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Incorporating Corrigendum No.1



BSI Standards Publication

Maritime works –

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

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Summary of pages

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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

BS 6349-1-2:2016+A1:2017 supersedes BS 6349-1-2:2016, which is withdrawn.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.

The start and finish of text introduced or altered by Corrigendum No. 1 and Amendment No. 1 is indicated in the text by tags **C1** **C1** and **A1** **A1**.

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.

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.

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*

[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 (m^2)
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 (m^2)
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	n th 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/s ²)
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
$\beta_{\max}$	dimensionless coefficient used in wave shoaling and breaking estimation
$\beta_{\max}^*$	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_1	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>i</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>i</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>i</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*

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		$\gamma_{G,sup}$	1.05	0.95	1.20	0.95	1.00	1.00
			$\gamma_{G,inf}$						
	Concrete self-weight		$\gamma_{G,sup}$	1.05	0.95	1.35	0.95	1.00	1.00
			$\gamma_{G,inf}$						
	Weight of soil and other materials		$\gamma_{G,sup}$	1.05	0.95	1.35	0.95	1.00	1.00
			$\gamma_{G,inf}$						
	Superimposed dead loads ^{B)}		$\gamma_{G,sup}$	1.05	0.95	1.20	0.95	1.00	1.00
			$\gamma_{G,inf}$						
	Geotechnical actions		$\gamma_{G,sup}$	1.05	0.95	1.35	0.95	1.00	1.00
			$\gamma_{G,inf}$						
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	—
	Uneven settlements		$\gamma_{G,sup}$	—	—	1.20 ^{D)}	—	1.00	—
			$\gamma_{G,inf}$						
	Prestressing		γ_p as defined in the relevant Eurocode or for the individual project						

Table 1 (continued)

Action type	Scenario	Item	Symbol (in accordance with BS EN 1990: 2002+A1)	EQU (Set A)	STR/GEO (Set B)	STR/GEO (Set C)
		Pedestrian loads (gr3)	γ_Q	1.35	—	1.15
Equipment loads ^{d)}		Gantry crane	γ_Q	1.35	—	1.15
		Mobile harbour crane	γ_Q	1.35	—	1.15
		Construction crane	γ_Q	1.35	—	1.15
		Port vehicles ^{e)} $\langle A_1 \rangle$	γ_Q	1.35	—	1.15
		Cargo handling equipment ^{e)} $\langle A_1 \rangle$	γ_Q	1.35	—	1.15
Cargo loads		Containers	γ_Q	1.50	—	$\langle C_1 \rangle$ 1.30 $\langle C_1 \rangle$
		General cargo	γ_Q	1.50	—	$\langle C_1 \rangle$ 1.30 $\langle C_1 \rangle$
		Bulk cargo	γ_Q	1.50	—	$\langle C_1 \rangle$ 1.30 $\langle C_1 \rangle$
		Liquid products	γ_Q	1.05	—	1.00
Environmental loads		Wind actions ^{g)}	γ_Q	1.50	—	1.30
		Operational wind (an environmental operating limit as defined in BS 6349-1-1: 2013)	γ_Q	1.50	—	1.30
		Thermal actions ^{g)}	γ_Q	1.50	—	1.20
		Snow	γ_Q	1.50	—	1.30
		Ice	γ_Q	1.50	—	1.30
		Water currents	γ_Q	1.50	—	1.30
		Wave	γ_Q	1.50	—	1.30

Table 1 (*continued*)

Action type	Scenario	Item	Symbol (in accordance with BS EN 1990: 2002+A1)	EQU (Set A)	STR/GEO (Set B)	STR/GEO (Set C)
		Operational wave (an environmental operating limit as defined in BS 6349-1-1: 2013)	γ_Q	1.50	—	1.30
		Tidal lag or variable ground water ^{c)}	γ_Q	1.50	—	1.30
Ship operational loads ^{d)}		Berthing (from characteristic energy) ¹⁾	γ_Q	1.35	—	1.15
		Berthing (from design energy) ¹⁾	γ_Q	1.20	—	1.10
		Mooring	γ_Q	See Table 8	—	See Table 8
		Ship ramps	γ_Q	1.50	—	1.30
		Ships propulsion	γ_Q	1.50	—	1.30
Construction loads (execution)			γ_Q	1.50	—	1.30
Geotechnical actions ¹⁾		Earth pressures	γ_Q	—	—	—

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.

A1) ^{F)} For port vehicles and cargo handling equipment, the vehicle loading needs to allow for either an appropriate impact factor on one axle only or braking/turning effects. Where away from obvious discontinuities or bumps, the impact factor for slow speed vehicles is to be taken as 1.25 on the most heavily loaded axle. Within 5 m of discontinuities or obvious bumps, the factor is to be increased such that it is 1.8 at the discontinuity or bump and reduces linearly to 1.25 at 5 m from the discontinuity of bump. For fast moving vehicles, an impact factor of 1.8 is to be used. Braking and turning effects on wheel loads can be assessed using the Interpave document, The structural design of heavy duty pavements for ports and other industries [28]. Braking/turning effects and impact factors do not need to be considered jointly. **A1)**

^{G)} 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).

