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Eurocode 1 — Actions on structures — Part 1-8: Actions from waves and currents on coastal structures

Einführendes Element — Haupt-Element — Ergänzendes Element

Élément introductif — Élément central — Élément complémentaire

ICS:

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| CCMC will prepare and attach the official title page. |

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European foreword

This document (prEN 1991-1-8:2024) has been prepared by Technical Committee CEN/TC 250 “Structural Eurocodes”, the secretariat of which is held by BSI.

CEN/TC 250 is responsible for all Structural Eurocodes and has been assigned responsibility for structural and geotechnical design matters by CEN.

This document is currently submitted to the CEN Enquiry.

The first generation of EN Eurocodes was published between 2002 and 2007. This document forms part of the second generation of the Eurocodes, which have been prepared under Mandate M/515 issued to CEN by the European Commission and the European Free Trade Association.

The Eurocodes have been drafted to be used in conjunction with relevant execution, material, product and test standards, and to identify requirements for execution, materials, products and testing that are relied upon by the Eurocodes.

The Eurocodes recognise the responsibility of each Member State and have safeguarded their right to determine values related to regulatory safety matters at national level through the use of National An-nexes.

Introduction

**0.1 Introduction to the Eurocodes**

The structural Eurocodes comprise the following standards generally consisting of a number of parts:

* EN 1990, *Eurocode — Basis of structural and geotechnical design*
* EN 1991, *Eurocode 1 — Actions on structures*
* EN 1992, *Eurocode 2 — Design of concrete structures*
* EN 1993, *Eurocode 3 — Design of steel structures*
* EN 1994, *Eurocode 4 — Design of composite steel and concrete structures*
* EN 1995, *Eurocode 5 — Design of timber structures*
* EN 1996, *Eurocode 6 — Design of masonry structures*
* EN 1997, *Eurocode 7 — Geotechnical design*
* EN 1998, *Eurocode 8 — Design of structures for earthquake resistance*
* EN 1999, *Eurocode 9 — Design of aluminium structures*
* New Eurocodes under development, e.g. Eurocode for design of structural glass

The Eurocodes are intended for use by designers, clients, manufacturers, constructors, relevant author-ities (in exercising their duties in accordance with national or international regulations), educators, software developers, and committees drafting standards for related product, testing and execution standards.

NOTE Some aspects of design are most appropriately specified by relevant authorities or, where not specified, can be agreed on a project-specific basis between relevant parties such as designers and clients. The Eurocodes identify such aspects making explicit reference to relevant authorities and relevant parties.

**0.2 Introduction to EN 1991 (all parts)**

EN 1991 specifies actions for the structural design of buildings, bridges and other civil engineering works, or parts thereof, including temporary structures, in conjunction with EN 1990 and the other Eurocodes.

EN 1991 does not cover the specific requirements of actions for seismic design. Provisions related to such requirements are given in EN 1998 (all parts), which complement and are consistent with EN 1991.

EN 1991 is also applicable to existing structures for their:

* structural assessment,
* strengthening or repair,
* change of use.

NOTE In these cases additional or amended provisions can be necessary.

EN 1991 is applicable for the design of structures where materials or actions outside the scope of the other Eurocodes are involved.

NOTE In this case additional or amended provisions can be necessary.

EN 1991 is subdivided in various parts:

* EN 1991‑1‑1, *Eurocode 1 — Actions on structures — Part 1‑1: Specific weight of materials, self-weight of construction works and imposed loads for buildings*
* EN 1991‑1‑2, *Eurocode 1 — Actions on structures — Part 1‑2: Actions on structures exposed to fire*
* EN 1991‑1‑3, *Eurocode 1 — Actions on structures — Part 1‑3: Snow Loads*
* EN 1991‑1‑4, *Eurocode 1 — Actions on structures — Part 1‑4: Wind Actions*
* EN 1991‑1‑5, *Eurocode 1 — Actions on structures — Part 1‑5: Thermal Actions*
* EN 1991‑1‑6, *Eurocode 1 — Actions on structures — Part 1‑6: Actions during execution*
* EN 1991‑1‑7, *Eurocode 1 — Actions on structures — Part 1‑7: Accidental actions*
* EN 1991‑1‑8, *Eurocode 1 — Actions on structures — Part 1‑8: Actions from waves and currents on coastal structures*
* EN 1991‑1‑9, *Eurocode 1 — Actions on structures — Part 1-9: Atmospheric icing*
* EN 1991‑2, *Eurocode 1 — Actions on structures — Part 2: Traffic loads on bridges and other civil engineering works*
* EN 1991‑3, *Eurocode 1 — Actions on structures — Part 3: Actions induced by cranes and machines*
* EN 1991‑4, *Eurocode 1 — Actions on structures — Part 4: Silos and tanks*

**0.3 Introduction to EN 1991-1-8**

EN 1991-1-8 gives guidance for the determination of actions by waves and currents to be used for the design of structures in the coastal zone/area.

As wind is the governing physical parameter for wave generation on the sea surface and storm surge, i.e waves and design water levels are strongly correlated with wind, EN 1991-1-4 is particularly relevant in this respect.

**0.4 Verbal forms used in the Eurocodes**

The verb “shall" expresses a requirement strictly to be followed and from which no deviation is permitted in order to comply with the Eurocodes.

The verb “should” expresses a highly recommended choice or course of action. Subject to national regulation and/or any relevant contractual provisions, alternative approaches could be used/adopted where technically justified.

The verb “may" expresses a course of action permissible within the limits of the Eurocodes.

The verb “can" expresses possibility and capability; it is used for statements of fact and clarification of concepts.

**0.5 National annex for EN 1991-1-8**

National choice is allowed in this document where explicitly stated within notes. National choice includes the selection of values for Nationally Determined Parameters (NDPs).

The national standard implementing EN 1991-1-8 can have a National Annex containing all national choices to be used for the design of buildings and civil engineering works to be constructed in the relevant country.

When no national choice is given, the default choice given in this document is to be used.

When no national choice is made and no default is given in this document, the choice can be specified by a relevant authority or, where not specified, agreed for a specific project by appropriate parties.

National choice is allowed in EN 1991-1-8 through notes to the following clauses:

|  |  |  |  |
| --- | --- | --- | --- |
| 4.3.3(4) | 4.7.1(2) – 4 choices | 4.7.2(1) | 4.7.2(2) |
| 4.7.4(1) | 4.7.4(2) | 4.7.5(1) | 4.7.5(2) |
| 4.8(1) | 4.8(2) | 4.8(3) | 4.9.2(1) |
| 4.9.3(1) | 4.9.4(4) – 2 choices | 4.10(2) | 5.2.1(11) |
| 5.5.2(2) | 6.1.3(1) | 6.2(4) | 6.3.1(8) |
| 7.2.2(2) – 2 choices | 7.2.3(1) – 2 choices | 12.4.1.1(2) |  |

National choice is allowed in EN 1991-1-8 on the application of the following informative annexes:

|  |  |  |  |
| --- | --- | --- | --- |
| Annex A | Annex B | Annex C | Annex D |
| Annex E | Annex F | Annex G |  |

The National Annex can contain, directly or by reference, non-contradictory complementary information for ease of implementation, provided it does not alter any provisions of the Eurocodes.

# Scope

## Scope of EN 1991-1-8

(1) EN 1991-1-8 gives principles and rules to determine the values of wave and current actions on structures and civil engineering works in the coastal zone, i.e. works connected or in close vicinity to the shore.

NOTE 1 As opposed to offshore conditions, waves or currents in the coastal zone are substantially affected by the presence of seabed or shore.

NOTE 2 In typical seabed morphology the affected area lies between the shoreline and the deep-water limit.

(2) EN 1991-1-8 describes the principles for defining the design sea conditions, including design water level variability for structures in the coastal area.

NOTE Wave and current conditions vary for different construction sites. It is very important to assess the wave and current conditions at a given site. Assessment procedures for these conditions and for their uncertainties are included in this document.

(3) In EN 1991-1-8 the following structure types are specifically addressed:

* cylindrical structures;
* suspended decks;
* sub sea pipelines;

— breakwaters:

— mound breakwaters;

— vertical face breakwaters;

— composite breakwaters;

— coastal embankments:

— revetments;

— seawalls;

— permanently moored floating structures.

NOTE Additional guidance can be needed for:

— floating platforms in the coastal zone related to oil and gas production or processing;

— floating platforms in the coastal zone for renewable energy production.

(4) EN 1991-1-8 does not fully cover principles for determining actions from waves and currents on:

* port structures like piers, jetties, quay walls, marine terminals in sheltered marine areas;
* installations for mooring and berthing of ships.

(5) For hydraulic pressures on structural envelopes caused by quasi-static water levels, and for under pressures in the ground, see EN 1997 (all parts).

(6) EN 1991-1-8 does not cover:

* tsunamis;
* hydraulic consequences of an accidental breakdown of water actions from retaining structures;
* waves from passing ships;
* currents induced by jets or ship’s propellers;
* coastal structures where flood risk and/or erosion or sediment management is the dominant function.

## Assumptions

(1) The assumptions given in EN 1990 apply to this document.

(2) Qualification and experience related to this document refer also to the following fields:

1. physical coastal environment including physics of waves and currents, statistical properties and propagation of such;
2. marine hydrodynamics, wave and current disturbance by structures in general and wave and current actions on structures in the coastal zone including i) fixed structures, and ii) floating structures;
3. advanced methods including probabilistic methodology and physical model testing.

# Normative references

The following documents are referred to in the text in such a way that some or all of their content constitutes requirements of this document. For dated references, only the edition cited applies. For undated references, the latest edition of the referenced document (including any amendments) applies.

NOTE See the Bibliography for a list of other documents cited that are not normative references, including those referenced as recommendations (i.e. in ‘should’ clauses), permissions (‘may’ clauses), possibilities ('can' clauses), and in notes.

EN 1990:2023[[1]](#footnote-1), *Eurocode — Basis of structural and geotechnical design*

# Terms, definitions and symbols

## Terms and definitions

For the purposes of this document, the terms and definitions given in EN 1990 and the following apply.

