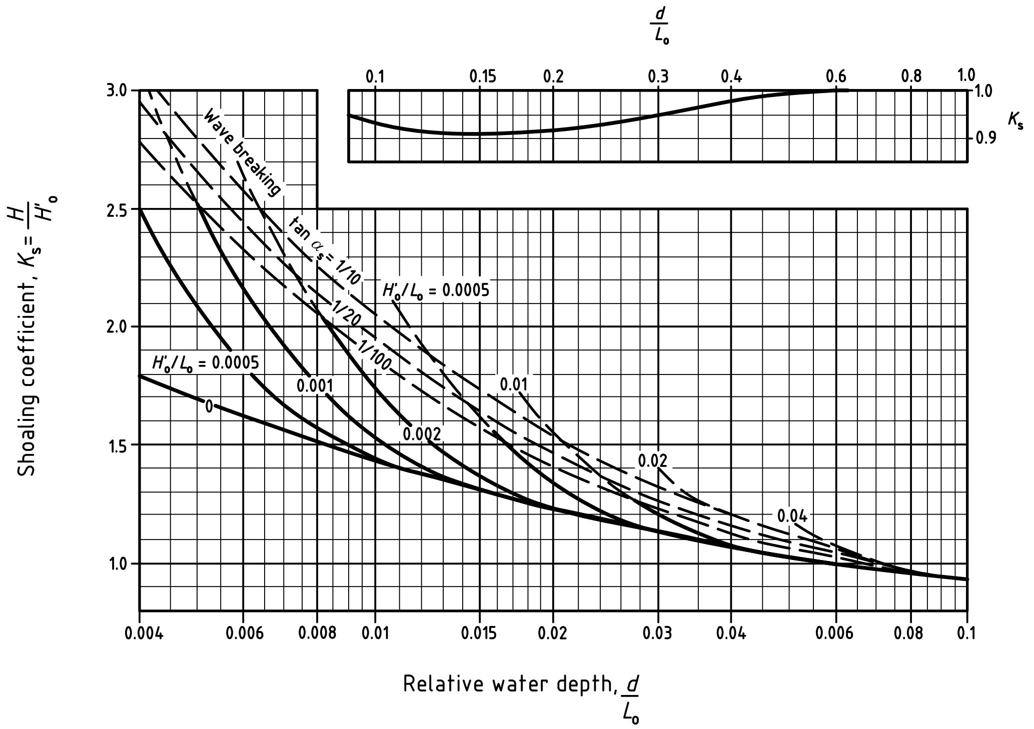
Ratio of still water depth to deep water wave length $\frac{d}{L_o}$

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

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

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

Figure B.1 **Wave shoaling and estimation of wave height in the surf zone** *(2 of 5)*



b) Diagram of non-linear wave shoaling

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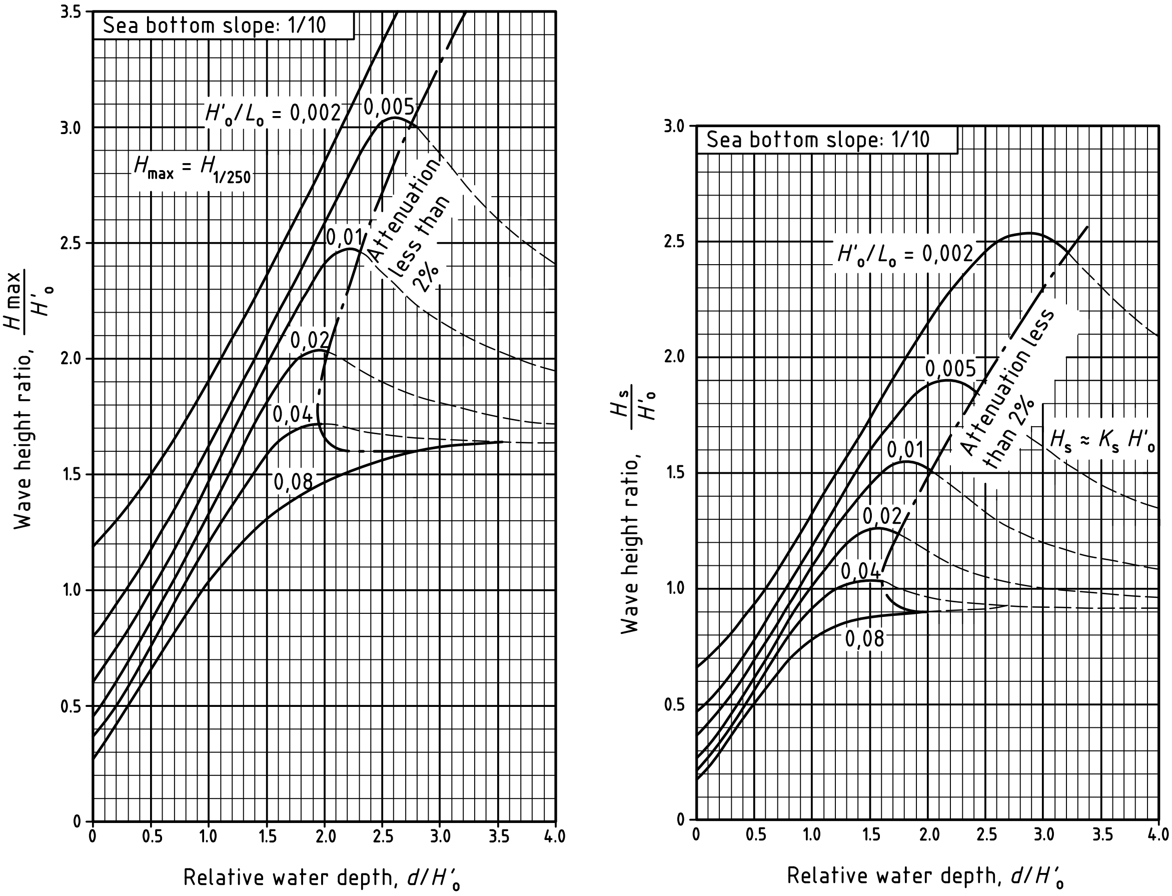

c) Diagrams for the estimation of wave height in the surf zone

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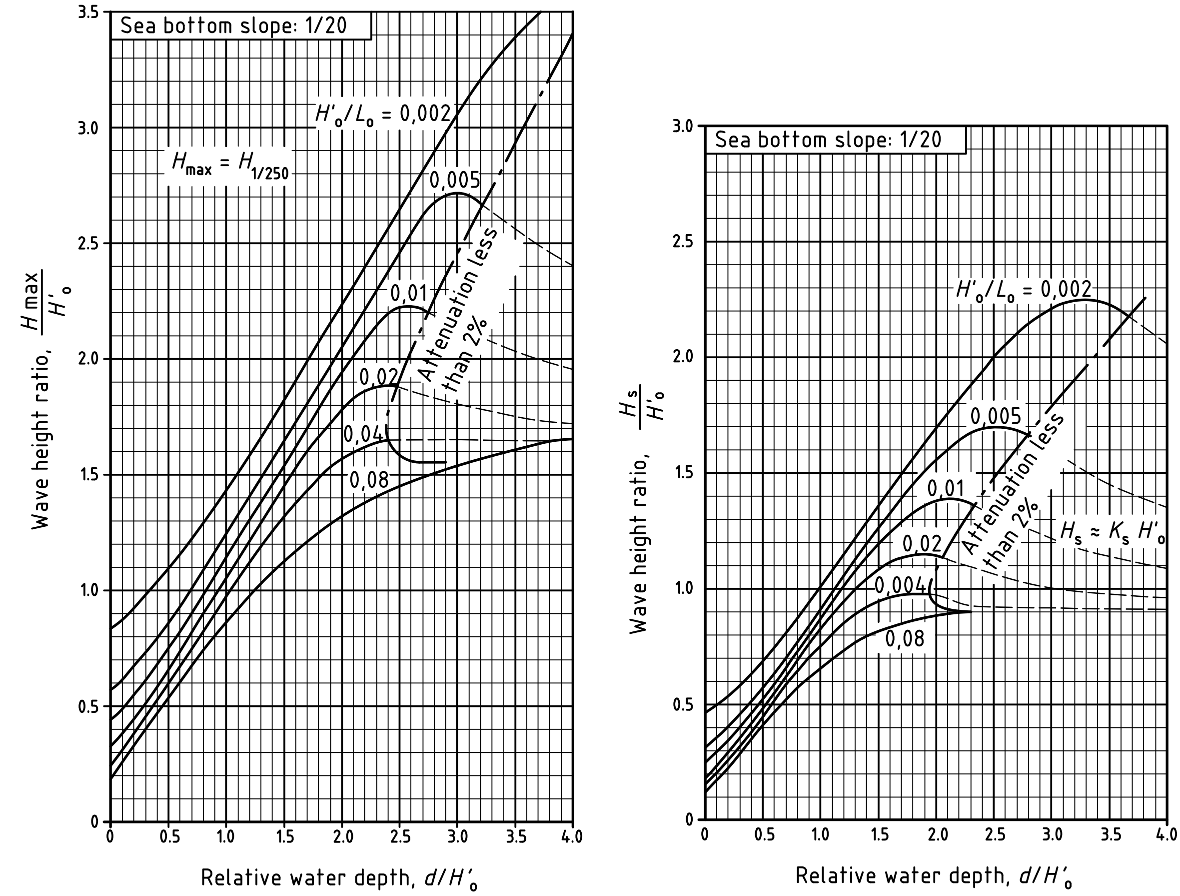
Figure B.1 **Wave shoaling and estimation of wave height in the surf zone** *(3 of 5)*



d) Diagrams for the estimation of wave heights in the surf zone (sea bottom slope of 1/20)
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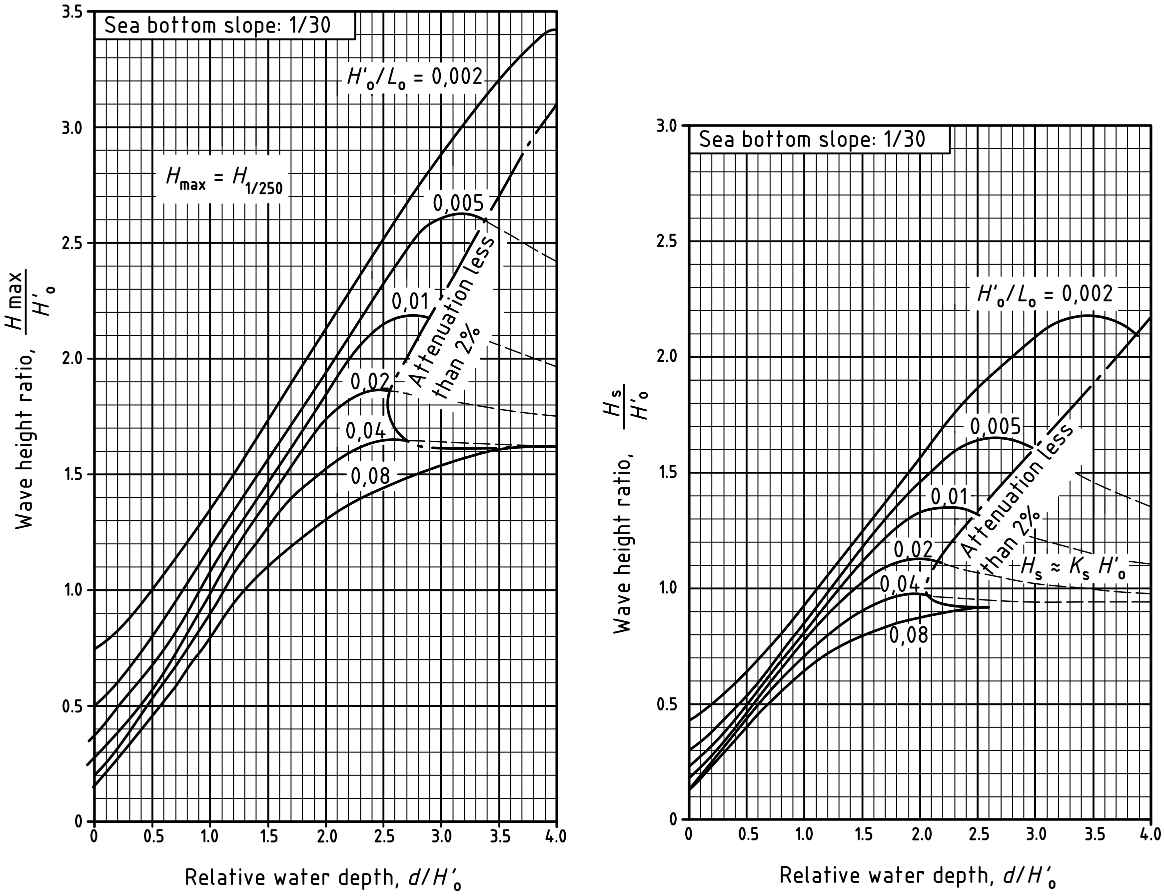
Figure B.1 **Wave shoaling and estimation of wave height in the surf zone** *(4 of 5)*



e) Diagrams for the estimation of wave height in the surf zone (sea bottom slope of 1/30)
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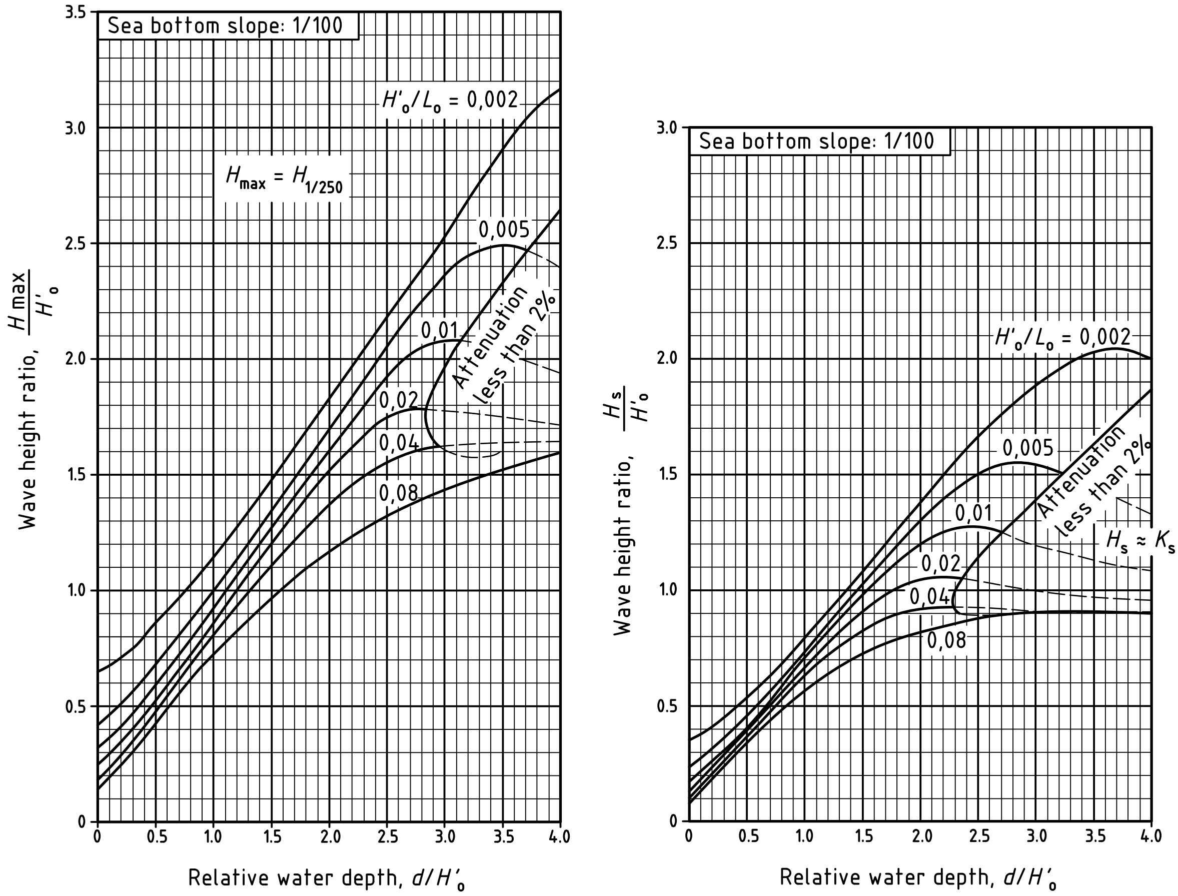
Figure B.1 **Wave shoaling and estimation of wave height in the surf zone** *(5 of 5)*