^{H)} For partial factors related to loads on mooring structures, see 29.2, Table 8.

^{I)} 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.

¹⁾ 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	$\Sigma \gamma_{G,sup} G_{k,sup}$	$+ \gamma_p P$	—	$+ \gamma_{Q,1} Q_{k,1}$	$+ \Sigma \gamma_{Q,i} \psi_{0,i} Q_{k,i} \dots$	6.10
	Favourable	$\Sigma \gamma_{G,inf} G_{k,inf}$	$+ \gamma_p P$	—	$+ \gamma_{Q,1} Q_{k,1}$	$+ \Sigma \gamma_{Q,i} \psi_{0,i} Q_{k,i} \dots$	6.10
Accidental	Unfavourable	Either: $\Sigma G_{k,sup}$ or: $\Sigma G_{k,sup}$	$+ P$	$+ A_d$	$+ \psi_{1,1} Q_{k,1}$	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.11b
		$\Sigma G_{k,sup}$	$+ P$	$+ A_d$	$+ \psi_{2,1} Q_{k,1}$	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.11b
	Favourable	Either: $\Sigma G_{k,inf}$ or: $\Sigma G_{k,inf}$	$+ P$	$+ A_d$	$+ \psi_{1,1} Q_{k,1}^{C)}$	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.11b
		$\Sigma G_{k,inf}$	$+ P$	$+ A_d$	$+ \psi_{2,1} Q_{k,1}^{C)}$	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.11b
Seismic	Unfavourable	$\Sigma G_{k,sup}$	$+ P$	$+ A_{Ed}^{D)}$	—	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.12b
	Favourable	$\Sigma G_{k,inf}$	$+ P$	$+ A_{Ed}^{D)}$	—	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.12b
Accidental construction (execution)	Unfavourable	$\Sigma G_{k,sup}$	$+ P$	$+ A_d$	—	$+ \Sigma \psi_{2,i} Q_{k,i}^{E)}$	A2.2
	Favourable	$\Sigma G_{k,inf}$	$+ P$	$+ A_d$	—	$+ \Sigma \psi_{2,i} Q_{k,i}^{E)}$	A2.2
Serviceability limit states							
Characteristic	Unfavourable	$\Sigma G_{k,sup}$	$+ P$	—	$+ Q_{k,1}$	$+ \Sigma \psi_{0,i} Q_{k,i}$	6.14b
	Favourable	$\Sigma G_{k,inf}$	$+ P$	—	$+ Q_{k,1}$	$+ \Sigma \psi_{0,i} Q_{k,i}$	6.14b
Frequent	Unfavourable	$\Sigma G_{k,sup}$	$+ P$	—	$+ \psi_{1,1} Q_{k,1}$	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.15b
	Favourable	$\Sigma G_{k,inf}$	$+ P$	—	$+ \psi_{1,1} Q_{k,1}$	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.15b
Quasi-permanent	Unfavourable	$\Sigma G_{k,sup}$	$+ P$	—	—	$+ \Sigma \psi_{2,i} Q_{k,i}$	6.16b
	Favourable	$\Sigma G_{k,inf}$	$+ P$	—	—	$+ \Sigma \psi_{2,i} Q_{k,i}$	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)} $A_{Ed} = \gamma_1 A_{Ek}$ where the seismic effects are as defined in BS EN 1998.

^{E)} Q_{ck} 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_{cl} , 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, $d_{\text{set},i}$, 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.

- $d_{\text{set},i}$ 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 $d_{\text{set},i} \pm \Delta_{\text{dset},i}$ where $\Delta_{\text{dset},i}$ 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

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_{fwhk}	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

Table 3 (continued)

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)}
	Snow	0.80	0	0
	Ice	0.80	0.80	0
	Water currents	0.60	0.20	0
	Operational water currents ^{l)}	0.60	0.20	0
	Wave	0.60	0.20	0
	Operational wave	0.60	0.20	0
	Tidal lag ^{l)}	0.60	0.20	0
Operational loads	Berthing	0.75	0.75	0
	Mooring	0.50	0.20	0 ^{l)}
	Ship ramps	0.70	0.50	0
	Ships propulsion	0.75	0.75	0
Construction and installation loads		1.00	0	1.00

^{A)} Definitions of ψ factors are given in BS EN 1990:2002+A1, 4.1.

^{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 Projections (<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.

¹ Last accessed 17 June 2016.

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.

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.