### Terms relating to physical environment and environmental processes

3.1.1.1

shallow water

typically water depths less than one twentieth the wavelength

Note 1 to entry: Region of water in which waves propagate is often classified into three categories: deep water, intermediate depth, and shallow water. According to this classification, shallow water represents the zone of depth less than about one-twentieth of the wavelength.

Note 2 to entry: Intermediate (or transitional) water represents the zone of depth between about one-twentieth and one-half of the wavelength

3.1.1.2

deep water

water of such a depth that surface waves are little affected by bottom topography

Note 1 to entry: Intermediate (or transitional) water represents the zone of depth larger than about one-half the wavelength.

3.1.1.3

tide

astronomical tide

water movements that essentially are generated by the global response of oceans to celestial effects

Note 1 to entry: On the continental shelves and in coastal waters, particularly bays and estuaries, the effect is amplified by shallow water and coastal platforms.

3.1.1.4

storm surge

phenomenon of the rise of the sea surface above astronomical water level in the open coast, bays. and in estuaries due to the actions of wind stress and atmospheric pressure variations (static and dynamic)

3.1.1.5

foreshore

water zone of the shore between the mean low water mark (springs) and the mean high water level (springs)

Note 1 to entry: Definitions of foreshore can be different in some countries due to legislation or engineering practice.

3.1.1.6

surf zone

area between the outermost breaker and the limit of the wave run-up

3.1.1.7

wave climate

usually statistical description of wave conditions at a particular location over months, seasons, or years

Note 1 to entry: The wave climate is often expressed by the statistics of wave height; mean zero crossing or spectral peak wave period; and wave direction.

3.1.1.8

wave breaking

wave transformation process when the wave reaches a critical wave height, overturns on itself and breaks, dissipating energy

Note 1 to entry: Wave breaking can be due to water depth limitation or excess wind energy input.

3.1.1.9

wave set-up

rise of water level inshore of depth-induced wave breaking associated with water transport by wave action

Note 1 to entry: Wave set-up can amount to more than 10 % of the offshore significant wave height.

3.1.1.10

gravity wave

wave in a fluid or at the interface between two fluids for which the predominant restoring forces are gravity and buoyancy

EXAMPLE Wind-generated surface waves.

3.1.1.11

regular waves

waves with a single frequency, amplitude and direction

Note 1 to entry: Often the definition of regular waves is expanded to include waves of constant shape during propagation with some of them containing more than one determined spectral components, while the said definition refers only to monochromatic (or linear) waves.

3.1.1.12

irregular waves

waves with random wave periods (and in practice, also heights), which are typical for natural wind-induced waves

3.1.1.13

wind waves

waves being generated by and/or developed by wind

3.1.1.14

swell

swell waves

wind-generated waves of long period that have propagated out of their generating area and are not affected by wind actions anymore

Note 1 to entry: Swell characteristically exhibits a more regular and longer period and has flatter crests than waves within their generating area.

Note 2 to entry: 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.

3.1.1.15

internal wave

gravity wave which propagates within a stratified fluid

3.1.1.16

reflected wave

wave that carries seaward a part of the incident energy when a wave impinges on a beach, seawall or other reflecting surface

3.1.1.17

refraction

process by which the direction of a wave front moving in non-deep water at an angle to the depth contours is changed so that the wave crests tend to become more aligned with those contours

Note 1 to entry: The part of a wave front in shallower water moves slower than its associated part in deeper water, so when the depth along a wave crest varies, the wave “bends”.

3.1.1.18

tidal currents

alternating or circulating currents associated with tidal variation

Note 1 to entry: Tides and tidal currents are generally strongly modified by the coastline.

3.1.1.19

density driven currents

currents induced by horizontal gradients of water density

Note 1 to entry: Such currents can be generated by changes in the salinity and/or temperature caused by the influx of fresh water from land run-off as through an estuary heat flux from coastal power stations, or other reasons.

3.1.1.20

wind driven current

currents induced by wind stress on the sea surface

Note 1 to entry: In coastal waters, wind driven currents are influenced by the bottom topography and the presence of coastline.

3.1.1.21

gust

wind agitation caused by a steep rise and fall in wind speed lasting for less than one minute

Note 1 to entry: The wind speed of a gust can be averaged, for design purposes, over longer time periods.

3.1.1.22

wind set-up

rise of water level at the downwind side of a bounded water body caused by wind stresses on the water surface

Note 1 to entry: In coastal waters wind set-up can be developed over the continental shelf for winds blowing against the coastline.

3.1.1.23

tsunami

long gravity waves with period of several minutes to one hour and wavelength of some kilometres, generated by a sudden vertical movement of the sea floor associated to submarine earthquake, by a land slide or volcanic eruption, and other causes

Note 1 to entry: Tsunami wave heights at the coast can be up to a few tens of metres.

### Terms relating to analysis of metocean parameters

3.1.2.1

annual maximum method

method of estimating extreme wave heights or water levels, based on a sample of annual maximum wave heights or water levels

3.1.2.2

Peak-over-Threshold method

POT method

method of estimating extreme parameter values applied to wave heights, based on a sample of peak values/heights of storm waves exceeding some threshold level

3.1.2.3

total sample method

method of estimating wave heights exhibiting unconditional exceedance probabilities, by considering all wave heights measured at the site

3.1.2.4

extreme value

random quantity that can be considered in a semi-probabilistic analysis by a representative value

Note 1 to entry: Extreme events are used in the verification of the non occurrence of limit states.

3.1.2.5

marginal return period of the value *x* of a random variable *X*

*T*(*x*)

inverse of the probability that the random variable *X* is larger than *x*, i.e. T(x) = 1/Prob (*X* > *x*)

Note 1 to entry: The unit of *T*(*x*) is the average duration between two consecutive outcomes of *X*.

3.1.2.6

joint return period of the couple (*x*, *y*) of random variables (*X*, *Y*)

*T*(x, y)

inverse of the probability that the random variable *X* is larger than *x* and simultaneously that the random variable *Y* is larger than *y*, i.e. T(x,y) = 1 / Prob (X>x and Y>y)

Note 1 to entry: The unit of *T*(*x*, *y*) is the average duration between two consecutive and assumed to be simultaneous outcomes of *X* and *Y*.

3.1.2.7

*n*th spectral moment

integral over frequency of the spectral density function multiplied by the n-th power of the frequency, either expressed in Hertz (cycles per second) as  or expressed in circular frequency (radians/second) as 

Note 1 to entry: As *ω* = 2*πf*, the relationship between the two moment expressions is: *mn*(*ω*) = (2*π*)*n* *mn*(*f*).

Note 2 to entry: The integration extends over the entire frequency range from zero to infinity. In practice the integration is often truncated at a frequency beyond which the contribution to the integral is negligible and/or the sensor no longer responds accurately. Care is needed when utilizing moments of order higher than 2, as for standard spectral models, the 4th moment will not converge; the value is in effect determined by the choice of truncation.

3.1.2.8

wave spectrum

function expressing the energy density distribution of the sea surface variance in the frequency domain and for all directions

Note 1 to entry: Usually, the term wave spectrum is used for the frequency-energy spectrum determined from linear Fourier transform.

Note 2 to entry: For the generation of representative sea states, 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 3 to entry: The area under the wave spectrum is the zero-th spectral moment *m*0, which is a measure of the total energy in the sea state and can be used to calculate spectral wave height *H*m0.

3.1.2.9

directional wave spectrum

function expressing the energy density distribution of waves in the frequency and directional domains

Note 1 to entry: This function is often expressed as the product of frequency wave spectrum and the directional spreading function.

3.1.2.10

spectral peak period

period at the maximum (peak) energy density in the spectrum

Note 1 to entry: In practice, in addition to the maximum peak there are often secondary peaks in a spectrum.

3.1.2.11

two-dimensional model

2D model

mathematical or physical model in which the flow parameters vary in two spatial dimensions

3.1.2.12

three-dimensional model

3D model

mathematical or physical model in which the flow parameters vary in three spatial dimensions

3.1.2.13

directional spreading function

function expressing the relative distribution of wave energy in the directional domain

3.1.2.14

extreme waves

waves in a storm event that can be considered in a semi-probabilistic analysis by representative values based on adequate statistical definitions

Note 1 to entry: In case of a storm event, short term statistics are used to express extreme waves in terms of the significant wave height or the maximum wave height or any other short term statistical parameter thereof, and the mean or significant wave period at the peak of storm event.

3.1.2.15

random waves

superposition of a number of regular waves with different frequencies and amplitudes

Note 1 to entry : The term is often used to denote the laboratory simulations of irregular waves that occur in nature.

3.1.2.16

seiche

standind oscillation of the water surface of an enclosed or semi-enclosed body of water at its natural period

3.1.2.17

level

elevation

vertical distance from a stated geodetic datum

### Terms relating to statistical metocean parameters

3.1.3.1

metocean event

storm event defined by its sea state conditions

Note 1 to entry: A metocean event is usually expressed by short-term distributions of the parameters still water level, wave height, wave period and wave direction, while other metocean parameters can be also included at specific cases.

3.1.3.2

datum level

reference level for survey, design, construction and maintenance of coastal structures, often set at a chart datum or national geodetic datum

3.1.3.3

chart datum

locally assigned reference datum related to navigation charts

Note 1 to entry: Chart datum is usually an approximation to the level of the lowest astronomical tide.

3.1.3.4

international marine chart datum

chart datum set at the lowest astronomical tide level, as adopted by the International Hydrographic Organization (IHO)

3.1.3.5

lowest astronomical tide

LAT

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

Note 1 to entry: LAT is not reached every year and does not represent the lowest sea level which can be reached, because storm surges (negative) and tsunamis can cause considerably lower levels to occur.

Note 2 to entry: LAT level is approximately realized every 19 years.

3.1.3.6

highest astronomical tide

HAT

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

Note 1 to entry: HAT is not reached every year and does not represent the highest sea level that can be reached, because storm surges and tsunamis can cause considerably higher levels to occur.

Note 2 to entry: HAT level is approximately realized every 19 years.

3.1.3.7

still water level

undisturbed water level SWL

theoretical water surface level that exists in the absence of wave action, but taking into account other effects as needed such as wind and atmospheric pressure influences

Note 1 to entry: 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 to entry: 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.