f) Diagrams for the estimation of wave heights in the surf zone (sea bottom slope of 1/100)
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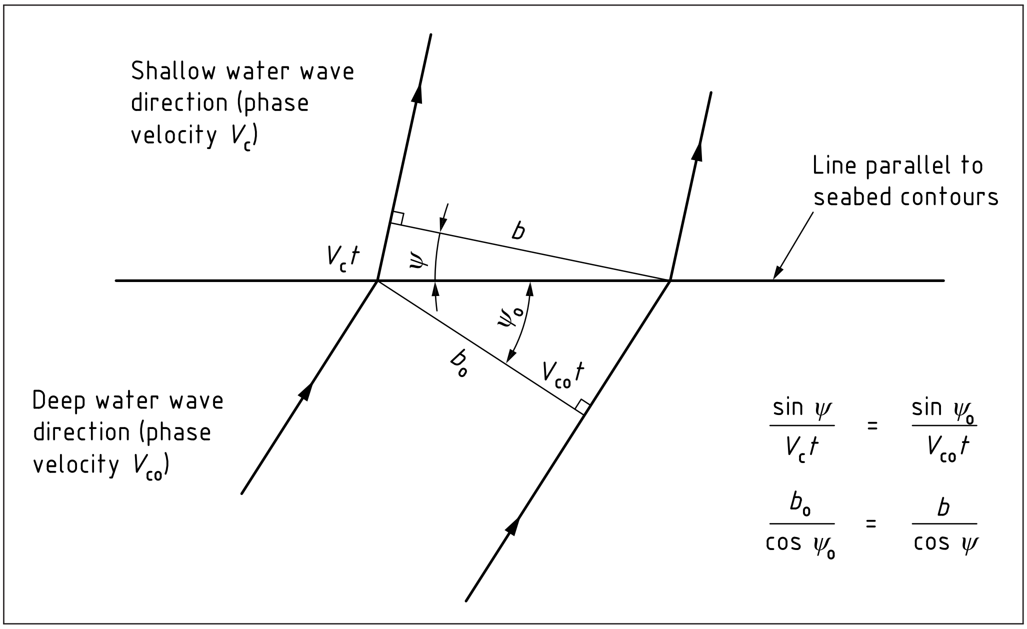
B.2 Refraction and shoaling

Wave crests tend to align themselves parallel to the bed contours because of refraction.

A schematic diagram of refraction is shown in Figure B.2, which is representative of situations where the bed profile can be approximated to straight parallel contours. In most practical situations, the bed contours are irregular or curved in plan and a numerical wave model approach is recommended. Where numerical modelling is not practicable and for an indicative assessment only, 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 being assessed. The divergence or convergence of adjacent rays indicates a concentration or dispersion of the wave energy along the wave crest.

Figure B.2 **Schematic diagram of wave refraction**



In the absence of currents and assuming no wave generation, dissipation, diffraction or reflection, the local wave height resulting from both shoaling and refraction may be related to the deep water value by assuming that wave energy is conserved between orthogonals, such that $Ec_g b$ remains constant, where:

E is the wave energy density, given by $\rho g H^2/8$

where:

ρ is the density of water, in kilograms per cubic metre (kg/m³);

g is acceleration due to gravity, in metres per second squared (m/s²);

H is the wave height, in metres (m);

c_g is the wave group velocity, in metres per second (m/s);

b is the distance between adjacent wave orthogonals, in metres (m).

The inshore wave height can then be calculated from:

$$H = K_s K_r H_o$$

where:

H_o is the wave height in deep water, in metres (m);

K_s is the wave shoaling coefficient, given by $v(v_{cgo}/v_{cg})$;

where:

v is the vertical component of water particle velocity, in metres per second (m/s);

v_{cgo} is the wave group velocity in deep water, in metres per second (m/s);

v_{cg} is the wave group velocity, in metres per second (m/s);

K_r is the wave refraction coefficient, given by $v(b_0/b)$;

where:

b_0 is the wave ray separation in deep water.

Values of the shoaling coefficient, together with wave length and group velocity, can be obtained from Figure B.1a).

The shoaling effect upon random waves, including the effects of wave breaking, can be estimated from Figure B.1b).

Where simple ray-tracing techniques are adopted, the wave refraction coefficient can be found by measurement of the relative divergence or convergence of wave rays obtained by refraction analyses.

Abrupt changes in submarine contours can lead to wave diffraction and reflection, which causes energy transfer and thereby invalidates the previous expressions.

B.3 Bottom friction

As a result of friction with the seabed, waves 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, is:

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

where:

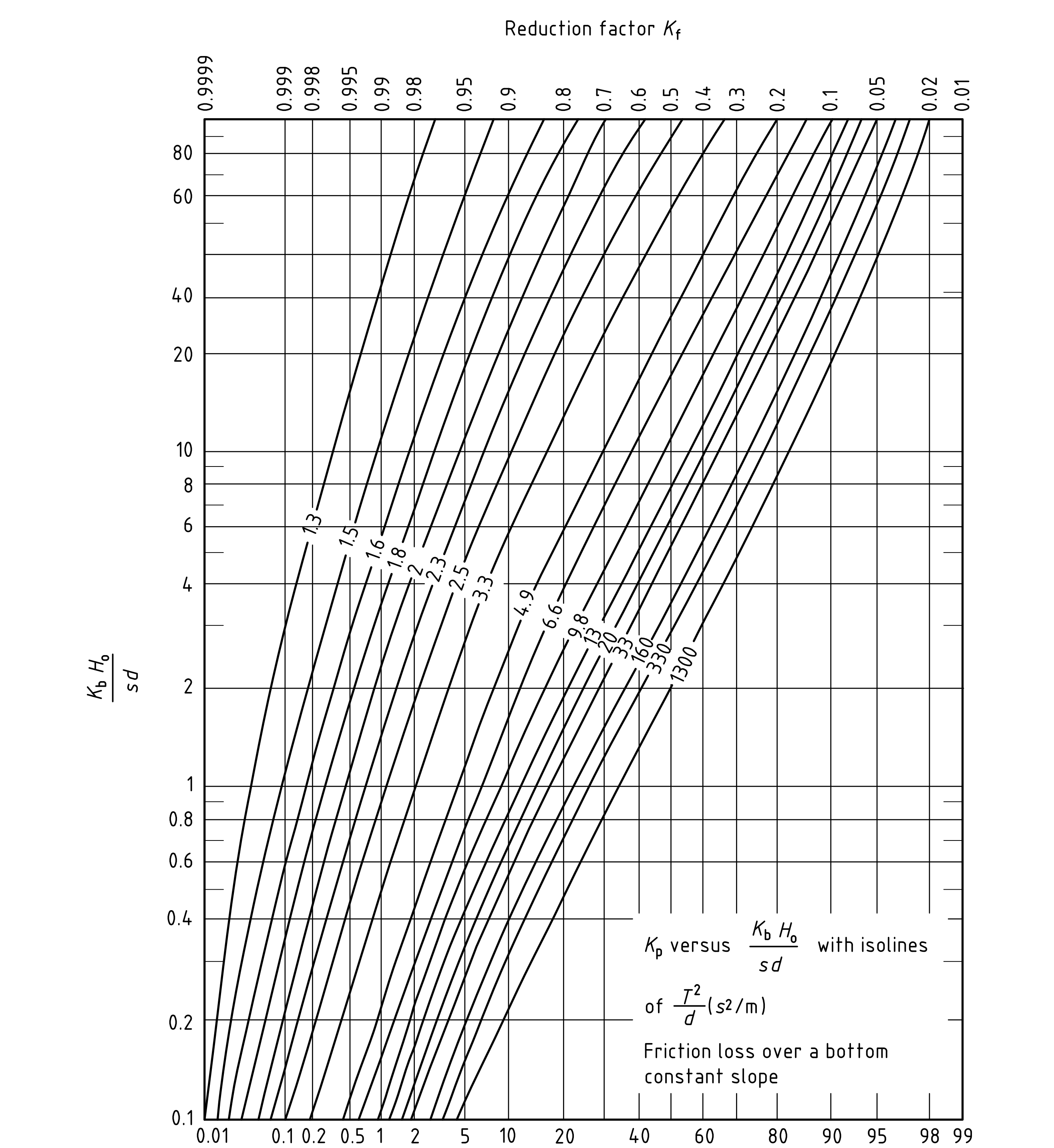
K_b is the bed friction factor;

ρ is the mass density of the water, in kilograms per cubic metre (kg/m³);

u is the horizontal orbital velocity at the seabed, in metres per second (m/s), calculated in the absence of bed friction.

The product of this force with u, averaged over a wave period, gives the amount of energy dissipated per unit area in unit time and this equals 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 B.3 for the special case of a seabed of constant slope, s_d, 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.

Figure B.3 **Wave height reduction factor for bottom friction**



It can be seen from Figure B.3 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 seawalls or beaches. 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 B.3 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 or otherwise rough seabed. 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. Studies of this kind 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.

B.4 Wave breaking

This subclause provides a brief overview of the complex subject of wave breaking and is provided for concept design purposes and ease of reference only. Further information can be found in CIRIA publication C683 [2]. In most cases it is recommended that numerical or physical models are employed where wave breaking is expected to be a dominating factor in the design. This is due to the fact that the available empirical methods tend to be applicable to very simple situations only, i.e. constant seabed slopes, perpendicular wave incidence only.

In deep water, waves break before reaching a limiting steepness (approximately H_o/L_o < 1 in 7). The following expression (see *Mouvements ondulatoires des mers EN profondeur constant ou décroissante* [29]) provides a simple means of estimating the limiting wave steepness as waves move into shoaling water:

$$\left(\frac{H}{L}\right)_b = 0.14 \tanh\left(\frac{2\pi h}{L}\right)_b$$

in which subscript b indicates breaking conditions. When $h/L < 0.04$ ($h/L_o < 0.01$) this expression gives $H_b \approx 0.88h_b$, indicating that in shallow water it is the depth of water alone which governs the breaking wave height. However, in practice, the breaker index H_b/h_b (= γ_{br}) depends on the Iribarren number, with spilling breakers having $\gamma_{br} \approx 0.7$~0.8, plunging breakers having $\gamma_{br} \approx 0.9$~1.1 and collapsing breakers having $\gamma_{br} \approx 1.2$~1.3. To account for these variations, the above equation may be modified to read as follows:

$$\left(\frac{H}{L}\right)_b = 0.14 \tanh\left(\frac{\gamma_{br}}{0.88} \times \frac{2\pi h}{L}\right)_b$$

In shallower depths, as well as depending on the initial wave energy, wave breaking is also governed by the water depth and the slope of the sea-bed and hence the rate at which the wave height changes as it propagates, with the depth-limited breaker index γ_{br} (=H/h) varying between approximately 0.5 and 1.5 as h reduces. The often quoted value of depth-limited breaker index being equal to 0.78 times the still water depth can be derived from the theory describing individual waves over a flat seabed, and is therefore not an adequate estimate of breaker height in most situations.

A rule of thumb for irregular waves on mild sloping foreshores (with constant bottom slope less than 1 in 50) gives a breaker index around 0.5 to 0.6.

Definitions of the various types of breaking wave are illustrated in Figure B.4. The definitions are based on the surf similarity parameter, ξ, or Iribarren number, I_r (see *Computation of set-up, longshore currents, run-up and overtopping due to wind generated waves* [30]), which is calculated as follows:

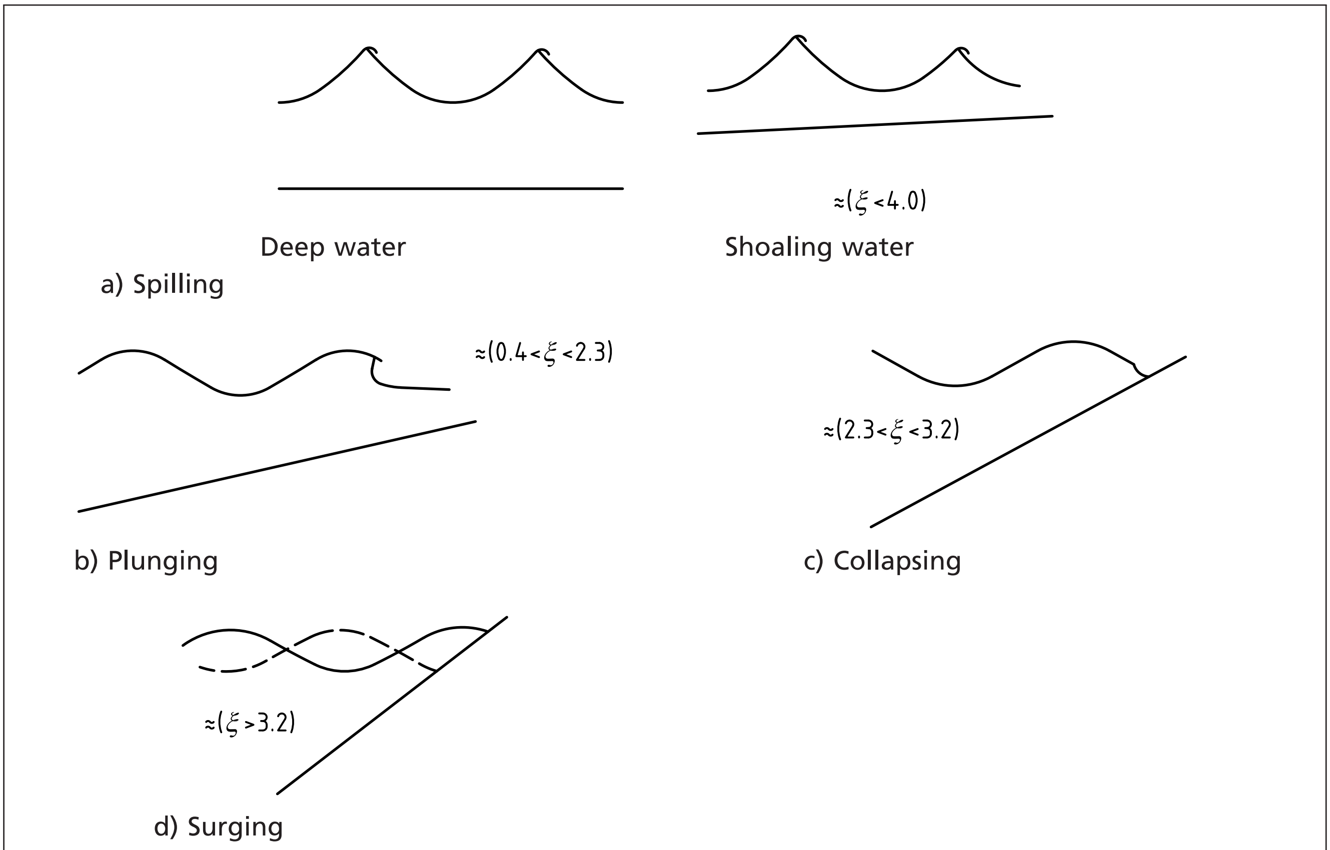
$$\xi = \frac{\tan\beta}{\left(H/L_o\right)^{0.5}}$$

where:

β	is the bed slope;
H	is the wave height, in metres (m) measured at the toe of the slope.

There are no precise boundaries between the different types of breaking wave. Factors such as the slope permeability also have an influence on the types of breaking.

Figure B.4 **Breaker types**



Spilling breakers can exist where the sea-bed has a gentle slope. Plunging waves have crests which curl over the wave's leading face and crash into the preceding trough, entrapping air and releasing much of the wave energy in a single violent impact. Collapsing waves are in the transition stage between plunging and surging. They never fully break, as in plunging, yet the bottom of their leading face collapses, producing foam. Collapsing waves lead to the greatest run-up. Surging waves do not break, potentially resulting in substantial reflection, although the actual degree of reflection depends upon the roughness and permeability of the slope.

A widely adopted method to account for shoaling and wave breaking on uniform foreshore slopes (and normal incidence to the slope) is given in *Random seas and design of maritime structures* [3]. The method is based on 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 steepness, H_o/L_o, the bottom slope and the relative water depth, h/L_o, 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. The maximum wave height is 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 $H_{1/250}$ = 1.8H_s.

Figure B.1c), Figure B.1d), Figure B.1e) and Figure B.1f) 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 B.1b).

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:

$$H'_o = K_d K_r H_{s0}$$

The period of the equivalent deep water wave is assumed to be equal to the deep water significant wave period as follows:

$$T_p = T_{p0}$$

Thus H_o, in general, varies and is different for each geographical position that is assessed.

The wave height can be estimated using the following equations.

If $h/L_o \geq 0.2$:

$$H_s = K_s H'_o$$

If $h/L_o < 0.2$:

H_s is the lowest of the following:

$H_s = \beta_0 H'_o + \beta_1 h$; or

$H_s = \beta_{max} H'_o$; or

$H_s = K_s H'_o$

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

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

β_{max} is the greater of the following:

β_{max} = 0.92;

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

$$\beta_0 * \ 0.052 \left(\frac{H'_o}{L_o}\right)^{-0.38} \exp\left[20 \tan^{1.5} \alpha_s\right]$$

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

β_{max}* is the greater of the following:

β_{max}* = 1.65;

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

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

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

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_o = 0.2. Caution is therefore required when applying these formulae and, where possible, comparison or validation with other methods is advisable.

B.5 Wave diffraction for a flat seabed

B.5.1 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;

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.

B.5.2 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.

Chapter Four of CIRIA publication C683 [2] includes a series of figures that can be used to make a preliminary estimate of the likely effects of wave diffraction.

In most cases it is expected that a numerical model approach will be taken to assess diffraction due to natural or man-made obstructions, in which case the designer needs to attain an understanding of the relative scientific merits of different theory-based models in terms of their reliability in representing the physical processes likely to occur.

B.5.3 Currents induced by wave diffraction

A secondary effect of wave diffraction at harbour entrances (or gaps between detached breakwaters for beach control) is the induction of currents in the lee of the breakwaters.

From the contours of the difference coefficient shown in diffraction figures in Chapter Four of CIRIA publication C683 [2] 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 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.

Annex C (informative) Wave prediction using charts

C.1 General

The methods of wave prediction described in this annex are for preliminary estimates only, to support concept designs where alternative sources of wave data (see Clause **11**) are not available. The methods provided here 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.

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.

C.2 Wind speed and duration

The wind speed to be used, unless otherwise stated, can be assumed to be the speed at 10 m above sea level, averaged over the relevant duration.

In the absence of direct measurements of wind speed at multiple stations and 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, can be used in preference to direct measurements of wind velocity.

Over smaller well-defined fetches, coastal measurements of wind speed can be used. In these situations it is usual to adjust the mean coastal wind speed to obtain the equivalent wind speed over the open sea. If coastal wind data is used in this way, then the designer can assess whether the wind speed and direction over the entire fetch is the same as those at the coastal station. The resulting forecast is likely to be unreliable in those situations where the fetch length typically exceeds half the radius of the cyclonic wind pattern.

C.3 Fetch length

The fetch used in wave forecasting techniques can 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 that this effect is small. The fetch length may therefore be assumed 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.

C.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.

Typical deep-water wave prediction curves, which correlate well with the results of spectral techniques over a wide range, are shown in Figure C.1 and Figure C.2. 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^2T^2 shown on Figure C.1 and Figure C.2 represent lines of constant energy. These can be used to obtain wave parameters from the cumulative effects of varying wind speed histories; however, care is required to check that the fetch limitations are not exceeded in such cases.

For example, for a fetch of 120 km, over which the wind speed averages 20 m/s from 1 000 h 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.

Figure C.1 **Significant wave prediction chart – Fetch lengths up to 1 500 km**

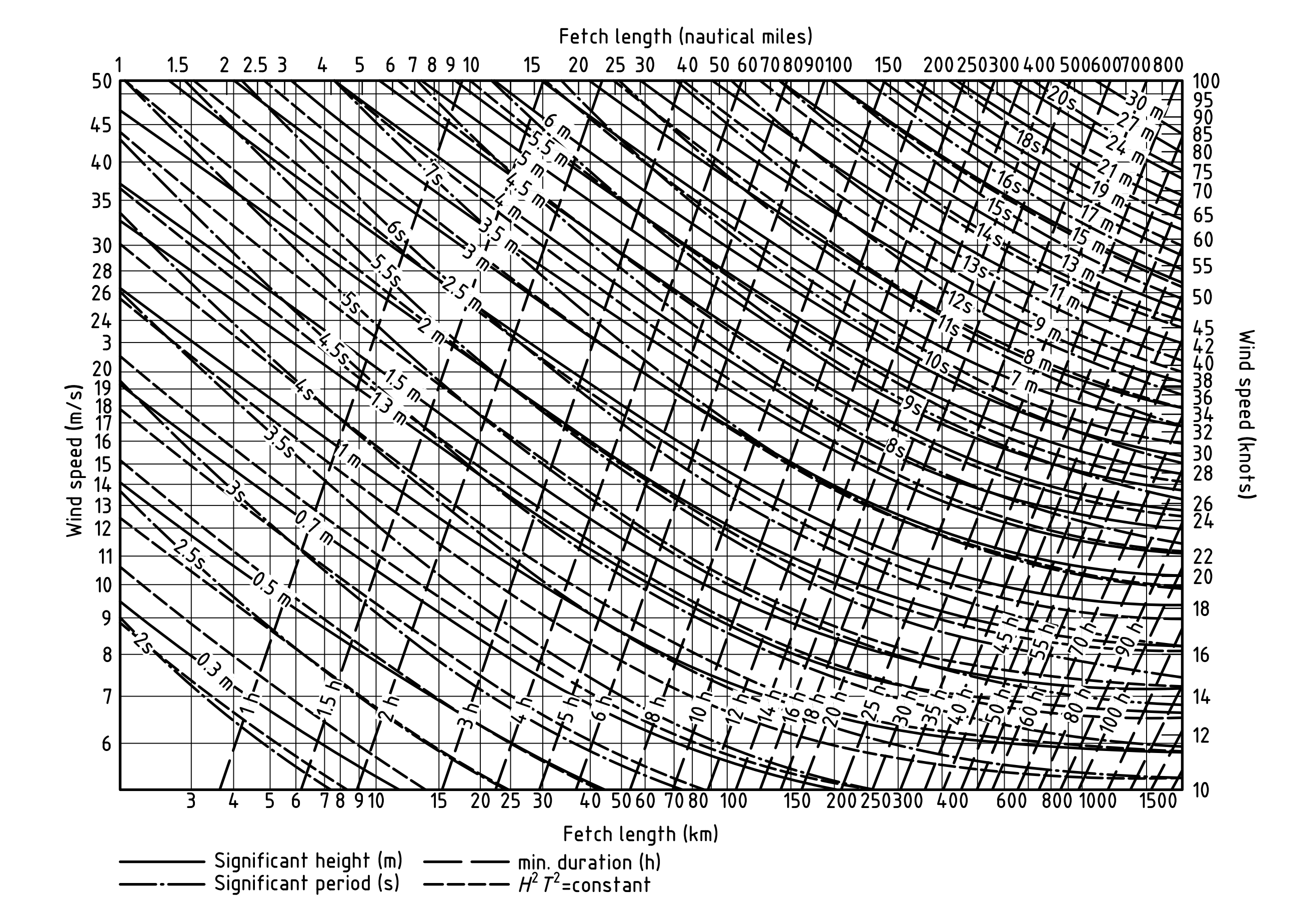
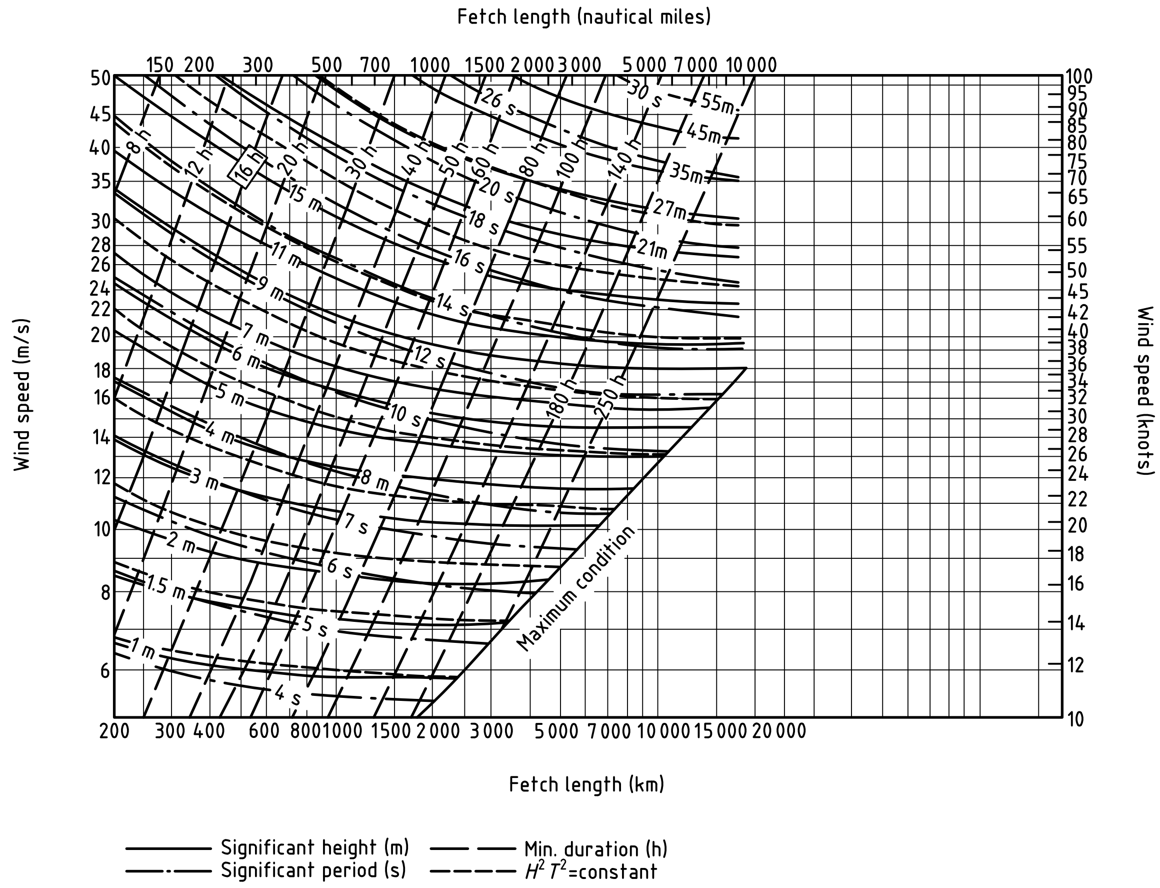


Figure C.2 **Significant wave prediction chart – Fetch lengths from 200 km to 20 000 km**



C.5 Prediction by wave spectra

Studies, particularly those 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 Figure C.3 and Figure C.4, in which the spectral density, *S*(*f*), is plotted against wave frequency, *f*.