3.1.3.8

water depth

vertical distance between the sea floor and still water level

Note 1 to entry: As there are several options for the still water level, there can be several water depth values. Generally, design water depth is determined to LAT or to mean sea level.

Note 2 to entry: The water depth used for calculating wave kinematics varies between the maximum water depth of the highest astronomical tide plus a positive storm surge, and the minimum water depth of the lowest astronomical tide less a negative storm surge, where applicable. The same maximum and minimum water depths are applicable to bottom-founded and floating structures, although water depth is usually a much less important parameter for floating structures. Water depth is, however, important for the design and analysis of the mooring system and risers for floating structures.

3.1.3.9

mean sea level

MSL

long-term value of water surface level calculated from observations over a complete astronomical tidal cycle of 18,6 years

Note 1 to entry: The simplest way to calculate MSL is to take the arithmetic average of hourly (or three-hourly) observations, but more elaborate methods include the application of low-pass numerical filters to eliminate tides and surges before taking the average.

Note 2 to entry: The average of all high and low water levels called the mean tide level, is close to but not identical with mean sea level.

Note 3 to entry: 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.

3.1.3.10

mean water level

**MWL**

average elevation of the water surface over a given time period

Note 1 to entry: The mean water level is usually determined from hourly tidal level readings.

Note 2 to entry: The monthly mean water level can vary over seasons by a few tens of centimetres.

3.1.3.11

mean high water springs

MHWS

average level of high waters occurring at the time of spring tides measured over an astronomical tidal cycle

3.1.3.12

mean low water springs

MLWS

average level of low waters occurring at the time of spring tides, measured over an astronomical tidal cycle

3.1.3.13

extreme high water

highest sea level predicted 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

3.1.3.14

extreme low water

lowest level that is predicted 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

3.1.3.15

extreme water level

EWL

Statistically determined highest or lowest water level, by the combination of astronomical tide, storme surges, and climate change influence, referenced to a fixed datum

3.1.3.16

design water level

water level selected for design and analysis of coastal structures, in view of the acceptable level of risk of failure/damage due to an extreme event of a defined return period

Note 1 to entry: Generally, it is the water level that mostly affects the safety of the structures/facilities in question.

3.1.3.17

sea state

condition of sea surface within a short time span, being expressed by still water level, wave heights, wave periods and wave directions

Note 1 to entry: The time span normally used is from 20 minutes up to a few hours.

Note 2 to entry: The length of time span is chosen as the result of compromise between the requirement of a short duration to guarantee that the sea state is constant during the recording time and that the duration is sufficiently long with necessary number of waves for reliable statistical estimates of characteristic wave heights and wave periods.

3.1.3.18

wave height

vertical distance between a trough and the preceding (following) crest, as determined by zero-down (zero-up) crossing

3.1.3.19

wavelength

horizontal distance between two successive zero-down or zero-up crossings in a wave record

3.1.3.20

wave number

inverse of the wavelength times 2*π*

3.1.3.21

wave period

time for two successive zero-down or zero-up crossings to pass a fixed point

3.1.3.22

wave phase velocity

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

3.1.3.23

wave steepness

ratio of wave height to wavelength, *H*/*L*

3.1.3.24

wave crest

highest point of a wave profile

3.1.3.25

maximum wave height

highest wave height

*H*max

height of the highest wave in a given wave record

3.1.3.26

*T*-year wave height

extreme wave height of a sea state corresponding to the return period of *T* years

Note 1 to entry: In applications this height is usually the significant wave height or the maximum wave height.

3.1.3.27

significant wave height

*H*s

*H*1/3

mean wave height of the highest third of the waves of a given wave record determined from time-domain analysis, *H*s = *H*1/3

Note 1 to entry: Often the spectral wave height *H*m0 is used instead of *H*s, determined from frequency-domain analysis of the wave record as function of the zero-th spectral moment *m*0 of the Fourier spectrum. In deep water *H*1/3 and *H*m0 are in general very close in value and are both considered good estimates of *H*s, but in intermediate and shallow waters, *H*1/3 and *H*m0 estimates may differ substantially.

3.1.3.28

significant wave height estimated from wave spectrum

*H*m0

spectral wave height determined from the zero-th spectral moment of the Fourier spectrum of the wave record, 

3.1.3.29

representative wave height

wave parameters selected for design and analysis of coastal structures, chosen in view of the acceptable level of risk of failure/damage due to an extreme event of a defined return period

3.1.3.30

storm-representative wave

statistical outcome of short-term distribution of the wave heights in a sea state

EXAMPLE The significant wave height and the maximum wave height.

Note 1 to entry: The storm-representative wave represents an engineering abstraction, typically expressed in terms of a regular wave used for calculation of quasi-static wave actions on a structure.

3.1.3.31

reflection coefficient

ratio of the height of reflected wave to the height of incident wave

3.1.3.32

refraction coefficient

ratio of the height of waves having been affected by the refraction effect to their height in deep water, with the shoaling effect being eliminated

3.1.3.33

stability number

ratio *H*s/Δ*D*n

3.1.3.34

mean wave period

*T*m

average period of all zero crossings of waves in a given record by application of a time-domain analysis

Note 1 to entry: Instead of *T*m the spectral mean wave periods *T*m-1,0 or *T*m0,2 can be used. These frequency-domain parameters are obtained by the moments of the Fourier transform of the wave record.

3.1.3.35

peak period

peak wave period

*T*p

wave period determined by the inverse of the frequency at which the wave energy spectrum reaches a maximum

3.1.3.36

significant wave period

average period of the one-third highest waves of a given wave record in time-domain analysis

Note 1 to entry: Mean (from time-domain analysis) or peak wave (from frequency-domain analysis) 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.

3.1.3.37

representative wave period

wave period statistically determined by short term distribution

EXAMPLE The significant wave period or the maximum wave period in a sea state.

3.1.3.38

residual current

part of the total current not belonging to the tidal stream, hence not constituted from harmonic tidal components

Note 1 to entry: Residual currents are caused by a variety of physical mechanisms and comprise a large range of natural frequencies and magnitudes in different parts of the world.

### Terms relating to metocean effects in interaction with structures

3.1.4.1

air gap

vertical distance measured from the still water level/top of the wave crest up to the bottom of the deck structure

3.1.4.2

wave overtopping

passing of water over the top of a structure as a result of wave run-up or surge action

3.1.4.3

diffraction

diffraction of waves in a process by which wave energy is transmitted laterally along wave crests

Note 1 to entry: This process results in bending of wave fronts and propagation of diffracted waves into sheltered areas, such as the breakwater shadow.

3.1.4.4

diffraction coefficient

ratio of the height of diffracted waves to the height of incident waves

3.1.4.5

reflection

process by which part of the wave energy is returned seaward when waves impinge on an obstacle

Note 1 to entry: Waves reflect from seawalls, breakwaters, but also from beaches and at locations with sharp depth changes.

3.1.4.6

run-up/run-down

process of waves running up and down a beach or a coastal structure, also their upper and lower levels reached relative to still water level

3.1.4.7

hydrodynamic loads

quantity representing the action from currents and waves on coastal structures

EXAMPLE Pressures, forces, moments, tensions flow velocities, wave overtopping discharges.

3.1.4.8

slamming actions

actions developed when a water surface and a structure suddenly collide

3.1.4.9

transmission

process of wave energy passing over and through a (low) crested structure and generating waves behind the structure

3.1.4.10

wave pressure

water pressure exerted on a structure induced by waves, excluding hydrostatic pressure

3.1.4.11

buoyancy

resultant of upward forces, exerted by the water on a submerged or floating body, equal to the weight of the water displaced by this body

3.1.4.12

uplift force

upward water pressure in the pores of a material or the base of a structure due to waves, excluding buoyancy

3.1.4.13

drag coefficient

*C*D

coefficient used to determine the drag force induced by currents/waves

3.1.4.14

lift coefficient

*C*L

coefficient used to determine the lift force induced by currents/waves

3.1.4.15

inertia coefficient

*C*M

coefficient used to determine the inertia force induced by currents/waves

3.1.4.16

scour

removal/erosion of underwater sand and stone material by shear forces associated with waves and currents, especially critical at the base or toe of a structure

3.1.4.17

hydrodynamic estimate approach

HEA

methodology to assess metocean parameters that relates to the consequence class of the structure and the local hydrodynamic conditions

3.1.4.18

design approach

DA

collective description of the design path, based on the HEA level and the structure design and response uncertainty, to verify whether a structure complies with reliability requirements

3.1.4.19

importance factor

*φ*I

multiplier of the characteristic and design marginal and joint return periods of metocean events allowing for the consequence class of the structure

3.1.4.20

dynamic amplification factor

DAF

dimensionless number which multiplied by responses caused by static load accounts for dynamic loading

3.1.4.21

empirical (design formulae)

design formulae based on research using full scale and/or model tests

3.1.4.22

vortex induced vibration

VIV

vibration induced by vortexes shed alternatively from either side of a cylinder in a current and/or waves

### Terms relating to coastal structures

3.1.5.1

breakwater

structure protecting a shore area, harbour, anchorage, intake or outfall from waves and currents

3.1.5.2

armour layer

outer layer composed of armour units protecting breakwaters, embankments or other rubble mound structures against waves and currents

3.1.5.3

armour unit

relatively large quarry stone or concrete shaped unit selected to fit specified geometric dimensions, density and strength requirements

3.1.5.4

crown wall

concrete superstructure on a rubble mound breakwater

3.1.5.5

roundhead

circular-shaped head of a breakwater

Note 1 to entry: Often a roundhead is reinforced by heavier armour units and/or reduced slope than in the trunk.