Figure C.3 **JONSWAP wave spectrum**

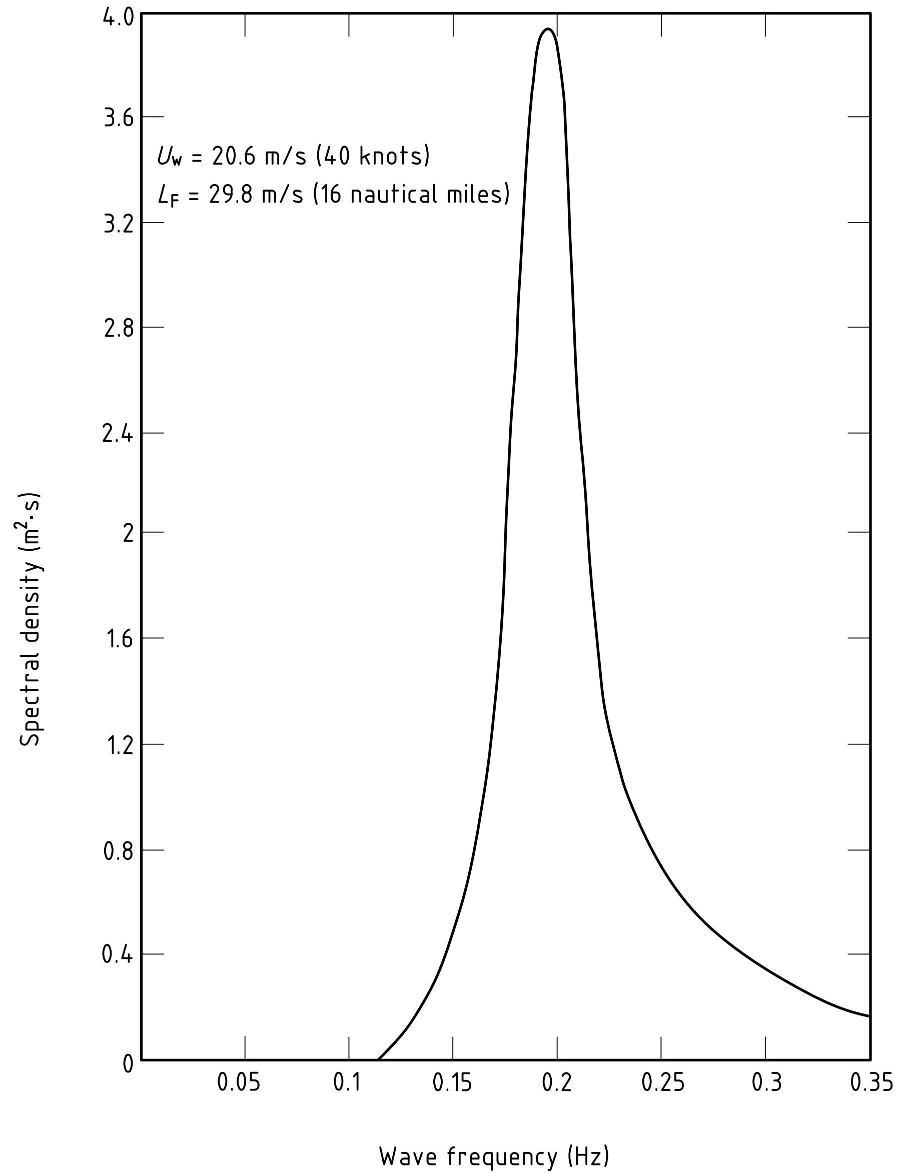
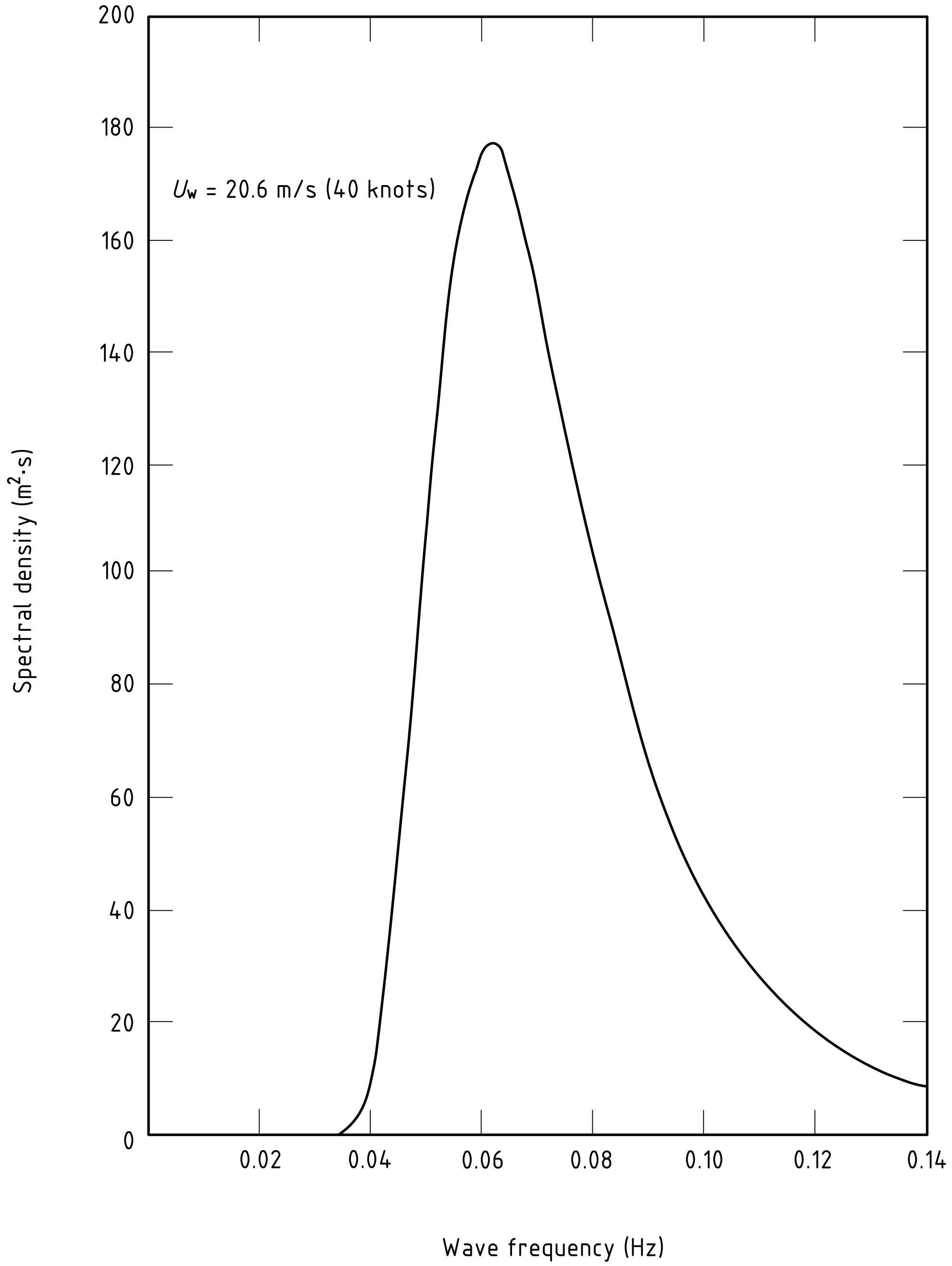


Figure C.4 **Pierson–Moskowitz wave spectrum**



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 spectral significant wave height, H_{m0}, is given by the relationship:

$$H_{m0} = 4\sqrt{m_0}$$

where the zeroth spectral moment, m_0, represents the area under the spectrum obtained by integration.

The analysis of empirical data for those situations where the waves are fetch-limited has resulted in the JONSWAP spectrum, in which the spectral density is given by:

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

where:

$$k_J = 0.076\left(\frac{gL_F}{U_w^2}\right)^{-0.22}$$

$\gamma = 3.3$

$$a = \exp\left[\frac{(f/f_p - 1)^2}{2\omega^2}\right]$$

where:

$\omega = 0.07$ for $f \le f_p$

or

$\omega = 0.09$ for $f > f_p$

$$f_p = 3.5\frac{g}{U_w}\left(\frac{gL_f}{U_w^2}\right)^{-0.33}$$

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_p is the frequency at which the peak occurs in the spectrum.

Figure C.3 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 C.5, 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:

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

where:

$k_P = 0.008\ 1$

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

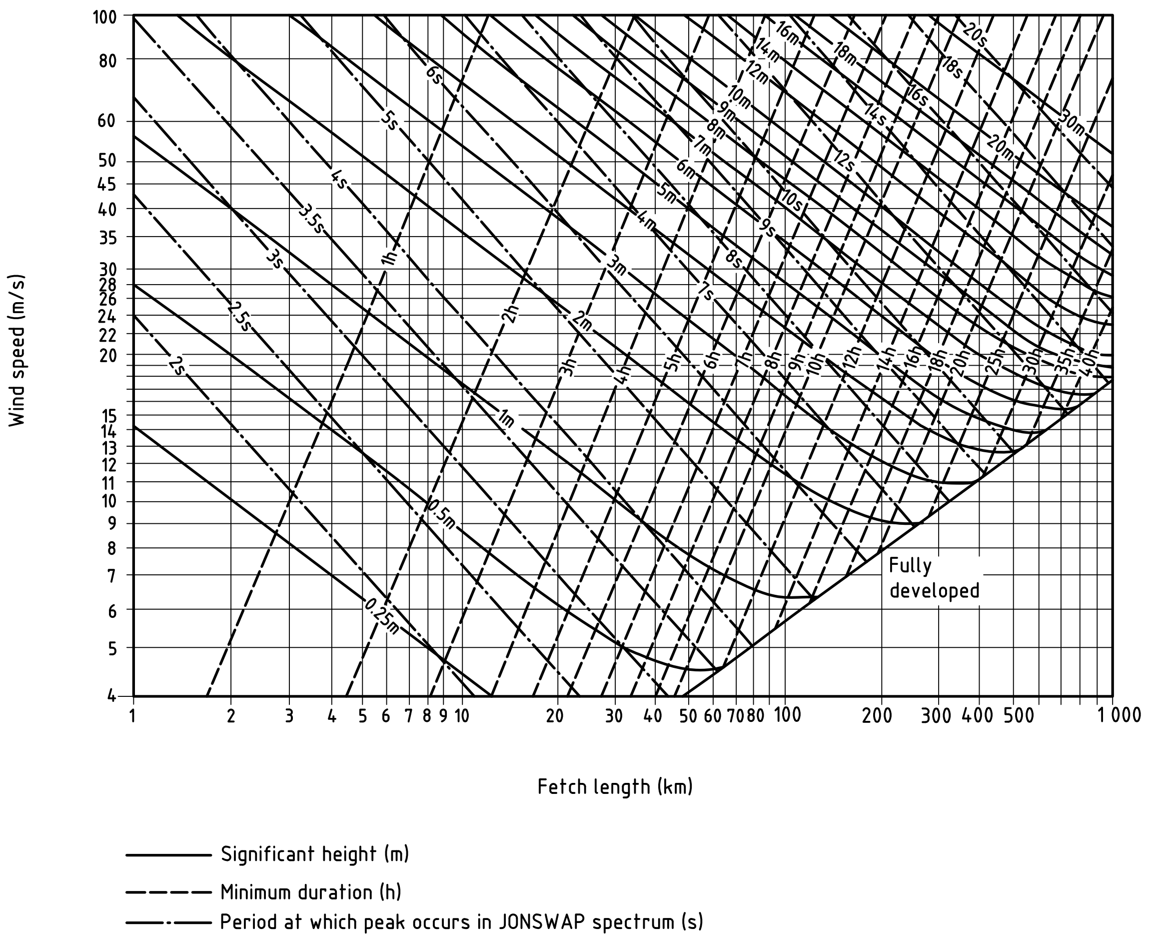
$U_{19.5}$ is the wind speed at 19.5 m above the sea surface.

Figure C.4 shows the Pierson–Moskowitz spectrum for the case where $U_{19.5}$ = 22.66 m/s.

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 exceeds that obtained from the fully developed spectrum. This difficulty can be overcome by using Figure C.5, where the contours of H_s have been adjusted to give a smooth transition between the two spectra.

Figure C.5 **Significant wave height and peak period for wave spectra**



Annex D (informative) Independent extreme values analysis

D.1 Reliability of extremes analysis

Estimates of extreme values obtained by statistical analysis (extrapolation) rely on the duration of raw data as being typical or representative of long-term conditions. If storm events were particularly severe or mild during the period for which data is available, then the extrapolations give overestimates or underestimates of extreme values. The extrapolations are expected to become more reliable when based on a longer duration of data.

Statistical analysis techniques assume that the parameter-generating mechanisms remain constant in the long term. For this reason, extrapolations to return periods in excess of, say, 100 years need to be viewed with caution, because their reliability can be affected by long-term changes in the climatic pattern.

The raw data set needs to be screened to select peak values arising from separate storms, so as to ensure statistical independence. Care is needed to ensure that this step is not overlooked in an attempt to increase a limited input data set by including non-independent events.

The broad approach described in this annex can be applied to differing parameters, e.g. wave height, water-level or other parameters such as current velocity or wave period, but modifications to the methodology might be required to reflect the different characteristics of each.

D.2 Probability density functions

The method consists of plotting the peak values (selected above a defined threshold and from separate events within the raw data) against their cumulative probabilities of occurrence. It uses an appropriate probability density 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.

Taking wave height estimation as an example (for which the significant wave height is generally used for extremes analysis), 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 density functions have been found to be appropriate in different situations. Sometimes one distribution fits the lower wave heights well and another distribution fits the higher wave heights better, possibly indicating two different wave populations. In these cases the distribution with the best fit to the larger waves is used for extrapolation. The following distributions might be appropriate and are provided for information to support preliminary analyses only.

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. Otherwise plot y against $\log H_n$ where y can be obtained, with the aid of tables, from:

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

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

Many other distributions exist and are often applied to such analyses; those commonly adopted include the exponential, generalized extreme value (GEV) and generalized pareto.

The resulting straight line plot gives the probability of certain wave heights (or other parameters) being equalled or exceeded during the period over which the original set of data was obtained. Provided that 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. It follows that the maximum height, H_{nx}, obtained from n_x values representative of a period of observation, T_0, will have a return period equal to T_0. 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, has a probability of occurrence of:

$$\frac{T_0}{T_R(n_x + 1)}$$

on the probability plot, which can then be used to obtain the relevant extreme wave height.

In practice it is desirable to use several years of records from which to abstract the necessary data, because any shorter duration is unlikely to yield a representative set.

The quality of a particular fitted distribution has to be assessed by visual inspection as well as more quantitative statistical measures. The designer needs to select a combination of statistical model and threshold (or number of events) that best fits the distribution and is conservatively biased to account for uncertainty due to the limited duration of the available hindcast data set compared to the return periods being estimated.

D.3 Extrapolation of wave periods

Depending on the source of data adopted, the wave period contained therein could be the mean wave period, T_m, the significant wave period, T_s (=$T_{1/3}$), or the period at which the peak occurs in the wave spectrum, T_p (or potentially other definitions such as the mean energy wave period, $T_{m-1,0}$). For standard wave spectral shapes the following inequalities apply:

$$T_m < T_s < T_p$$

It is usual to produce a scatter diagram or plot of wave height against wave period on which curves of constant wave steepness can also be plotted, or alternatively lines of best fit representing the wave height/wave period relationship (biased toward extreme or storm events). In general, this diagram suggests a prevalent value of wave steepness that can be assigned to the design condition. However, if there is considerable scatter in the values of wave steepness, it might be necessary to assess a range of values to be associated with the design wave height. A more involved approach requires wave period to be related to wave height using a conditional log-normal or extremal model. It might also be necessary to separate the wave height/wave period population into directional sectors, if such an approach was deemed appropriate in the treatment of wave height extremes.

For a given design wave height, the lower values of wave steepness, 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 needs to be included in the design parameters.

For storm waves, the wave steepness, $2\pi H_s/(gT_m^2)$, in terms of significant wave height and the deep water wave length associated with the mean period of primary waves, is typically within the range 0.03 to 0.06. For a fully developed sea state it can be taken as 0.05 irrespective of the significant wave height, although this might not be appropriate in areas influenced strongly by swell waves.

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 mean wave period, T_m. For a fully developed sea state, $T_p/T_m \approx 1.4$.

Annex E (informative) Wave and current actions

E.1 Steady current 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 following expression:

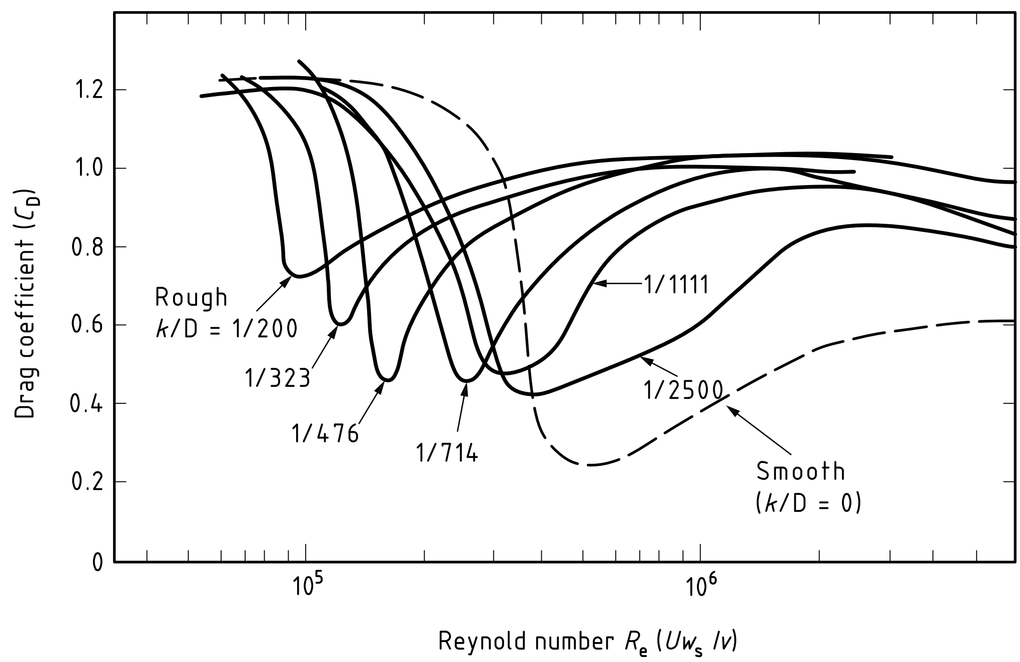
$$F_D = \frac{1}{2}\left(C_D \rho V^2 A_n\right)$$

where:

- F_D is the steady drag force, in newtons (N);
- C_D is the dimensionless time-averaged drag coefficient for steady flow;
- ρ is the mass density of the water, in kilograms per cubic metre (kg/m³);
- V is the incident current velocity, in metres per second (m/s);
- A_n is the area of structural member normal to flow, in square metres (m²).

Values of current drag force coefficients for circular section piles (or tubes/cylinders of other types) are dependent on the Reynolds number and surface roughness. Suggested values are given in Figure E.1 for circular sections with different degrees of surface roughness, due to surface finish/structural elements or marine growth.

Figure E.1 **Drag force coefficient values for circular cylinders**



E.2 Flow-induced oscillations

Current action on slender elements can generate unsteady flows where vortex-shedding occurs. The frequency of vortex-shedding can coincide with (or be a multiple of) the natural frequency of the structural element and thus cause unfavourable harmonic response. Further guidance is provided in ISO 21650:2007, **G.2.8**, and a simplified method of analysis for preliminary purposes only is provided below.

The critical flow velocity (for circular sections) can be calculated from the expression:

$$V_{crit} = Kf_N W_S$$

where:

V_{crit} is the critical flow velocity, in metres per second (m/s);

K is the dimensionless 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;

f_N is the natural frequency of the structure, in hertz (Hz);

W_S is the diameter of cylinder, in metres (m).

The values for f_N and W_S can be derived taking due account of the effect of marine growth, although since the critical condition for flow-induced oscillation usually occurs during construction, marine growth might be negligible.

*NOTE 1 Guidance on the calculation of the natural frequencies of structural members is given in Clause **34**.*

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 type of structure are given in Figure E.2. 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 E.1 to the values obtained from Figure E.2.

NOTE 2 The curves presented in Figure E.2 are conservative in that they assume the water surface is at the top of the pile.

Figure E.2 **Critical flow velocity for circular piles for in-line oscillations**

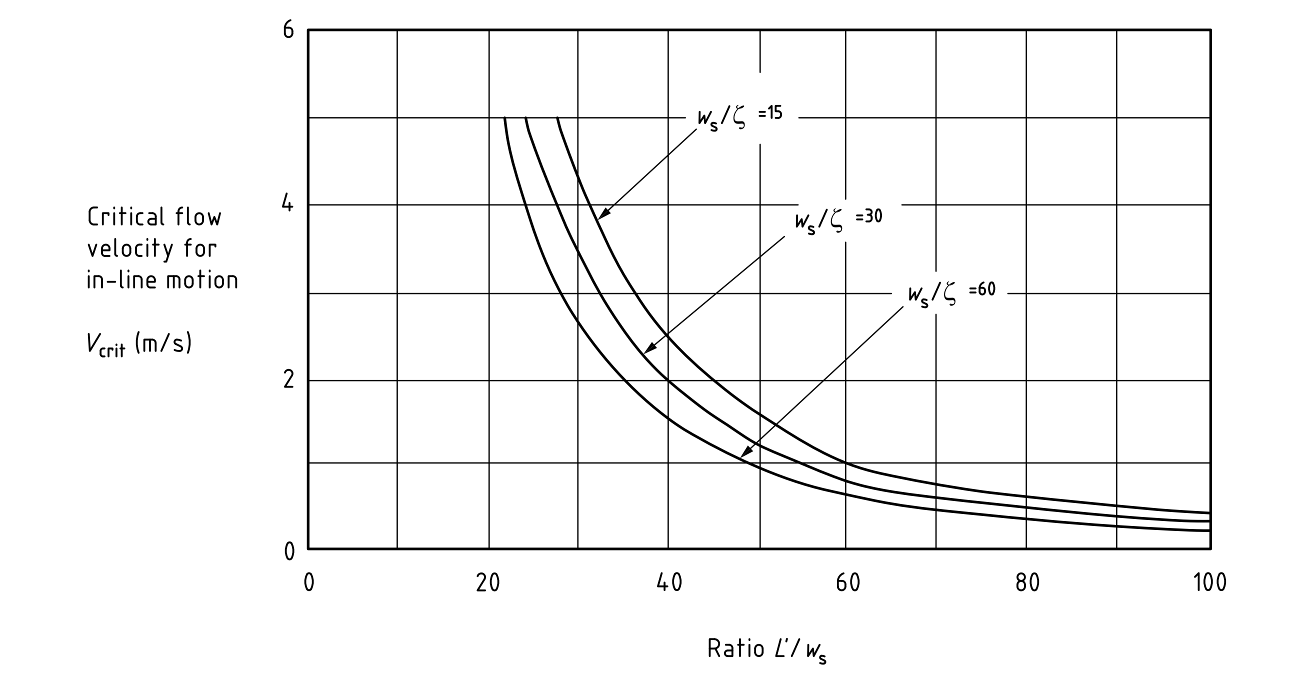


Table E.1 **Modification factors for critical flow velocity**

Motion	Modification factor			
	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

NOTE 3 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, the properties of the structure and its elements are preferably selected on the basis of either:

- a critical flow speed that is higher than the design current speed; or
- a mass and damping that are sufficient to prevent significant motion.

The first criterion may be assumed to be satisfied if the current speed is less than $1.2f_N W_S$. The second criterion may be assumed to be satisfied if the mass damping coefficient is greater than 2.0 in the case of in-line motion and greater than 25 in the case of cross-flow motion, where the mass damping coefficient is calculated from the expression:

$$\frac{2\bar{m}\Delta}{\rho W_s}$$

where:

Δ is the logarithmic decrement of structural damping (taken as 0.07 for most maritime structures);

ρ is the mass density of the water, in kilograms per cubic metre (kg/m³);

W_s is the diameter of cylinder, in metres (m);

$\bar{m}$ is the equivalent excited effective mass per unit length, in kilograms per metre (kg/m), given by:

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

where:

m_L is the mass per unit length of the cylinder (including contained water and the added hydrodynamic mass), in kilograms per metre (kg/m);

$y(x)$ 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, in metres (m);

l' is the length from apparent fixity level to water level, in metres (m).

E.3 Wave action on vertical (or inclined) cylindrical structures

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, which can be approximated by:

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

where:

- H is the wave height, in metres (m);
- d is the still water depth (at the structure), in metres (m);
- L is the wave length, in metres (m).

The Morison equation can be expressed as follows:

$$F_W = F_D + F_I$$

where:

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

$$F_D = \int_0^{L_s} \left(1/2 C_D \rho W_s \left| U \right| U\right) dL_s$$

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

$$F_I = \int_0^{L_s} \left(C_I \rho A \dot{U}\right) dL_s$$

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 force coefficient;
- C_I is the inertia force coefficient;
- U is the instantaneous water particle velocity normal to the member axis, in metres per second (m/s);
- $\dot{U}$ is the instantaneous water particle acceleration normal to the member axis, in metres per second squared (m/s^2);
- ρ is the mass 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 E.1 and of C_I and C_D for some standard structural shapes in Table E.1.

The following expressions for instantaneous water level, particle velocities and acceleration are derived from linear theory.

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

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

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

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

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

where:

η is the height of the water surface above still water level, in metres (m);

u is the horizontal component of water particle velocity, in metres per second (m/s);

v is the vertical component of water particle velocity, in metres per second (m/s);

$\dot{u}$ is the horizontal component of water particle acceleration, in metres per second squared (m/s^2);

$\dot{v}$ is the vertical component of water particle acceleration, in metres per second squared (m/s^2);

(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, in metres (m);

H is the wave height, in metres (m);

L is the wave length, in metres (m);

T is the wave period, in seconds (s).

E.4 Wave action on walls

Waves incident upon an infinitely long vertical surface (or in practical terms a relatively long seawall or breakwater) can be reflected without breaking, in which case a standing wave is formed in front of the wall with a height, in the case of regular waves, twice that of the incident wave (known as clapotis).

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 advisable for a site-specific investigation to be made.

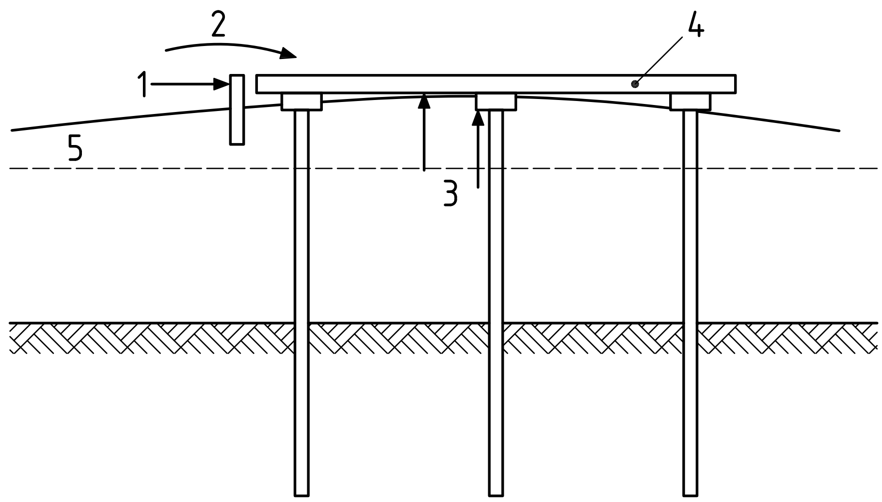
E.5 Wave action on horizontal structures

Hydraulic loads applied by waves to the deck or other projecting elements (beams, fenders) can be described as "wave-in-deck loads". These can be summarized as:

- uplift loads on decks;
- uplift loads on beams or other projecting elements;
- downward loads on decks (inundation and suction);
- horizontal loads (both seaward and shoreward) on beams or other projecting elements, e.g. fenders.

A schematic diagram of wave-in-deck loads acting on a piled jetty is shown in Figure E.3. The nature, occurrence and magnitude of these wave loadings vary significantly for different structures and wave conditions. Horizontal elements such as deck slabs can be subject to large vertical forces upward or downward, particularly under conditions that inundate the deck. Vertically faced elements like beams and fenders can experience significant forces, both horizontally and vertically, if sufficiently thick.

Figure E.3 **Schematic diagram of wave-in-deck loads**



Key

1 Horizontal loads
2 Inundation loads
3 Uplift loads on deck and beams
4 Jetty deck
5 Still water line (SWL)

Annex F (informative) Approximate method of assessment of response and displacement of simple structures under cyclical loading

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

$$f_N = \frac{1}{2\pi}\sqrt{\frac{k_s}{m_e}}$$

where:

k_s 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:

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

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_s), is known as the dynamic amplifier. If this exceeds 1.2, more exact analytical methods are needed.

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 can be used.

In calculating the stiffness of a piled system, the effective length of pile from deck level to apparent fixity level can be used. The apparent fixity level lies at a depth below seabed of between $4W_s$ for stiff clays and $8sW_s$ for soft silts, where W_s is the pile diameter, but allowance needs to 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 can be taken to include:

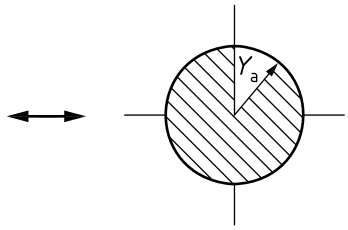
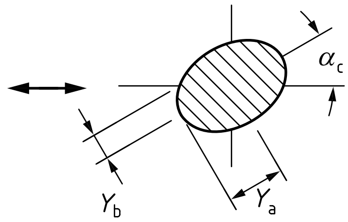
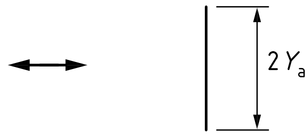
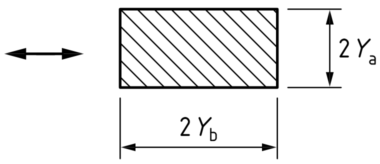
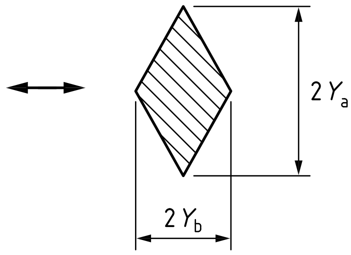
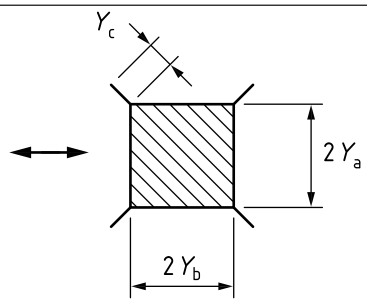
1) the actual mass of the structure, including the mass of any attached marine growth, with no allowance for buoyancy;

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 F.1 for a number of cross-sections.

Table F.1 **Added mass of entrained water**

Cross-sectional shape of prismatic member		**Added mass of entrained water per unit length** A)	
Circular section		$\rho\pi Y_a^2$	
Elliptical section		$\rho\pi\,(Y_b^2\cos^2\alpha_c + Y_a^2\sin^2\alpha_c)$	
Flat plate		$\rho\pi Y_a^2$	
Rectangular section		$K_1\rho\pi Y_a^2$	
		Y_a/Y_b	K_1
		0.1	2.23
		0.2	1.98
		0.5	1.70
		1.0	1.51
		2.0	1.36
		5.0	1.21
		10.0	1.14
Lozenge section		$K_2\rho\pi Y_a^2$	
		Y_a/Y_b	K_2
		0.2	0.61
		0.5	0.67
		1.0	0.76
		2.0	0.85
Square section with corner projections		$K_3\rho\pi Y_a^2$	
		Y_c/Y_a	K_3
		0.05	1.61
		0.10	1.72
		0.25	2.19

A) ρ is the water density.