3.1.5.6

toe

lowest part of a coastal structure, providing often support for slope protection and forming the transition to the seabed

3.1.5.7

trunk

longitudinal section of a breakwater or similar structure, excluding the roundhead(s)

3.1.5.8

embankment

collectively representation of revetments and seawalls

3.1.5.9

seawall

protective structure against coastland erosion and wave overtopping, built along the shoreline with nearly vertical seaward face inducing wave reflection

3.1.5.10

revetment

protective structure against coastland erosion and wave overtopping, built along the shoreline with sloping seaward face inducing wave breaking

3.1.5.11

floating structure

permanently moored floating object

3.1.5.12

jetty pier

deck structure supported by vertical or near-vertical piles projecting into the sea, frequently in a direction normal to the coastline

3.1.5.13

free span

pipeline section not supported by the seabed

3.1.5.14

slender structures or members

structural members that are geometrically slender. i.e. with small ratio of a characteristic cross sectional dimension to length

3.1.5.15

moorings

system to hold a floating object in position by annulling excessive movements

3.1.5.16

anchors

units placed on the seabed, such as ship anchors, piles driven into the seabed or concrete blocks, to which mooring lines are attached to restrain a floating object from excessive movements

3.1.5.17

berm

horizontal step in the sloping profile of an embankment or breakwater

3.1.5.18

core

inner portion of a breakwater, embankment and other rubble mound structure, often with small permeability

3.1.5.19

crest

highest point of a coastal structure

3.1.5.20

filter

intermediate layer, preventing fine materials of an underlayer from being washed through the voids of the upper layer

3.1.5.21

geotextile

synthetic fabric used as filter in earthworks

Note 1 to entry: Geotextiles can be woven or non-woven.

3.1.5.22

stone

quarried or artificially broken rock for use in construction, either as aggregate or cut into shaped blocks as dimension stone

3.1.5.23

riprap

usually, wide-graded quarry stone, bulk placed to prevent erosion

3.1.5.24

lee

shelter or sheltered side from wind or waves provided by neighbouring natural or artificial structures or installations

3.1.5.25

marine growth

additional layer on subsurface structures formed by marine plants, colonies of animals, etc.

3.1.5.26

permeability

capacity of bulk material in permitting movement of water through its pores

Note 1 to entry: Bulk materials can be sand, crushed rock and soft rock in situ.

3.1.5.27

pore pressure

interstitial pressure of water within a mass of soil or rock

3.1.5.28

scale or physical model

simulation of a structure and its (hydraulic) environment, usually in much smaller dimensions, to enable the study of the structure behaviour under specified conditions

Note 1 to entry: The model can be built with a fixed or a movable bed.

## Symbols and abbreviations

### Latin upper-case letters

|  |  |
| --- | --- |
| *A* | Vibration amplitude |
| *A*e | Eroded area around SWL |
| *A*p | Projected surface area |
| *B* | Width [length] |
| *B*(*γ*) | Normalizing factor |
| BEM | Boundary element method |
| *C* | Celerity |
| *C* | Non-dimensional force coefficient |
| *C*D | Drag coefficient |
|  | Drag coefficient corresponding to maximum of fluctuating in-line forces (drag forces) |
|  | Drag coefficient corresponding to maximum of fluctuating in-line forces (drag forces) in steady flow |
| *C*DC | Drag coefficient with respect to current |
| *C*DS | Drag coefficient for steady flow |
| *C*L | Lift coefficient |
|  | Lift coefficient corresponding to maximum of transversal forces (lift forces) |
|  | Lift coefficient corresponding to maximum of transversal forces (lift forces) in steady flow |
| *C*LS | Lift coefficient for steady flow |
| *C*M | Inertia coefficient |
| *C*a | Added mass coefficient |
| CFD | Computational fluid dynamics |
| *D* | Diameter or characteristic cross-sectional dimension |
| *D*(*θ|f*) | Directional spreading function |
| *D*er | Percentage of eroded volume |
| *D*n | Nominal block diameter, *D*n=(*M*/*ρ*app)1/3 |
| *D*n50 | Median nominal diameter of armour stones |
| *D*0 | Normalizing constant |
| DA | Design approach |
| DAF | Dynamic amplification factor |
| Ds-EM | Deep-sea extremes methods |
| *F*c | Current drag force |
| *F*D | Drag force [force] |
| *F*G | Buoyancy reduced weight of caisson [force] |
| *F*h(t) | Horizontal force time history |
| *F*H | Horizontal wave force [force] |
| *F*I | Inertia force [force] |
| *F*s | Slamming force [force] |
| *F*U | Wave induced uplift force [force] |
| *F*ν(t) | Vertical force time history |
| FTA | Full transfer approach |
| GBS | Ground based structure |
| *H* | Wave height |
| *H*b | Breaking wave height |
| *H*i | Height of incoming wave of normal incidence |
| *H*max | Maximum wave height |
| *H*m0 | Significant wave height extracted from the spectrum |
| *Hn* | Representative wave heights of wave group *n* |
| *Hq* | Storm-representative wave height corresponding to an annual exceedance probability of *q* |
| *H*s | Significant wave height, *H*s = *H*1/3 |
| *H*s,toe | Significant wave height at structure toe |
| *H*s0 | Deep-water significant wave height |
|  | Significant wave height with return period *T* |
| *H*1/3 | Significant wave height based on time domain analysis |
| *H*2% | Wave height exceeded by 2 % of waves |
| HAT | Highest astronomical tide |
| HEA | Hydrodynamic estimate approach |
| JDs-EM | Joint deep-sea extremes methods |
| *K*D | Stability coefficient in Hudson’s formula |
| *K*s | Stability parameter |
| *KC* | Keulegan-Carpenter number =*u*max*T/D (=πH/D;* in deep water*),* |
| *L* | Length |
| *L*r | Model scale |
| *Lo* | Wavelength in deep water |
| LAT | Lowest astronomical tide |
| *M* | Mass |
| *Ma*z | Vertical added mass |
| *Ma*x | Lateral added mass |
| MDs-EM | Marginal deep-sea extremes method |
| MHWS | Mean high water springs |
| MLWS | Mean low water springs |
| MSL | Mean sea level |
| *N* | Number of data points |
| *N*d | Ratio of number of units moved out of place |
| *N*f | Number of simulations to failure |
| *N*od | Damage number expressed by the relative number of displaced units out of armour layer |
| *P*f | Probability of failure |
| *P*f,*L* | Failure probability level for *L* years service life |
| *P*f,*L,T* | Failure probability level for *L* years service life and action return period *T* |
| *R* | Radius of a cylinder |
| *Re* | Reynolds number = (*u*max*D*)/ϑ, where *u*max is the maximum orthogonal local particle velocity |
| *R*u | Maximum run-up or water-surface elevation measured vertically from the still-water level [length] |
| *R*u,2% | Run-up level achieved by only 2 % of the incoming waves at the toe of the structure |
| *S*d | Damage parameter expressed by the non-dimensional eroded area |
| *S*(*f*) | Spectral density function |
| SDi | Severe displacement ultimate limit state |
| *Sp* | Directional spreading |
| *St* | Strouhal number |
| SWL | Still water level (no waves or wave effects) |
| *T* | Return period |
| *T*lf | Design service life [years] |
| *T*m | Mean wave period (time-domain) |
| *T*m0,2 | Spectral wave period estimated from the zero-th and second moments of wave spectrum (frequency-domain) |
| *T*m-1,0 | Spectral mean energy wave period, *T*m-1,0 = *m*-1/ *m*0 |
| *T*p | Period corresponding to the peak of the wave spectrum (energy frequency spectrum) |
| *T*s | Significant wave period |
| *T*z | Zero crossing wave period |
| *T(x)* | Marginal return period of the value x of a random variable *X* |
| *T(x, y)* | Joint return period of the couple (*x*, *y*) of random variables (*X*, *Y*) |
| TLP | Tension leg platform |
| *U* | Current velocity |
| *U*C | Mean current velocity |
| *U*w | Wave particle velocity |
| *U*0 | Current velocity |
| *V* | Relative velocity of water surface to that of the surface of a member |
| *V* | Group waves celerity |
| *V*r | Reduced velocity |
| VIV | Vortex induced vibrations |
| WL | Water level |

### Latin lower-case letters

|  |  |
| --- | --- |
| *a*x | Horizontal acceleration |
| *a*0 | Distance between highest water level and bottom of deck |
| *c* | Instantaneous wetted height |
| *c*d | Apparent soil cohesion |
| *c*u | Undrained shear strength of clay [force/length2] |
| *e* | Relative surface roughness |
| *f* | Frequency |
| *f* (1) | Linear force based on linear wave theory for a regular wave and complete reflection off a wall |
| *f* (2) | Non-linear mean force |
| *fj* | Natural frequency of vibration for the *j*-th mode of a structural member |
| *f*p | Frequency at spectral peak |
| *f*v | Vortex shedding frequency |
|  | Joint probability function of multidimensional variable *X* taking value |
| *g* | Acceleration due to gravity |
| *h* | Water depth [length] |
| *h*s | Water depth in front of a structure |
| *h*S | Threshold level |
| *k* | Surface roughness height |
| *k*nl | Non-linear wave number |
| *l* | Characteristic length of a structural member |
| *m* | Dependence parameter of the Gumbel-Hougaard copula |
| *m*e | Effective virtual mass per unit length |
| *m*n | *n*th moment of wave spectrum |
| pdf | probability density function |
| *q* | Annual probability of exceedance |
| *r* | Dolos waist ratio [dimensionless] |
| *s* | Spreading parameter |
| *s*max | Peak spreading parameter |
| *t* | Thickness |
| *u* | Local water particle velocities normal to the member axis |
|  | Local water particle accelerations normal to member axis |
| *u*\* | Friction velocity |
| *u*max | Maximum orthogonal local particle velocity |
| *w* | Vertical velocity underneath the deck |
|  | Velocity of member normal to its axis positive in the wave direction |
|  | Acceleration of member normal to its axis positive in the wave direction |
| *z* | Vertical motion of a structure |
| *z*s | Stretched vertical coordinate |
| *z*0 | Bed roughness length |

### Greek upper-case letters

|  |  |
| --- | --- |
| *Π*R | Ursell number |
| *Δ*mod | Relative density in model |
| *Δ*pro | Relative density in prototype |
| Δ*SL* | Sea level variation |
| *Ψ* | Wake amplification factor |