Unless better information is available it can be assumed that marine growth has the values indicated in Table F.2, below LAT.

Table F.2 **Indicative mass allowance for marine growth**

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

It is important to obtain location-specific data on marine fouling potential for final design since heavier fouling can occur. For example, seaweed fouling by kelp can reach 3 m in thickness in some North Sea offshore installations.

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 **24.3**.

Annex G (informative) Wind and current forces formulations

G.1 General

This annex gives methods of assessment of current and wind forces for assessment of mooring actions using empirical formulations based upon ship model test data.

The formulations used in previous editions of this standard are reproduced here (referred to as the BS 6349 formulation and described further in **G.2**). They are maintained without modification and are based in turn on a range of published data brought together since the 1950s.

For oil and gas industry application, the methodology set out in OCIMF MEG 3 [N2] (referred to as the OCIMF formulation and described further in **G.3**) is recommended.

It is important to recognize the difference between the formulations to avoid errors or inappropriate application.

The historical background to the development of the formulations is given here since it was considered that this is needed to remain accessible to designers, as important background information which might still be useful in preliminary assessment of wind and current forces on vessels.

G.2 BS 6349 formulation

Overall wind and current forces can be described by either:

- longitudinal and transverse forces combined with a moment about a vertical axis (e.g. through the centre of gravity), all acting at the centre of the vessel; or
- two transverse forces, one at each perpendicular, combined with a longitudinal force.

The latter method has been adopted for this standard, and the magnitude and sense of the forces may be evaluated using the expressions given as follows.

For wind forces:

[C1⟩ $F_{TW} = C_{TW}\rho_A A_L V_W{}^2 \times 10^{-4}$ ⟨C1]

[C1⟩ $F_{LW} = C_{LW}\rho_A A_L V_W{}^2 \times 10^{-4}$ ⟨C1]

where:

F_{TW} is the transverse wind force, forward or aft, in kilonewtons (kN);

NOTE 1 Forward and aft forces can be combined to give the total transverse wind force.

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/m³ and can be taken to vary from 1.309 6 kg/m³ at 0 °C to 1.170 3 kg/m³ at 30 °C;

A_L is the longitudinal projected area of the vessel above the waterline, in square metres (m²);

V_W is the design wind speed, in metres per second (m/s) at a height of 10 m above water level.

For current forces:

[C1⟩ $F_{TC} = C_{TC}C_{CT}\rho L_{BP} d_m V_c{}'^2 \times 10^{-4}$ ⟨C1]

[C1⟩ $F_{LC} = C_{LC}C_{CL}\rho L_{BP} d_m V_c{}'^2 \times 10^{-4}$ ⟨C1]

where:

F_{TC} is the transverse current force, forward or aft, in kilonewtons (kN);

NOTE 2 Forward and aft forces can be combined to give the total transverse wind force.

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;

NOTE 3 This is to be included when the depth to draught ratio is less than six.

C_{CT} is the depth correction factor for transverse current drag forces;

NOTE 4 This is to be included when the depth to draught ratio is less than six.

ρ is the mass density of water, in kilograms per cubic metre (kg/m³), and can be taken as 1 000 kg/m³ for fresh water and 1 025 kg/m³ 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 current velocity, averaged over the mean draught of the vessel, of the component of current in the direction under consideration, transverse or longitudinal, in metres per second (m/s).

Values for wind force coefficients are given in Figure G.1, Figure G.2 and Figure G.3 for various angles of wind approach for various types of vessel, both in the ballasted and loaded condition.

Figure G.1 **Envelope of wind force coefficients for dry cargo vessels and small tankers**

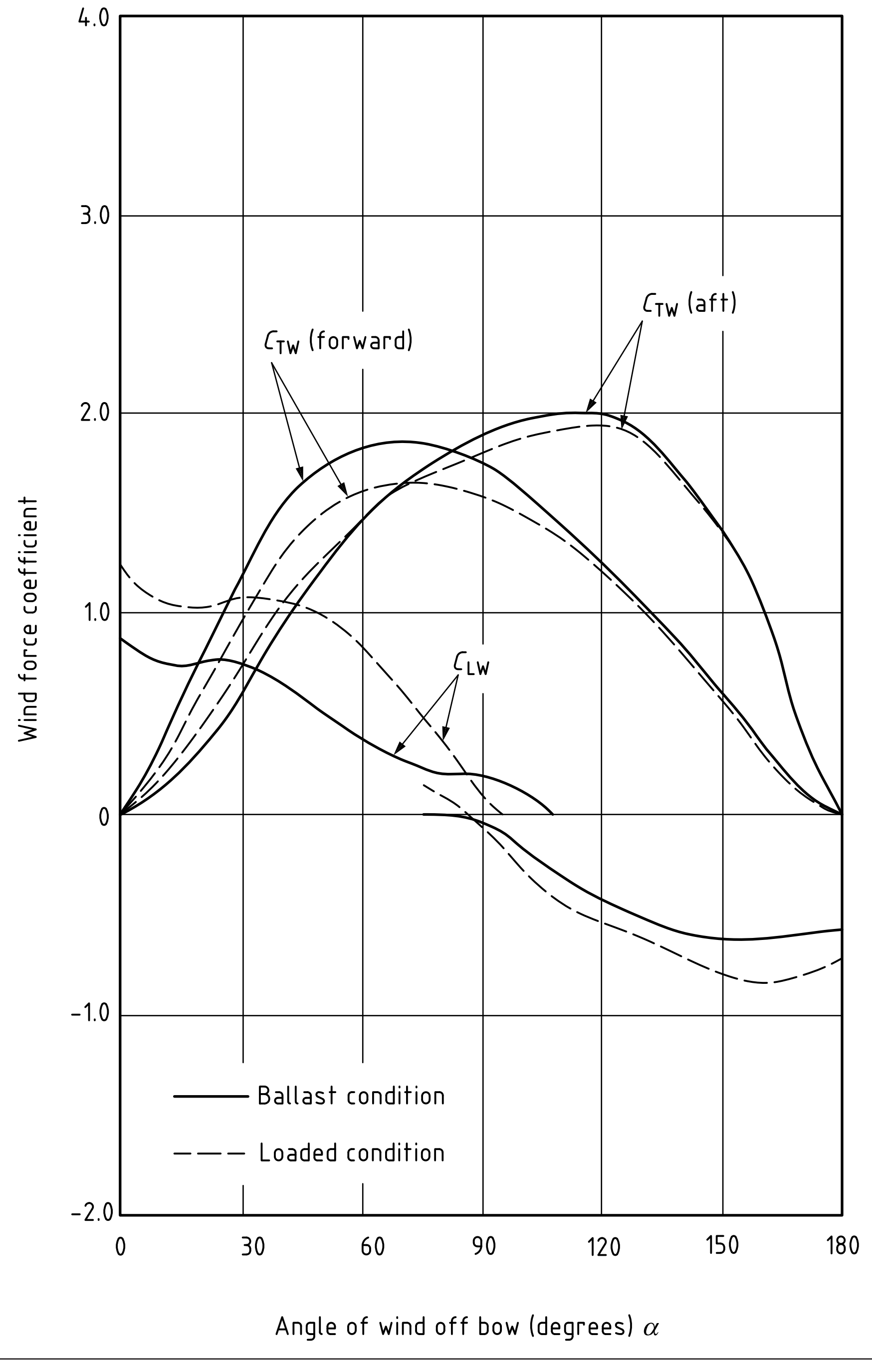


Figure G.2 **Wind force coefficients for very large tankers with superstructures aft**

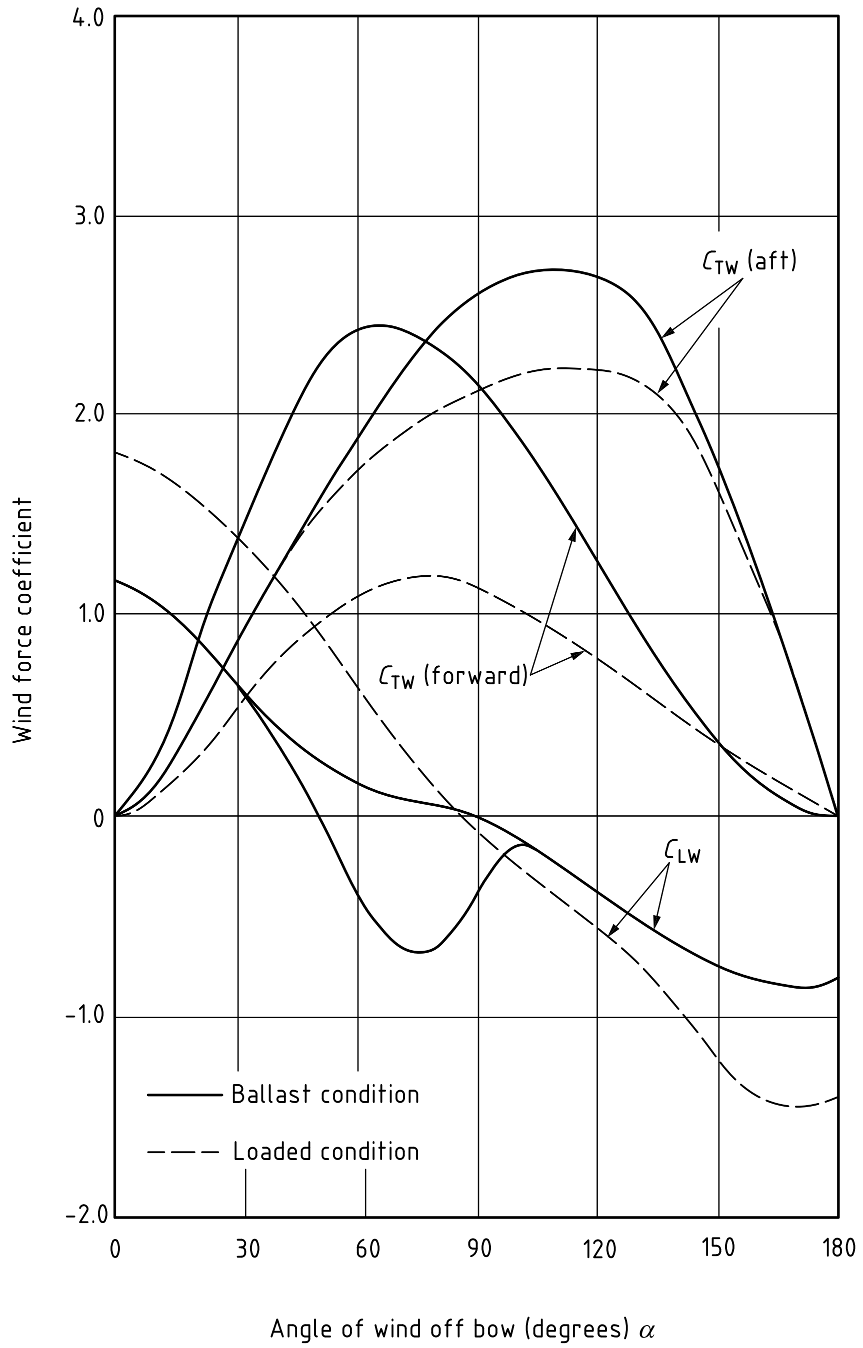
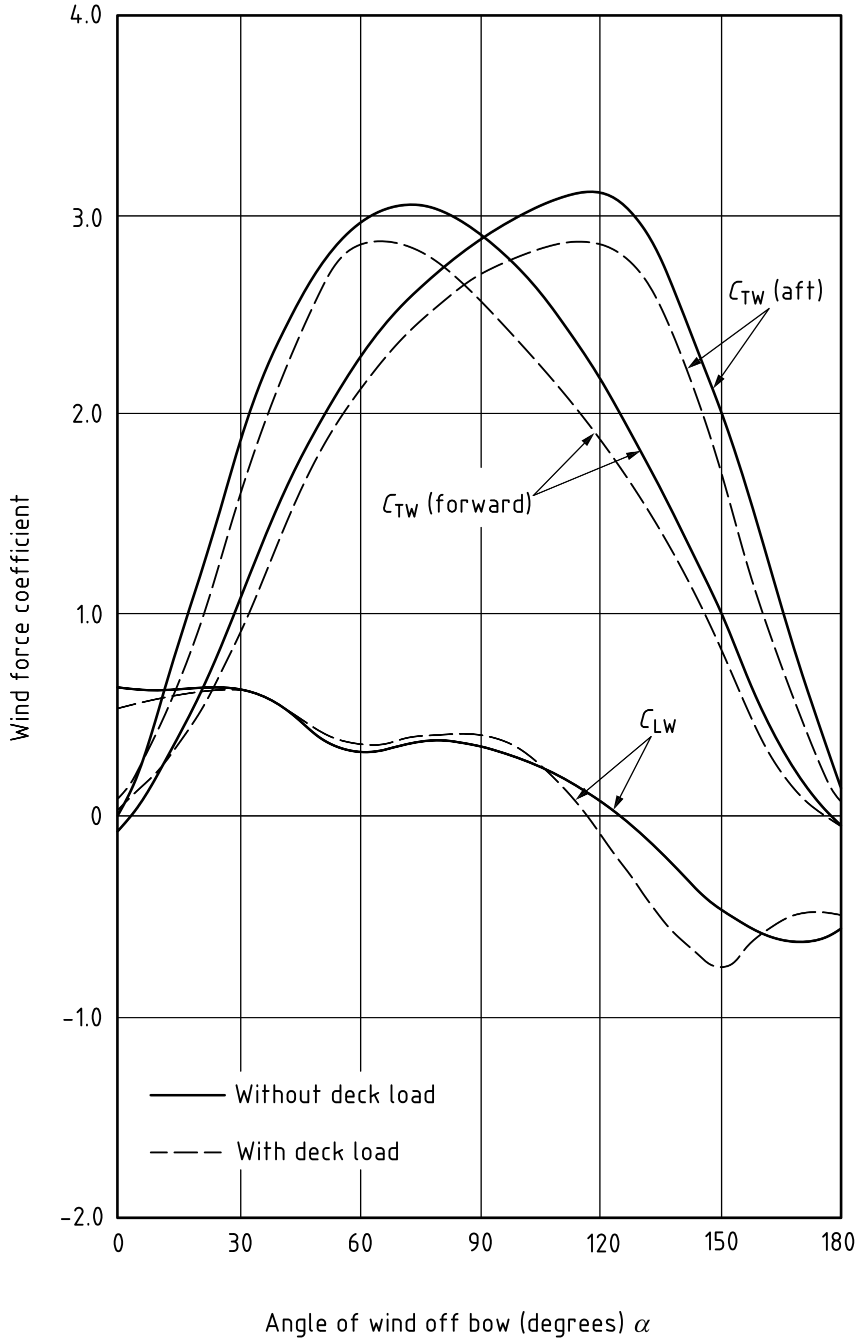


Figure G.3 **Wind force coefficients for typical container ship**



Values for current force drag coefficients are given in Figure G.4 and correction factors for shallow water effects in Figure G.6 and Figure G.6.

Figure G.4 **Current drag force coefficients, all ships, deep water case**

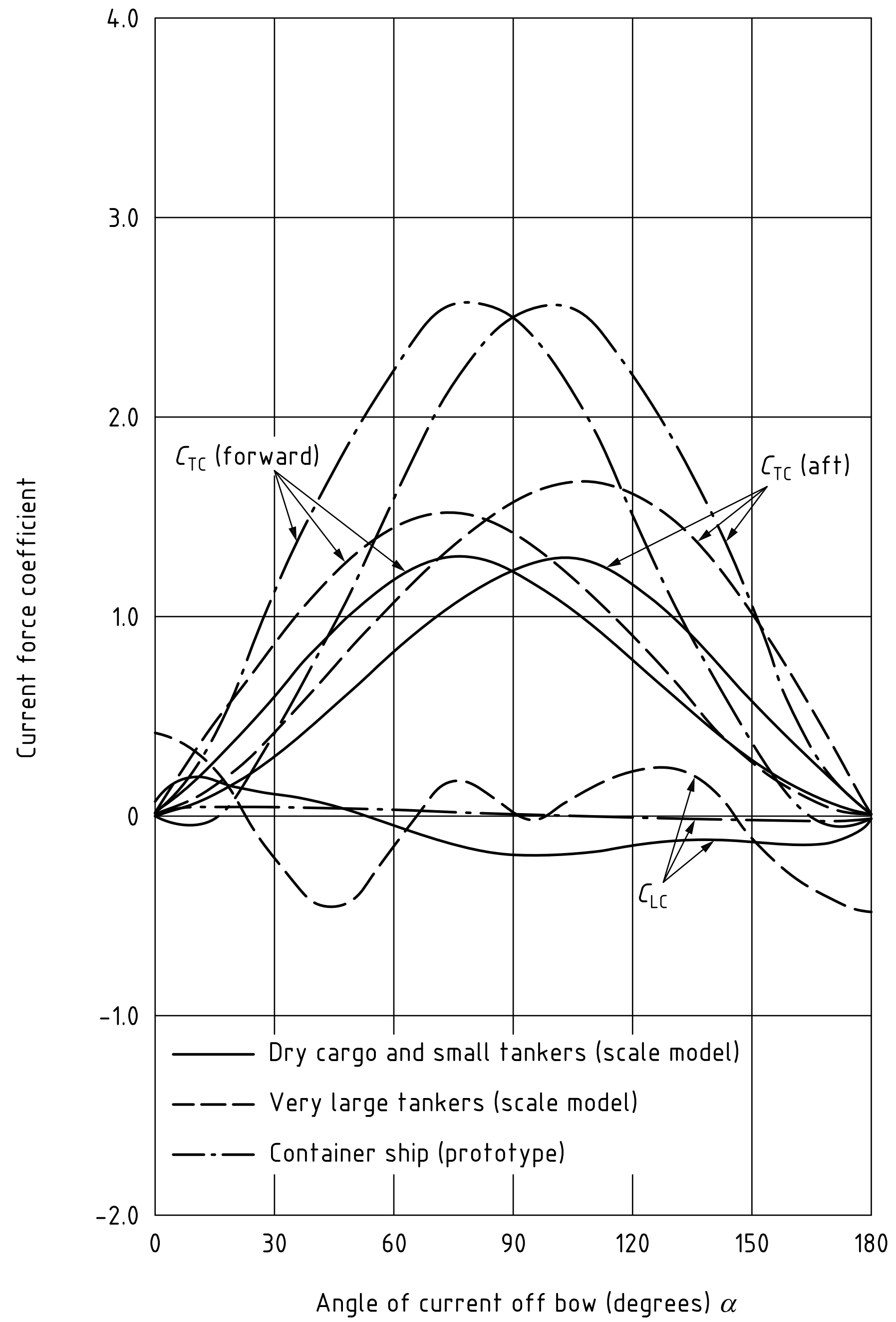


Figure G.5 **Water depth correction factors for lateral current forces**

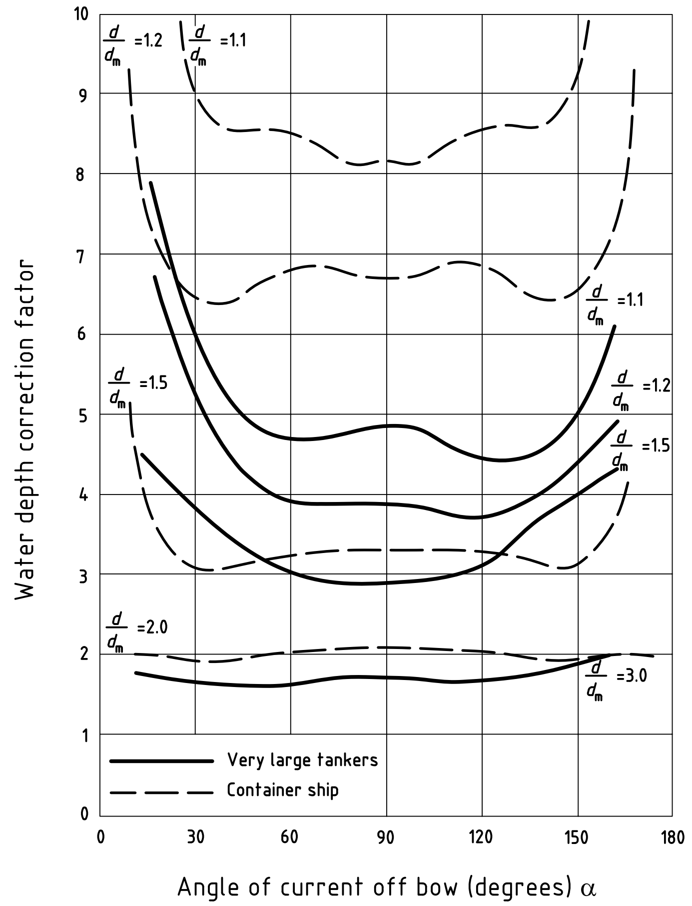
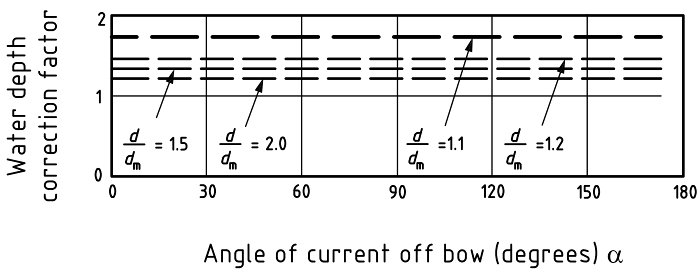


Figure G.6 **Water depth correction factor for longitudinal current forces on container ships**



Values of coefficients for very large crude carriers (VLCCs) and tankers in Figure G.1 and Figure G.2 are included for the purposes of comparison with other ship types. However, as noted in **29.3**, the OCIMF formulation is recommended for evaluation of wind and current drag coefficients for VLCCs and other oil and product tankers down to 16 000 DWT and gas carriers in the range 75 000 m^3 to 125 000 m^3.

Typical values for the lengths, draughts and lateral areas of bulk carriers and container vessels are given in BS 6349-1-1:2013 and in Figure G.7 and Figure G.8. These figures can be taken as guides for preliminary design in the absence of specific vessel data.

Figure G.7 **Typical longitudinal projected areas of tankers**

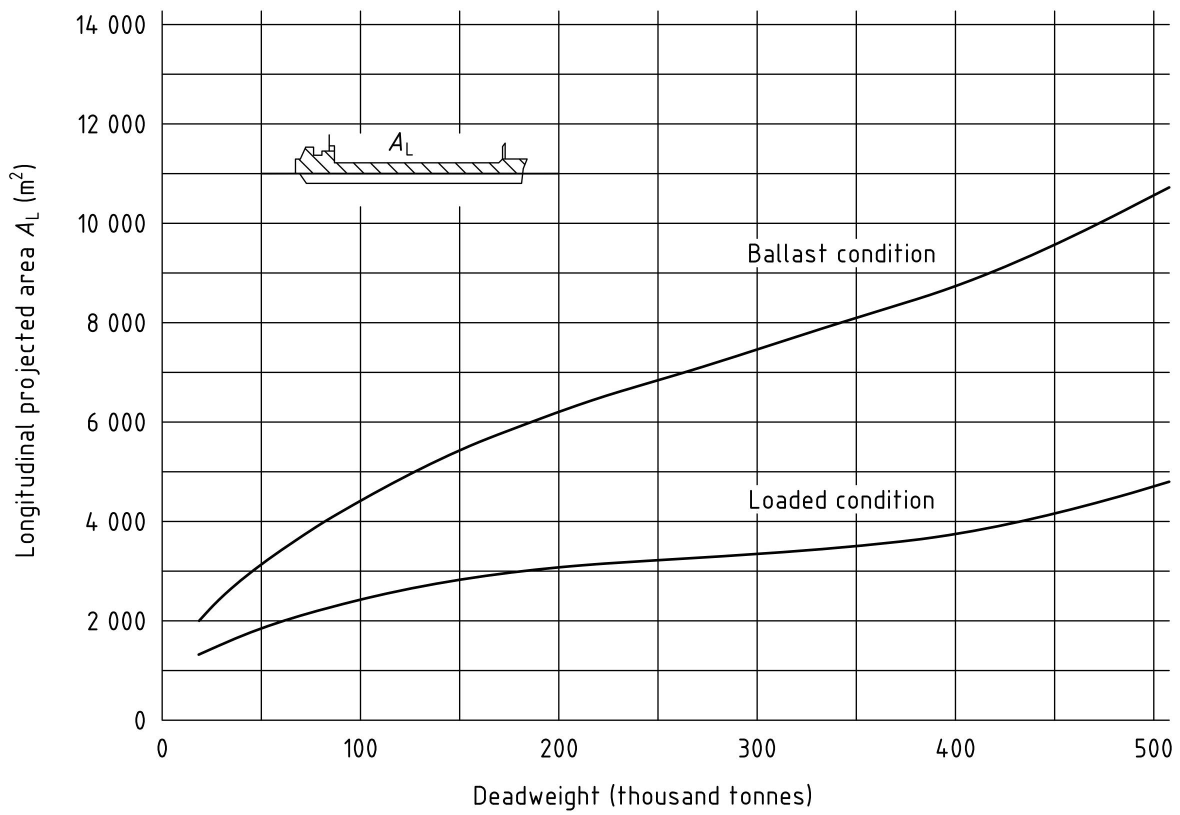
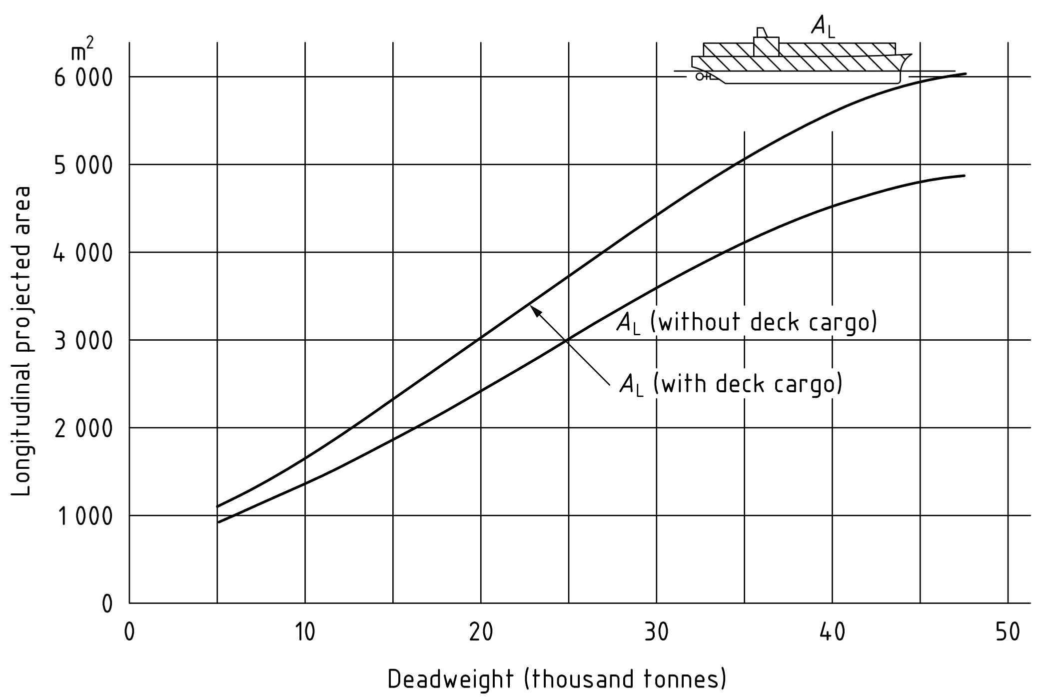


Figure G.8 **Container ships: lengths and longitudinal projected areas**



G.3 OCIMF formulation

The OCIMF formulation as included in OCIMF MEG 3 [N2] is different to the BS 6349 formulation. PIANC MarCom Report 116 [31], Appendices B and C include a comparison between BS 6349, PIANC and Spanish ROM standard formulations.

OCIMF MEG 3 contains guidance on assessment of applicability of the coefficients to a wider range of tanker sizes and more modern designs, noting the following.

- For smaller oil and product tankers, geometric similarity is significant in assessing applicability of coefficients. This means that similarity of the ratio of freeboard to hull breadth is necessary for application of the transverse wind drag coefficients, and similarity of the ratio of draught to hull breadth is necessary for application of the transverse current drag coefficients.
- For gas carriers, the wind coefficients apply to membrane and spherical tank designs in the range 75 000 m^3 to 125 000 m^3. For gas carriers outside this range, generally acceptable wind drag coefficient data is not available and, unless model tests are to be carried out in each case, a conservative extrapolation from the published gas carrier data is advised.
- The wind tunnel oil tanker models correspond to typical tankers before the International Convention for the Prevention of Pollution from Ships (MARPOL) of 1973 with a ratio of overall length to loaded freeboard in the range of 50 to 60, and a ratio of ballast freeboard to full load freeboard of 3. Post-MARPOL (SBT and double hull) tankers generally have higher freeboard than pre-MARPOL tankers, and interpolation or extrapolation would most appropriately be on the basis of the ratio of midships freeboard to hull breadth.

OCIMF MEG 3 also points out that current drag coefficients are based upon test data from out-of-print OCIMF publication *Prediction of wind and current loads on VLCCs* [32], in turn based on tankers with a length to beam ratio in the range of 6.3 to 6.5. Some tankers have a lower length to beam ratio and earlier studies indicate that the longitudinal current drag coefficient might be 25% to 30% higher, with a length to beam ratio of 5.0. Unlike the longitudinal wind drag, the longitudinal current drag is calculated in terms of the hull length times draught.

G.4 Wind speeds and spectrum

Typically BS 6349-1-1:2013 recommends the use of the 60 s averaging period wind speed for the assessment of handling and mooring of large ships such as oil and gas carriers. For steady state analysis of a moored ship, the OCIMF MEG 3 formulation is based upon a 30 s averaging period wind speed, in turn considered representative of the typical time for most significant mooring load response of a moored ship.