### Greek lower-case letters

|  |  |
| --- | --- |
| 𝛼 | Weibull distribution scale parameter |
| 𝛼 | Coefficient |
| 𝛽 | Reliability index |
| 𝛽 | Current flow velocity ratio |
| 𝛽s | Weibull distribution shape parameter |
| 𝛾 | Peak enhancement factor |
| 𝛾F | Partial factor for actions |
| 𝛾Sd | Partial factor associated with the uncertainty of the action and/or action effect model |
| *δ* | Generalized logarithmic decrement |
| *δ*h | Hydrodynamic damping |
| *δ*soil | Soil’s damping or other damping |
| ζa | Amplitude of the wave |
| *η* | Wave surface elevation |
| *η*b | Maximum elevation of the free surface |
| *η*max | Maximum crest height |
| *θ* | Wave direction |
| *θ*m | Mean wave direction |
| *κ* | van Karman’s constant, *κ* = 0,4 |
| 𝜆 | Wavelength |
| 𝜆c | Curling factor |
| 𝜆nl | Non-linear wavelength |
| 𝜇 | Mean value |
| 𝑣 | Kinematic viscosity |
| 𝑣° | Angle of friction in granular material [degrees] |
| *ρ*app | Applied mass density of block material |
| *ρ*arm | Density of armour block |
| *ρ*mg | Density of marine growth and biofouling |
| *ρ*mod | Density of model |
| 𝜌𝑤 | Apparent mass density of water |
| 𝜎 | Standard deviation |
| 𝜎 | Spectral width parameter |
| *ση* | Standard deviation of the surface elevation |
| *σθ* | Angular standard deviation |
| 𝜑 | Packing density coefficient |
| 𝜑I | Importance factor |
| *ψ*0 | Combination factor applied to a variable action to determine its combination value |
| *ψ*1 | Combination factor applied to a variable action to determine its frequent value |
| *ψ*2 | Combination factor applied to a variable action to determine its quasi-permanent value |

# Basis of wave and current action assessment

## General

(1) Actions from waves and currents shall be represented by appropriate hydrodynamic loads.

## Design approaches

### General

(1) The design approach, which sets out the recommended requirements in terms of the assessment that should be applied to estimate the action or action effect arising from the marine environment, should be selected following the selection of the hydrodynamic estimate approach (HEA).

NOTE 1 For the hydrodynamic estimate approach, see 4.6(1) – 4.6(3).

NOTE 2 For the selection of design approach, see 4.6(4).

### Semi-probabilistic design approach

(1) In the semi-probabilistic approach the representative values of hydrodynamic loads due to the actions from waves and currents shall be assessed following the indications given in this Clause 4.

NOTE 1 See EN 1990:2023, 1.1(4) and EN 1990:2023, C.3.1(3).

NOTE 2 In this document, the semi-probabilistic design approach is denoted DA1.

### Reliability-based design approach

(1) The available metocean data and the available environmental sea conditions shall suit the probabilistic method used to implement the reliability-based design approach in accordance with EN 1990.

NOTE In this document, the reliability-based design approach is denoted DA2.

(2) Where a sufficient length of record of environmental sea conditions is available considering the design service life of the coastal structure and the target value for the reliability index *β*, one of the following probabilistic methods may be used:

* using a statistical summary or a sample of environmental deep-sea conditions, e.g. a scatter table or a matrix of schematized conditions (possibly above/over a specified threshold) yielding structure’s responses from which the relevant probability of failure can be identified;
* using series of structural responses based on a representative sample of environmental sea conditions at the vicinity of the structure and to which it is exposed (possibly above/over a specified threshold), or on a full-time series, from which the relevant probability of failure can be identified.

(3) Where only a limited duration record of environmental sea conditions is available, extrapolation of data can provide values of the important parameters for lower chances of occurrence (longer return periods), through appropriate fitting of extreme value distributions and extrapolation to the required return period using a valid mathematical method, e.g. a Monte-Carlo simulation.

NOTE Extrapolated data are also called “synthetic data”.

(4) The validity of the extrapolation method shall be verified.

NOTE Further guidance and recommendations relating to application of reliability-based design approach can be found in Clause 13 and Annex H.

### Risk-informed decision-making design approach

(1) The available metocean data of the required environmental sea conditions shall suit the probabilistic method used to implement the risk-informed decision-making design approach in accordance with EN 1990.

NOTE In this document, the risk informed decision-making design approach is denoted DA3.

(2) The same provisions related to the probabilistic methods specified in 4.2.3 for the assessment of wave and current actions shall apply.

NOTE The risk-informed decision-making design approach enables to determine the optimal probabilities of failure or the optimal safety indexes on a case-by-case basis.

### Design assisted by physical testing

(1) Hydraulic physical model tests may be used to validate the assessment of the wave and current actions undertaken in accordance with a relevant design approach (DA1 to DA3), considering the design service life and the consequence class of the coastal structure.

NOTE 1 In this document, the design assisted by physical testing is denoted DA4.

NOTE 2 DA4 is not a separate design approach but an option to be considered alongside DA1 to DA3.

NOTE 3 Further guidance and recommendations concerning application of physical models to coastal structures is provided in Clause 12 and Annex G.

NOTE 4 Further guidance on design assisted by testing is given in EN 1990.

## Action modelling

### Classification of actions from waves and currents

(1) Actions from waves and currents shall be considered as variable free dynamic actions.

(2) Actions on structural parts induced by the global dynamic response of floating or fixed structures subjected to waves and currents, shall be considered as direct actions.

(3) Unless a dynamic analysis is carried out, actions from waves and currents on fixed coastal structures may be modelled by equivalent quasi-static actions, for the duration of the design metocean event.

NOTE 1 Dynamic analysis are often carried out in the design of floating structures and often include dynamic wind action analysis.

NOTE 2 Quasi-static water-levels and their variations with space and time (e.g. storm surges) play an important role in the action of waves and currents (they influence wave heights, wave periods, current velocities…, and the propagation of metocean events to the structure’s site) and cause hydraulic pressures. They are not considered as a separate action.

### Metocean parameters

(1) Metocean events shall be represented by a set of metocean parameters.

NOTE Metocean parameters are addressed in Clause 5.

### General methods for the assessment of the hydrodynamic loads

(1) Hydrodynamic loads should comprise several of the following: wave or current-induced pressures and forces, forces/moments along structural parts of the structure, including foundations, tensions on mooring lines, including dynamic effects, wave conditions (height, period, direction) or flow velocities at the toe of a coastal structure or at other locations, mean wave overtopping discharges (acting as a functional requirements of coastal structures or as an indirect load of parts of coastal structures) see Clauses 6 to 11.

NOTE Non-dimensional stability numbers comprise action-related parameters and material (resistance)-related parameters: the wave stability number or Hudson number, the Isbash number (when considering the current velocity), the Shields parameter (when considering the shear stress caused by a moving fluid), etc. They appear in hydraulic limit state conditions.

(2) Hydrodynamic loads should be calculated either with the Deep-Sea Extremes Methods (Ds-EM), whose sub-divisions are the Marginal Deep-sea Extremes Method (MDs-EM) and the Joint Deep-Sea Extremes Method (JDs-EM), or with the Full transfer approach (FTA).

(3) For Deep-Sea Extremes Methods (Ds-EM) the following steps should be performed:

* selection of a small number of sets of offshore (deep-sea) metocean parameters according to the rules explained in 4.7 through statistical analysis using either marginal (Marginal Deep-sea Extremes Method – MDs-EM) or joint (Joint Deep-Sea Extremes Method – JDs-EM) probabilistic distributions of the metocean parameters when available;
* transfer of the selected sets to the site of the (coastal) structure;
* identification of the design values of the hydrodynamic loads as the most adverse responses of the structure coming out of the selected sets of metocean parameters.

(4) For the full transfer approach (FTA) the following steps should be performed:

* transfer of a sufficiently large set of the most extreme metocean events of the dataset to the site of the (coastal) structure;
* identification of the design values of the hydrodynamic loads based on the extreme value analysis of the distributions and/or frequency distributions of the responses of the structure.

NOTE The National Annex can select the method or provide guidance for the choice of a method.

## Design situations

(1) Actions from waves and current shall comply with the physical environment entailed by the design situations for which the structure is verified.

NOTE Persistent design situations include not only the normal “everyday use” conditions but also the severe conditions experienced infrequently, as caught up by the return periods used for the assessment of the design values of the action.

(2) Persistent design situations should be defined to allow for the variations of still water level.

NOTE 1 It can be useful to define a set of design situations commanded by the water levels, each of those situations being relevant for a given part of the structure for example, the toe of a rubble mound breakwater may be designed under a low water level situation, the main armour blocks under a mean water level situation, the crest and the harbour-side slope under a high-water level situation.

NOTE 2 Water levels influence wave heights, wave periods, current’s velocities, etc., and the propagation of metocean events to the structure’s site.

(3) Specific persistent design situations should be defined to allow for changing climate and sea level conditions.

NOTE It may be useful to define persistent design situations under the climate and sea level conditions prevailing at the beginning and at the end of design service life of the structure.

(4) Specific metocean events, hence specific actions from waves and currents, shall be considered during temporary environmental influence, and during the execution and maintenance phases, making several transient design situations.

NOTE 1 The coastal environment can involve sub surface construction works and subsea construction works that influence construction procedures, structural design, material selection and size (dimensions, volume).

NOTE 2 Examples of transient design situations are the towing of a caisson from a dry dock to the structure’s site, the account for intermediate breakwaters’ heights during construction.

(5) A fatigue design situation may be defined to allow for the repetition of load cycles by waves and currents.

NOTE Several equivalent static hydrodynamic loads can be used for the verification of fatigue limit state.

(6) Tsunamis caused by earthquakes should be considered in the seismic design situation.

(7) Wave and current actions may be neglected in a design situation provided that a sufficient air gap or ground space is provided between sea water and structure.

(8) Extreme metocean events much more severe than the ones described for persistent and transient design situations should be considered in the accidental design situation.

## Geometrical parameters

(1) Due care should be taken for possible effects of scouring, long term morphological developments, liquefaction, settlement and deformations of the structure, its protecting elements and its foundation both due to wave and current actions, and by other causes.

(2) Geometrical parameters intervening in the waves and currents action models shall take into account the design service life of the structure and the design situation to ensure that the structure provides the required level of performance over its entire service life.

NOTE 1 Geometrical parameters include structure-related parameters as well as near-structure topographic and bathymetric parameters (slope, seabed, etc.).

NOTE 2 Due to the interaction between environmental marine conditions and structure responses, the final geometrical properties of a structure and of the bathymetry may be the result of an iterative assessment process.

(3) The predictable time evolution of geometrical parameters shall be accounted for by considering their most adverse values likely to occur within the design situation, regarding the hydrodynamic loads and the limit states to be checked.

NOTE The most adverse values of the geometrical parameters correspond often to the end-of-service life conditions. However, this is not always true: for example, the steeper the slope of a berm breakwater, the higher the run-up.

(4) In particular, wave overtopping and resulting flooding should be analysed based on the end-of-service life conditions including allowance for settlement and climate change.

## Hydrodynamic estimate approaches

(1) The consequence class of the structure and the hydrodynamic uncertainty on the site shall be considered when determining the minimum Hydrodynamic estimate approach (HEA) level from Table 4.1.

Table 4.1 — HEA level selection matrix

|  |  |  |  |
| --- | --- | --- | --- |
| Consequence class | Hydrodynamic uncertainty | | |
|  | Low | Medium | High |
| CC1 | HEA1 | HEA1 | HEA2 |
| CC2 | HEA1 | HEA2 | HEA3 |
| CC3 | HEA2 | HEA3 | HEA3 |

(2) Hydrodynamic uncertainty shall be assessed considering the environmental sea conditions at the site of interest, the quality of the metocean data and the complexity of the local physical processes (boundary condition generation and transformation to the site).

EXAMPLE 1 Examples of low hydrodynamic uncertainty can include tidal range < 1 m, surge < 0, 5m, fetch-limited seas (with fetch < 10 km), uniform currents with spring tide velocities < 1 m/s, regular bathymetry.

EXAMPLE 2 Examples of high hydrodynamic uncertainty can include tidal range > 5 m, surge > 2 m, ocean seas (swell and wind-waves), non-uniform currents (stratified) and/ or tide or surge current velocities > 3 m/s, irregular bathymetry (e.g. reefs or sub-sea canyons).

(3) The HEA levels are defined in Table 4.2 (NDP).

Table 4.2 (NDP) — Hydrodynamic Estimate Approach methodology guidance

|  |  |  |  |
| --- | --- | --- | --- |
| HEA level | Boundary condition data | Pathway assessment | Other considerations |
| HEA1 | Locally or nationally determined metocean parameters data, mainly from published sources, with limited probabilistic definition or limited reliability information attached. | Wind-wavea hindcast using empirical calculations taking into account fetch/ duration limitations.  Extrapolation of sea-level or current values from published values for nearby sites. | Data unlikely to be calibrated or validated for developed statistical treatments. Expertise is required. |
| HEA2 | Individual (marginal) long-termb time-series data (or statistical distributions/ summaries thereof) of metocean parameters.  Measured data, or, if not available, calibrated and validated model data, e.g. regional or global hindcast models, or synthetic data. | Numerical wave transformationc model representing (with reasonable accuracy) all key physical processes expected.  Adjustment of statistically estimated sea-level or current values to account for site-specific physical processes, either by empirical or numerical model. | Data likely to be used for the statistical assessment of the representative values of the actions from waves and currents (hydrodynamic loads), including the combination of metocean parameters, based on marginal statistics. |
| HEA3 | Joint long-termb time-series data (or statistical distributions/ summaries thereof) of metocean parameters.  Measured data, or, if not available, calibrated and validated model data, e.g. regional or global hindcast models, or synthetic data. | Calibrated numerical wave/ water level/ current transformationc model representing (with reasonable accuracy) all key physical processes expected. | Data likely to be used for the statistical assessment of the representative values of the actions from waves and currents (hydrodynamic loads), including the combination of metocean parameters, based on joint statistics. |
| a When the predominant wave energy system only is analysed and can be expected to be wind-sea dominated.  b «Long-term» is to be understood in relation to the design service life and to the return periods of the metocean events, also considering the quality of data available (filtering/ removal of erroneous records).  c This is based on the typical situation where long-term boundary condition data is not available in close proximity to the structure’s site and must be transformed from a remote location. | | | |

(3) Following selection of the hydrodynamic estimate approach (HEA), the design approach may be selected from Table 4.3 which sets out the recommended requirements in terms of the assessment that should be applied to estimate the action or action effect arising from the marine environment.

Table 4.3 — Design approach selection matrix

|  |  |  |
| --- | --- | --- |
| **HEA level** | **Low-to medium structure design/ response uncertainty**a | **High structure design/ response uncertainty**a |
| HEA1 | DA1 | Not applicable |
| HEA2 | DA1 or DA2b | DA1 + DA4 or DA2 + DA4 |
| HEA3 | DA1 or DA2 or DA3 or any previous with DA4 | DA1c + DA4 or DA2 + DA4 or DA3 + DA4 |
| a Where the term «uncertainty» is defined in Note 3.  b See Clause 13 and Annex H for DA2 methods.  c More advanced design approaches than DA1 should be used at HEA3 level especially with highest structure design/response uncertainty.  NOTE 1 As explained in 4.2:   * DA1: Semi-probabilistic partial factors approach (loads and resistances) with appropriate sensitivity testing of key parameters based on the application of semi-empirical structure response formulae; * DA2: Probabilistic (reliability based) approach with allowable probabilities of failure or *β* indexes; * DA3: Risk-informed approach with a socio-economic optimisation to determine optimum probability of failure of the considered structure; * DA4: Design assisted by testing approach, in combination with DA1, DA2 or DA3.   NOTE 2 Guidance and limitation of use of DA2 and DA3 are given in EN 1990:2023, C.3.1.  NOTE 3 The term «structure uncertainty» is associated mainly with the structure (and sub-element) type and excludes uncertainty pertaining to the hydrodynamic (sea) conditions at the structure’s site. Low structure uncertainty can, for example, apply where the physical processes/ response mechanisms are relatively simple and/ or there is an established and validated structural analysis approach, whereas high structure uncertainty may apply where the physical processes/response mechanisms are complex and/ or there are several analytical methods available giving widely varying results and/or the conditions are significantly outside the application limits of an established structural analysis approach. | | |

NOTE Whatever the design approach, physical tests and numerical models based on Computational Fluid Dynamics (CFD) or other fluid-structure simulation methods are widely used for assessing wave and current actions to coastal structure as well as the structure’s response. Further guidance and recommendations concerning their application and calibration is provided in Clauses 6 to 11 and in Annexes D to F.

## Representative values of hydrodynamic loads

### General

(1) Representative values of hydrodynamic loads shall account for the uncertainty of the occurrence of and variability of metocean events.

NOTE 1 The distribution of individual wave heights and individual wave periods within metocean events (i.e. during storms) is sometimes directly accounted for in action models by calibration or through statistical and/or spectral descriptions. Wave heights mentioned in semi-empirical design formulae are therefore specified quantiles of the wave height short-term statistical distribution, according to the action model and to the type of structure. This is why the uncertainty associated with individual wave distributions (short term statistics) is not detailed in this subclause (see Clauses 5 to 11).

NOTE 2 The metocean events governing the design can depend on the limit state to be checked and on the part of the structure under consideration.

(2) Local or regional variations due to climate and sea level changes shall be taken into account when determining representative values of hydrodynamic loads.

NOTE 1 For the determination of representative hydrodynamic loads for a coastal structure a climate change scenario can be chosen that is based on recent IPCC (International Panel on Climate Change) projections, or local refinements thereof. From these climate scenarios a medium emission scenario, RCP4.5 (Representative Concentration Pathway with a possible range of radiative forcing values in the year 2100 of 4,5 W/m2), can be applied together with the most likely of all model outcomes, unless the National Annex describes another method.

NOTE 2 For non-adaptable and non-replaceable structures or parts thereof, the most adverse metocean parameters’ random processes during the design service life of the structure are considered, according to the climate change scenario in Note 1, unless the National Annex describes another method.

NOTE 3 For adaptable or replaceable structures or parts thereof, provided a maintenance and monitoring plan is explicitly addressed, the most adverse climate and sea level until the first major planned maintenance, are considered according to the climate change scenario in Note 1, unless the National Annex describes another method.

NOTE 4 In case the maintenance schedule is not known, the climate change scenario in Note 1, and the marine conditions (waves, currents, winds, water levels, etc.) for 2/3 of the intended design service life of the structural parts are considered, unless the National Annex describes another method.

NOTE 5 Further considerations on climate change are given in Clause 5.

(3) When a statistical data analysis of the metocean events is performed, representative values of hydrodynamic loads should be assessed using return periods or time-exceedance frequencies of metocean parameters that are representative of metocean events (i.e. using long term statistics).

(4) Taking into account a shorter duration of transient design situations, shorter return periods than those suited for persistent design situations may be defined.

(5) In the Deep-sea Extremes Method, when there is clear evidence of a moderate or greater degree of correlation between primary variables of metocean parameters, a joint probability assessment of the metocean parameters should be performed as long as this is compatible with the available data. The characteristic value of the hydrodynamic load should be based on the characteristic joint return period introduced in 4.7.2(2) and on the combination marginal return period introduced in 4.9.3.

NOTE When there is a very strong correlation, a full correlation can be assumed and fitting a joint probability distribution is not necessary anymore.

(6) In the Deep-sea Extremes Method a marginal (individual) probability assessment of the metocean parameters may be performed when:

* there is a little degree of correlation between primary variables of metocean parameters; or
* the structure’s response can be proved not to be sensitive to combined waves, currents and water levels; or
* the available data do not allow for a joint probability assessment.

(7) The characteristic value of the hydrodynamic load should be based on the characteristic marginal return period introduced in 4.7.2(1) and on the combination marginal return period introduced in 4.9.2.

(8) The representative value of a favourable/stabilizing action shall be the minimum value likely to occur in the design situation under consideration.

NOTE The minimum value is often equal to zero.

(9) In the Full Transfer Approach, representative values of the hydrodynamic loads on structural elements (i.e. the characteristic value, the frequent value, the quasi-permanent value) should be assessed with identical marginal return periods of the loads’ own distributions as those given in 4.7.2, 4.7.4 and 4.7.5.

### Characteristic value

(1) The value of the characteristic marginal return period shall be assessed.