In some circumstances a shorter duration might be justified, e.g. where roll response could be of shorter short duration.

When dynamic modelling is carried out to assess mooring loads, it is necessary to include a wind spectrum appropriate to the vessel's dynamic response modes through a range of gust averaging periods between 30 s and 3 s gusts.

G.5 Validity of drag factors

Apart from vessel dimensions and proportions, both BS 6349 and OCIMF formulations are representative of ideal situations and are valid for unsheltered vessels in open water. They are also deemed to be valid when moored to hydrodynamically open structures, for example for vessels moored against breasting dolphins/loading-platform, provided that the piles are relatively widely spaced.

Against vertical quaysides, or densely placed piled structures, judgement has to be made as to the application of drag coefficients, particularly if winds are off berth, or the currents are not end-on (bow/stern). In the former case, an allowance can be made for winds blowing a ship off-berth, by reducing the exposed windage of the hull by the area sheltered from the waterline to the quay deck.

In the case of strong currents (greater than approx. 4 knots), there have been some situations where vessels are moored against densely piled quays with strong currents at a fine angle to the ship (up to about 10°). These cases cannot be assessed in the normal way. The flow on the quay side of the ship slows down, with the effect that the water level on that side of the ship increases, an effect which leads to an increase in the differential head across the beam of the ship. This gives rise to a strong off-berth loading as a stand-off force, which can supplement the normal drag force, and, depending on conditions, can amount to as much again in magnitude.

G.6 Historical background to vessel drag coefficients

OCIMF tanker wind force coefficients are those from the out-of-print publication *Prediction of wind and current loads on VLCCs* [32]. This publication stated that the coefficients were for vessels in the 150 000 to 500 000 class. However, the more recent OCIMF MEG 3 [N2] states that the original wind and current coefficients were defined originally for VLCC size ships above 150 000 DWT, but further adds that more recent model test data on tanker forms which have superstructures aft and segregated ballast configurations confirms that these same coefficients are generally applicable to smaller ships, and that they may therefore be used for a range of ships down to approximately 16 000 t.

OCIMF published tanker wind force coefficient data for two different bow shapes, the V-shaped bow and the U-shaped bow. For the first the bow shape at the water line is relatively sharp and defined as “conventional” and probably appropriate for most tankers. The second is referred to as “cylindrical” and is appropriate for tankers with a very rounded bow shape extending down to the water line.

However, this difference in bow shape appears only to affect the ballasted tanker longitudinal wind force coefficient. At oblique angles of between approximately 40° and 90°, a suction effect takes place which significantly increases the longitudinal force. The OCIMF wind force tests show no other differences between these two bow shapes.

The OCIMF tests were conducted on models representing four different tankers in the 155 000 DWT to 500 000 DWT range, loaded and in-ballast. Other than state of loading, fully loaded versus ballasted, and the difference in bow shape, OCIMF makes no distinctions as to hull or superstructure shape. OCIMF does however state that the data for the various tankers were adjusted to define representative mean curves.

OCIMF also states that the data were further adjusted to reflect uncertainties inherent in model test results. The ballasted lateral force and yaw moment coefficient data were increased by 10% and the longitudinal force coefficient force data by 20%. OCIMF wind force coefficients are therefore probably conservative for most tankers. As already indicated, caution is needed in comparing the OCIMF data with those from other sources because of probable differences in how the coefficients are defined and adjusted.

Other wind force coefficients sometimes used for different shaped ships are available from the Royal Institution of Naval Architects paper *Large tankers – Wind coefficients and speed loss due to wind and sea* [33]. The data reported in that paper were based on tests conducted at the Aeronautical Research Institute of Sweden and financed by the Swedish Board for Technical Development.

Most of the work was carried out on a model of a 280 000 DWT ore/oil carrier. Alternate superstructures and scale factors were tested to represent tankers over the range from 100 000 DWT to 500 000 DWT. Several alternative aft house were tested on the 280 000 DWT model.

The OCIMF publication *Prediction of wind loads on large liquefied gas carriers* [34], also out of print, gives separate wind force coefficients for liquefied natural gas carriers (LNGCs). These data are based on tests conducted on models of gas carriers in the 75 000 m^3 to 125 000 m^3 class. OCIMF published wind force coefficient data for two different gas carrier tank configurations: prismatic tanks (membrane) and spherical tanks. The prismatic tank gas carrier models which were tested had higher freeboard than conventional tankers but otherwise did not appear to have much additional above-deck area. The spherical tank gas carrier model had essentially the same freeboard as the prismatic tank vessels, but in addition had pronounced spherical tanks projecting above the main deck. Thus they appeared to have an appreciably extra transverse above-deck wind area.

Relatively few measurements have been made of drag on stationary vessels in cross-flow at different angles in water channels with restricted depth and width. Longitudinal drag is sensitive to hull shape and direction of flow. However, the longitudinal drag force is a relatively small component of the total force, except at very small angles of attack.

The equations used are empirically based on results of model tests conducted at Newcastle University, supported by the Science and Engineering Research Council and several industrial sponsors. The tests were for a rectangular barge-shaped vessel at various draughts and water depths and held at various angles to water flow in a channel of finite width. The results of these equations seem to correspond closely with those of other equations for most angles. The equations are useful for moored ship studies as they account for bottom clearance and blockage effects in defined channels.

Annex H (informative) Additional background guidance on assessment of design situations, loads and partial factors for mooring loads

*NOTE This annex provides informative material relevant to the assessment of mooring loads and selection of appropriate partial load factors in Clause **29**, using Table 8 and Table 9.*

Combinations of actions for structural design, including those applying to ship operational loads and mooring loads in particular, are described in Section **2**.

The recommended partial factor for actions, γ_Q, for "unfavourable" mooring loads for general application, is given as 1.50 for EQU (Set A) and STR/GEO (Set B) and 1.3 for STR/GEO (Set C), which is consistent with environmental loads and cargo loads as variable actions.

For mooring structures for large ships where mooring loads are frequently dominant (e.g. for mooring dolphins at exposed locations for oil and gas tankers), Table 8 provides additional recommendation on partial load factors in the context of method of assessment of loads and related operational considerations. Further commentary on Table 8 is provided in Table H.1 according to the operating condition and method of assessment of actions.

For mooring structures with multiple hooks, designing mooring points for the full berth mooring system capacity might be overly conservative where multiple mooring lines are attached, and when it is not possible for such mooring loads to be applied in any credible accidental or human error scenario.

It has thus been the practice of some operators, port authorities and facility owners to apply simplified rules on maximum mooring loads on mooring points, based upon the number of mooring hooks and the rated MBL of the mooring lines of ships moored to the hooks. The total accidental mooring point loads, as multiples of rated hook SWL or ship's mooring line MBL as given in Table 9, are intended to provide such simplified guidance. Table H.2 gives a description of the scenarios used as the basis of these multiple hook loads. The values are relevant for exposed environments (see Note 1 to **29.1**); however, designers could decide to adopt these values in non-exposed environments where other factors, including equipment malfunction and human error, pose a risk of exceptional loadings that are inconsistent with operating procedures and identified through risk assessment.

The SWL of each mooring hook is assumed to be specified to be not less than the MBL of the attached mooring line, and Table 9 shows the total accidental mooring point load as multiple of the factored rated hook SWL. This is consistent with the recommendation in BS 6349-4:2014 to design mooring points based upon the rated capacity (SWL) of mooring equipment. However, for large ships it might be conservative to design based on SWL, if the MBL of mooring lines of design ships are known with confidence and assumptions in this respect are properly documented and applied to operations through the facility operating manual. This is the reason for the mention in Table H.2 of "rated MBL of vessel's mooring line, where appropriate".

Table 9 is an illustration of an approach to assessing loads in circumstances which are unplanned or out with normal operational procedures and circumstances. However, this is not to say that location and operation specific risk assessment by competent persons might not lead to higher or lower accidental loadings.

Table H.1 **Commentary on use of partial load factors from Table 8 according to method of assessment of actions and operational factors**

Operating condition	Method of assessment of loads	Commentary on partial factor for actions (unfavourable)
Normal	Actions assessed by mooring analysis Probable maximum calculated loads for specified normal environmental conditions (in combination with breasting loads and/or other consistent operating loads where applicable)	A partial factor, γ_Q, is recommended in Table 8 and is consistent with environmental and cargo transfer/handling actions. Different combinations of actions might be required to represent situations when cargo transfer is being carried out (up to DSOL for cargo transfer) separately from situations when cargo handing is suspended but the ship remains berthed (up to the DSOL for the ship to remain at the berth).
Extreme	Actions assessed by mooring analysis Probable maximum calculated loads for specified extreme operating environmental conditions (in combination with breasting loads where applicable)	A partial factor, γ_Q, is recommended in Table 8 and is consistent with environmental and cargo transfer/handling actions, since the mooring loads are assessed from mooring analysis from environmental effects on the moored ship and are therefore assumed to be subject to a similar level of uncertainty.
Extreme	Actions assessed on the basis of a ship's mooring equipment capacity Extreme operating condition – load is based on rated capacity of ship's mooring equipment when this is known at the design stage and when appropriate to operating requirements	A reduced partial factor, γ_Q, can be adopted where this is supported by the assessment of maximum credible mooring loads for the ship's mooring equipment. An example is where the future berth operator prescribes oil and gas carriers conforming to OCIMF MEG 3 [N2]. In such a case, ships operating in accordance with OCIMF MEG 3 would be equipped with mooring winches rendering at a rated holding capacity of 0.6 × line MBL, with mooring capacity further limited by the brake design load of 0.8 × line MBL. Such an approach is only possible when designers have confidence that the ships using the berth will have equipment and be operated in compliance with these assumptions. It is not always necessary to design the berth for this condition, e.g. at a sheltered location and with relatively benign metocean conditions. Risk assessment by designers and operators considering credible operational scenarios would be an appropriate tool to make design choices on whether it is necessary to design the berth to be occupied in extreme events.
Accidental	Assessment of the maximum credible total load on mooring points based on: • risk assessment for the specific operations proposed and the prevailing environmental conditions; and • the limiting capacity of the vessel's mooring equipment and lines used to moor the vessel multiplied by not less than 1.18	For accidental design situations designers needed to be satisfied that: • the loads are of an exceptional nature and of short duration; and • damage in such a scenario is acceptable and that loss of life, total loss of structural integrity and disproportionate consequences and escalation are unlikely to occur. • Appropriate factors (not less 1.18) are used to determine the design load from SWL of mooring fittings and winch brake design loads to reflect the margin on mooring equipment capacity above limiting capacities derived from nominal rated capacity of lines, hooks and winches.

Table H.2 **Accidental loads for multiple hooks**

Number of mooring hooks per mooring point (N)	Total accidental mooring point load as multiple of the factored rated hook SWL (or factored rated MBL of vessel's mooring line, where appropriate)	Scenario for derivation of total mooring point load from mooring line MBL
2	1.8 × 1.18 = 2.1	1 × 0.8 + 1 × MBL = line on one mooring hook at MBL and the other at ship's winch design brake holding load
3	2.4 × 1.18 = 2.8	3 × 0.8 × MBL = lines on each hook at ship's winch design brake holding load
4	3.0 × 1.18 = 3.5	3 × 0.8 × MBL + 1 × 0.6 × MBL = 3 × MBL lines on each hook at ship's winch design brake holding load, one line at ship's winch brake setting.

A comparison of factored design load for a mooring structure with two, three or four hooks is illustrated in Table H.3. In a theoretical situation where mooring forces were the dominant consideration in determining structural capacity, this suggests the following in this hypothetical example.

- The assessment of mooring actions in this case results in the lowest factored loads compared to both the extreme mooring-equipment limited load case and the accidental load case. However, this result depends on the specific circumstances and environment. This will be different in, for example, a more exposed location, or with a different ship, or with a different berth or mooring layout.
- The extreme mooring-equipment limited load case gives factored loads greater than those of the assessment from mooring analysis. However, it would not in all cases be appropriate as a design case, e.g. at a sheltered location and with benign metocean conditions where the loads could not occur even in extreme events of 5 to 10 year return period.
- The accidental case results in the highest factored loads in this example. As noted above, Table 9 is applicable to exposed locations, subject to passing ship effects or when other scenarios of human error or equipment malfunction have been identified through risk assessment. Since the accidental case can result in very significant design loads, it is important to avoid overly conservative assumptions or accidental scenarios.

Table H.3 **Illustrative comparison of total factored loads on mooring points for different operating conditions and methods of assessment**

Operating condition and method of assessment of mooring loads	Number of mooring hooks per mooring point	Unfactored loads as a multiple of mooring line MBL	Partial factor	Factored loads as a multiple of mooring line MBL
Extreme operating condition – assessment of actions by static or dynamic mooring analysis for conditions up to the DSOL for a moored ship (e.g. 0.4 × MBL average each line, since all lines would not usually simultaneously reach the limiting peak allowable load)	2 3 4	0.8 1.2 1.6	1.5 1.5 1.5	1.2 1.8 2.4
Extreme operating condition – actions assessed on the basis of ship's mooring equipment (e.g. 0.6 × MBL each line as an upper bound)	2 3 4	1.2 1.8 2.4	1.3 1.3 1.3	1.6 2.3 3.5
Accidental operating condition – actions assessed on the basis of ship's mooring equipment as in Table H.2	2 3 4	2.1 2.8 3.5	— — —	2.1 2.8 3.5

Annex I (informative)

Physical properties of commonly stored cargoes

Typical values of bulk densities and angles of repose are given in Table I.1. Typical values of stacked densities are given in Table I.2.

Table I.1 **Typical dry bulk densities and angles of repose**

Material	Dry bulk density t/m³	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 to 1.50	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 to 1.50	35
Manganese (Manganite)	1.79 to 2.39	35 to 45
Alumina	1.00 to 1.70	35
Basic chemicals		
Sulfur	1.12 to 1.20	35 to 40
Phosphate rock	1.03 to 1.10	30 to 34
Kaolin	0.90 to 0.94	30 to 35
Solid fuels		
Coal	0.72 to 1.12	30 to 45
Coke	0.36 to 0.51	37 to 40
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 to 1.70	34
Cement	1.20 to 1.52	25
Clinker	1.29 to 1.54	30
Waste products		
Domestic refuse	0.56	
Scrap iron	1.0 to 1.6	35
Foodstuffs (normally stored in sheds or silos)		
Cereal	0.51 to 0.76	40
Sugar	0.78	40
Salt	0.90	45
Soya bean	0.82	35 to 60
Copra	0.51	35

Table I.2 **Typical stacked densities for common commodities**

Commodity	Stacked density t/m³
Timber	
Softwood:	
• Douglas fir	0.5 to 0.61
Hardwood:	
• Oak	0.6 to 0.83
• Greenheart	1.14
Plywood	0.45
Woodchip	0.24 to 0.52
Timber products	
Paper (in bales)	0.80
Paper (in rolls)	0.40
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
Soya beans (in bags)	0.72
Flour (in bags)	0.83
Rice (in bags)	0.70
Metal products	
Aluminium ingots	1.24
Copper ingots	3.00 to 3.59
Copper coils	1.12
Steel bars	2.24 to 3.00
Steel ingots/plates	3.00 to 3.59
Steel coils	1.20 to 3.00

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