NOTE The characteristic value is assessed as the 100-year marginal return period (the “characteristic marginal return period”) unless the National Annex gives a different value.

(2) When a two-variables joint statistical analysis of the metocean parameters is performed, the value of the characteristic joint return period shall be assessed.

NOTE 1 The characteristic value is assessed as the 350-year joint return period (the “characteristic joint return period”) unless the National Annex gives a different value.

NOTE 2 The value of the characteristic joint return period for use in a three- or more variables statistical analysis is not covered in this document.

(3) When the physical processes are relatively simple/benign so that statistical analyses might not be needed (HEA1), the characteristic value may be the best estimate of the extreme value with the return period mentioned in (1) and (2).

### Combination value

(1) The combination value of the hydrodynamic loads with non-metocean actions shall be found by multiplying the characteristic or the design value by the *ψ*0 factors defined in EN 1990.

### Frequent value

(1) The frequent value shall be assessed:

* as a 0,02 % frequency of time exceedance; or
* as a 5-year marginal return period (the “frequent marginal return period”); or
* when a two-variables joint statistical analysis of the metocean parameters is performed, as a 15‑year joint return period (the “frequent joint return period”).

NOTE 1 The National Annex can select the method or provide guidance for the choice of a method for the assessment of the frequent value.

NOTE 2 The value of the frequent joint return period for use in a three- or more variables statistical analysis is not covered in this document.

(2) When the physical processes are relatively simple/benign so that statistical analyses might not be needed (HEA1), the frequent value may be:

* the best estimate of the value with the return period or the frequency of time exceedance that are mentioned in (1); or
* calculated by multiplying the characteristic value by the *ψ*1 factor defined in EN 1990.

NOTE The National Annex can select a method or provide guidance for the choice of a method.

### Quasi-permanent value

(1) The quasi-permanent value shall be assessed:

* as a 0,1 % frequency of time exceedance; or
* as a 1-year marginal return period (the “quasi-permanent marginal return period”); or
* when a two-variables joint statistical analysis of the metocean parameters is performed, as a 3-year joint return period (the “quasi-permanent joint return period”).

NOTE 1 The National Annex can select one method or provide guidance for the choice of a method for the assessment of the quasi-permanent value.

NOTE 2 The value of the quasi-permanent joint return period for use in a three- or more variables statistical analysis is not covered in this document.

(2) When the physical processes are relatively simple/benign so that statistical analyses might not be needed (HEA1), the quasi-permanent value may be:

* the best estimate of the value with the return period or the frequency of time exceedance that are mentioned in (1); or
* calculated by multiplying the characteristic value by the *ψ*2 factor defined in EN 1990.

NOTE The National Annex can select one method or provide guidance for the choice of a method.

(3) The quasi-permanent value of the still water level shall be the mean sea level and take into account the sea level rises due to climate change.

## Design value and importance factor

(1) The design value used in the verification of serviceability and ultimate limit states shall be assessed by multiplying the importance factor *φ*I with the characteristic marginal return period defined in 4.7.2(1) and the characteristic joint return period defined in 4.7.2(2).

NOTE The values of the importance factor *φ*I are given in Table 4.4 (NDP) unless the National Annex gives a different value.

(2) The design value used in the verification of the ultimate limit states shall be calculated with partial factors on action *γ*F as defined in EN 1990:2023, Table A.6.8 (NDP) or assessed directly (i.e. without partial factors).

NOTE 1 Water levels are never factored. Hydrostatic pressures can be factored, see Clauses 6 to 11.

NOTE 2 Time independent uncertainties due to hydrodynamic load action models are covered by means of partial factors given in EN 1990:2023, Table A.6.8 (NDP), design formulae, especially those based on model tests, and the limit state’s allowable design criteria.

NOTE 3 The National Annex can provide guidance on how to assess directly the design value of the action from currents and waves for use in the verification of ultimate limit states, using specific design marginal and joint return periods.

(3) For the verification of ultimate limit states of rubble mound breakwaters (such as the severe displacement ultimate limit state (SDi) of protecting armour units or the severe wave overtopping ultimate limit state), the design value of the hydrodynamic load should be determined directly as a nominal value according to a high design return period, without partial factor, see Clause 7.

Table 4.4 (NDP) — Importance factor *φ*I for the verification of serviceability and ultimate limit states

|  |  |
| --- | --- |
| Consequence class | Value of importance factor  *φ*I |
| CC3 | 2,0 |
| CC2 | 1,0 |
| CC1 | 0,5 |

NOTE 1 The importance factor *φ*I given in Table 4.4 (NDP) can be adjusted to the design service life of the coastal structure (denoted as *T*lf), as soon as *T*lf ≥ 20 years, taking the *φ*I value from Table 4.4 (NDP) x *T*lf (in years)/50, unless the National Annex describes a different method.

NOTE 2 *T*lf = 50 years is a common value for coastal structures, see EN 1990:2023, Clause A.6.

(4) The larger variability of the dynamic part of the tension in a mooring line may be accounted for by multiplying a model factor *γ*Sd with the partial factor on action *γ*F.

NOTE See Clause 11 for further details.

## Specific combinations rules for metocean parameters

### General provisions

(1) For the sake of practicability in the calculation of hydrodynamic loads, a limited number of metocean parameters can be selected in the implementation of Tukstra’s combination procedure, the other parameters being deduced through correlations with the former (if correlation is proven or reasonably to be expected).

(2) When the inputs of the hydrodynamic load or action formula involve non-metocean parameters, the leading metocean parameter may be combined with accompanying non-metocean parameters varying within a reasonable expected range.

NOTE Non-metocean parameters can be geometrical parameters.

(3) The set of values of leading and accompanying metocean parameters shall be physically realistic.

NOTE 1 Following Turkstra’s combination rule, the most adverse induced effect, after transfer to the structure’s site, coming out of the alternate leading and accompanying positions of the metocean parameters is kept for the design of the structure, according to the limit states to be checked.

NOTE 2 In particular, in the water levels’ assessment, the wave and wind set-up and the pressure anomaly are considered in consistency with the wave and wind conditions of the sea state.

(4) Combinations should refer to offshore (deep-sea) metocean parameters, i.e. before the transfer to the structure’s site.

(5) Combinations may be carried out in the water level design situations defined in 4.4(2).

(6) To calculate the characteristic value of the hydrodynamic load, the characteristic value of the leading metocean parameter shall be combined with the combination value of the accompanying ones.

(7) When the physical processes are relatively simple/benign so that statistical analyses might not be needed (HEA1), the combination value of the accompanying metocean parameters shall be appraised as an estimate of the return period values mentioned hereafter.

### Combination rules using marginal distributions of the metocean parameters (marginal deep-sea extremes method)

(1) If a marginal probabilistic analysis of the metocean events is performed to calculate the characteristic value of the hydrodynamic load the leading metocean parameter shall be assessed with the characteristic marginal return period given in 4.7.2(1) multiplied by the importance factor *φ*I given in Table 4.4 (NDP).

NOTE 1 The marginal return period of the accompanying metocean parameters (the “combination marginal return period”) is given in Table 4.5 (NDP) according to the level of dependence with the leading metocean parameter, in each high/low water level design situation, unless the National Annex gives a different value.

NOTE 2 As an example of the marginal deep-sea extremes method (in the verification of a serviceability limit state of a CC2 structure), with medium dependence between wave height and still water level, the 100-year offshore (deep-sea) wave is considered together with the 10-year still water level as calculated in the low WL situation or in the high WL situation; the 100-year still water level calculated in the low WL situation or in the high WL situation is considered together with the 10-year offshore (deep-sea) wave. Those metocean events are transferred to the site of the (coastal) structure and the characteristic value of the hydrodynamic load can be calculated with Turkstra’s rule.

### Combination rules using joint distributions of the metocean parameters (joint deep-sea extremes method)

(1) If a joint probabilistic analysis of the metocean events is performed to calculate the characteristic value of the hydrodynamic load the following steps should be performed.

* Plot the relation between the metocean parameters’ values whose combinations yield the characteristic joint return period given in 4.7.2(2) multiplied by the importance factor *φ*I given in Table 4.4 (NDP).
* Enter the value of the accompanying metocean parameters (the “combination marginal return period”).
* Read the value of the leading metocean parameter using the relation mentioned in the first bullet point, as the value associated to the accompanying metocean parameter.

NOTE 1 The marginal return period of the accompanying metocean parameters (the “combination marginal return period”) is given in Table 4.5 (NDP) according to the level of dependence with the leading metocean parameter, in each high/low water level design situation, unless the National Annex gives a different value.

NOTE 2 As an example of the joint deep-sea extremes method (in the verification of a serviceability limit state of a CC2 structure) with medium dependence between wave height and still water level: plot the relation between wave heights and water levels at 350-year joint return period; consider the 10-year (marginal return period) still water level as calculated in the low WL situation or in the high WL situation, together with the wave height red on the 350-year joint return period relation; consider also the 10-year (marginal return period) wave height together with the water level as calculated in the low WL situation or in the high WL situation, red on the 350-year joint return period relation; transfer those metocean events to the site of the coastal structure and calculate the hydrodynamic load with Turkstra’s rule.

NOTE 3 Monte Carlo simulation techniques can be useful for the assessment of the joint distribution of metocean events to the extreme values needed for the design.

Table 4.5 (NDP) — Synthesis of return periods and importance factors in the determination of the characteristic hydrodynamic loads in permanent/transient design situations

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Conse-quence class | Importance factor (*φ*I)a | Characteristic marginal return period (leading metocean parameter)b | Combination marginal return period (accompanying metocean parameter)c | Characteristic joint return periodd |
| CC3 | 2,0 | 200 y | Very High dependence: 140 y | 700 y |
| High dependence: 40 y |
| Medium dependence: 13 y |
| Low dependence: 6 y |
| CC2 | 1,0 | 100 y | Very High dependence: 70 y | 350 y |
| High dependence: 25 y |
| Medium dependence: 10 y |
| Low dependence: 5 y |
| CC1 | 0,5 | 50 y | Very High dependence: 40 y | 175 y |
| High dependence: 15 y |
| Medium dependence: 8 y |
| Low dependence: 4 y |
| a The importance factor is given in Table 4.4 (NDP).  b The characteristic marginal return period is equal to the value given in 4.7.2(1) multiplied by the importance factor.  c When the degree of dependence is not known, the “medium dependence” figure can be used.  d The characteristic joint return period is equal to the value given in 4.7.2(2) multiplied by the importance factor. Figures are valid for a two-variable joint analysis only. | | | | |

NOTE 4 The degree of dependence can be appraised through the value of the parameter (*m*) of the Gumbel-Hougaard copula fitting the joint dataset of the two metocean parameters:

* *m* larger than 1,20: very high dependence;
* *m* between 1,10 and 1,20: high dependence;
* *m* between 1,05 and 1,10: medium dependence;
* *m* lower than 1,05: low dependence.

### Specific combination rules between waves, currents and wind

(1) Swell and wind waves should be addressed through differentiated statistical analysis.

(2) The representative values of the wind parameters may be determined directly by considering an appropriate correlation function with the primary metocean parameters.

NOTE For further guidance, see A.2.2.4.

(3) Wind should be combined assuming strong correlation of the non-fluctuating part with wind waves unless other combination principles are technically justified, such as in the presence of swell.

(4) Dynamic or fluctuating parts of wind and waves in the design metocean events may be assumed to be uncorrelated.

NOTE 1 When the deep-sea extremes method is used, if swell and wind waves are assumed to be uncorrelated, only the combination rules stated in 4.9.2 is applied (i.e. with marginal distributions) by considering a three-parameters combination (i.e. waves, wind and currents, or waves, wind and water levels) instead of a two-parameters combination (i.e. waves and water levels), unless the National Annex describes a different value.

NOTE 2 As an example of the marginal deep-sea extremes method (in the verification of a serviceability limit state of a CC2 structure) with medium dependence, the 100-year marginal offshore (deep-sea) wave is considered together with the 10-year marginal current velocity and the 10-year marginal wind velocity ; the 100-year marginal current velocity is considered together with the 10-year marginal offshore (deep-sea) wave and the 10-year marginal wind velocity; the 100-year marginal wind velocity is considered together with the 10-year marginal current velocity and the 10-year marginal offshore (deep-sea) wave. One or more values of the still water level are assessed in each of those three combinations so as to be physically consistent with those three primary parameters. All of these metocean events are transferred to the site of the (coastal) structure and the characteristic value of the hydrodynamic load can be calculated with Turstra’s rule.

NOTE 3 The characteristic joint return period has not been calibrated for a three- or more variables joint distribution, unless cared for in the National Annex.

(5) The combination rule stated above can be applied to the assessment of the hydraulic load as a tension in a mooring line of a floating structure subjected to swell, currents, still water level variations, wind waves and wind blowing directly on the floating structure.

## Accidental metocean events

(1) In case accidental metocean events are considered, their value should be determined either:

* as a nominal value, on a case-by-case basis, considering events standing out of the ordinary statistics, outliers, freak rogue waves, extreme storms and surges, or allowing for some extreme issues of climate change; or
* with a probabilistic analysis based on available data.

(2) If a probabilistic analysis is performed to calculate the accidental value of the hydrodynamic load due to accidental metocean events, the offshore (deep-sea) metocean parameters shall be determined as the “accidental marginal return period”, independently of the consequence class and the design service life of the structure.

NOTE The accidental marginal return period is 10 000 years unless the National Annex gives a different value.

# Hydrodynamic conditions

## General

### Metocean design description

(1) All metocean variables that affect the hydrodynamic actions on coastal structures shall be included in the metocean design description.

NOTE Further guidance on hydrodynamic conditions is given in Annex A.

(2) Primary variables that shall be included in the metocean design description are wave height, wave period, water level and current velocity.

(3) Covariates that shall be included in the metocean design description are wave direction, spectral width, directional spreading, storm (peak) duration, current shear stress, current direction, current turbulence intensity, wave setup.

NOTE Covariates are often derived from primary variables through correlations when not assessed as peak values or most probable values given the primary variables (this can be also the case for wave periods).

(4) If relevant, wind conditions may be included in the metocean design description: wind speed, wind direction, wind gust, wind setup.

(5) Other relevant environmental conditions may need to be taken into account in the design, e.g. sea temperature, marine growth, ice, rainfall.

### Metocean data

(1) Metocean data shall be selected considering the conditions under which the structure has to meet its requirements.

NOTE The consideration can include the potential for long waves to be experienced at the site, i.e. long period wave motions associated with wave grouping, e.g. where a series of high waves follows a series of low waves.

(2) Available metocean data should be considered with respect to data quality, frequency and duration of the records and the parameters provided.

(3) The consideration of available metocean data should also include a comparison of hydrodynamic processes at the location where the data is collected and at the site.

(4) Metocean data should be based on long-term statistics as well as short-term statistics of storm events.

(5) The long-term statistics shall include annual probabilities of metocean variables for derivation of relevant return period values as required in Clause 4.

### Wave and current interactions with structures

(1) The influence of structures on waves and currents at the site should be considered, where relevant.

NOTE Wave induced fluid particle velocities for calculation of hydrodynamic actions can be strongly affected by structures in close proximity.

## Design event probability and extreme values analysis

### General

(1) Evaluation of hydrodynamic actions at the site should be made with due consideration of dependence between design parameters, e.g. wave height (period and direction) and water level at the site.

(2) Dependence between design parameters should be modelled by joint probabilities of occurrence.

(3) Wave data obtained at a location where the largest wave height is limited by the water depth should not be used for extreme statistical analysis of wave conditions in water deeper than the site of the data used.

(4) For HEA2 and HEA3 hydrodynamic conditions to be used for design should be established based on extreme value statistical analysis.

(5) Statistics of extreme sea conditions should be established on the basis of instrumentally measured and/or hindcast data, coupled with necessary transformation analysis to represent key physical processes that can influence conditions between the location of measurement/ hindcast data and the site of the structure.

(6) Extreme value analysis of wave/current conditions should consider directionality and should be carried out for appropriate sectors corresponding to the wave/current generation/transformation processes in the region or site of interest.

(7) For some applications it may be suitable to use the omni-directional extreme values.

(8) The duration of the wave data set should take into account the HEA level.

NOTE The duration is typically 15 – 30 years. Shorter duration data set than 15 years can be applied where annual variations are accounted for.

(9) Hindcast data should be calibrated with measured data close to the site.

(10) Caution should be taken for the water depth at which waves have been measured (or estimated), taking into account that shallow water depth imposes an upper physical limit to the largest possible wave height due to wave breaking.

(11) When a statistical data analysis of the metocean events is performed:

* a Bayesian statistical approach (combining the sample uncertainty and the stochastic uncertainty in a single safe value) should be carried out for estimating the representative values;
* otherwise, the representative values should be central statistical estimates (CSE) or upper/lower bound of the confidence interval of the return period values or of time-exceedance frequency values.

NOTE 1 In order to address the sample uncertainty, the representative value can be a central statistical estimate, an upper or a lower value accounting for the uncertainty due to the limited size of the data sample, or a nominal value, depending on the requirements of the action and resistance model adopted for the design.

NOTE 2 Semi-empirical action and resistance formulae often incorporate a safety margin to address the scatter uncertainty, commonly one Standard Deviation (SD) above the CSE.

NOTE 3 The reliability of any extrapolated record can be highly dependent on how representative the distribution of conditions in the limited record is of longer-term conditions.

NOTE 4 The procedure to be used is as given in (11) unless the National Annex gives a different procedure.

### Extreme value analysis

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

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

(2) An appropriate probability distribution should be selected that provides a good fit to the data.

(3) The confidence interval of the extrapolated values should be determined, and the design should take account of the sensitivity to the range of the extrapolated values.

(4) The Peaks-Over-Threshold (POT) method should be the preferred method to produce the data set of extreme values.

(5) To apply the POT method, suitable threshold and event separation values should be determined.

(6) The annual maximum method may be employed.

(7) The use of the total sample method (initial distribution method) where all data are accounted may be employed when the data set is short.

NOTE The POT and the annual maximum methods retain only a smaller subset of the initial data to obtain a dataset that is Independent and Identically Distributed (IID), while the initial distribution method utilizes all available data (dependent data).

(8) The wave period associated with the extreme wave height should be determined by applying joint distributions of wave height and associated wave period for extreme waves.

(9) Where relevant, the highest wave height and the significant wave height corresponding to a given return period should be estimated from the result of extreme statistical analysis.

NOTE The highest wave height can be estimated by converting the latter to the former on the basis of the Forristall distribution of individual wave heights (in deep water) and the wave transformation analysis.

(10) For shallow water with wave breaking wave height distributions for shallow water should be applied.

(11) When the combination of metocean parameters, e.g. wave height and water level, are important in the design of coastal structures, it should be established from the available data whether large wave heights and high-water levels tend to be dependent or independent of one another.

NOTE Further guidance relating to extreme value analysis, probability density functions and joint probability of wave and water-level conditions is given in CIRIA publication C68. Information relating to probability density functions for application to independent analyses is given in Annex H.

(12) The degree of dependency should be represented in terms of joint probabilities of exceedance.

NOTE 1 Other relevant combinations of metocean parameters are wave height and wave period, wave orbital velocities and currents or water levels and currents.

NOTE 2 In the absence of sufficient data for establishing whether or not high-water levels and large wave heights (or other parameters) are correlated, medium dependence can be assumed.

## Water levels

### Design water level

(1) A clear definition of the vertical datum to which water levels are stated and the relationship to any other datums used for the design should be provided.

NOTE The design water level at the site of interest can be influenced by a range of components, primarily including astronomical tides, meteorological surges and wave set-up, but also potentially by other influences including for example: fluvial/estuarial flows, tidal currents, set-up/set-down due to wind and long waves (including seiches or tsunamis), and in the long-term by climate change effects. The principal components most commonly constituting the design water level for most coastal structures are addressed here, namely tides, surges and wave set-up, but all components and influences relevant to the site of interest should be assessed.

(2) The influence of wave-structure interactions on the design water level should be taken into account during design, where appropriate.

(3) Over long (decadal) timescales, changes in mean sea level should be taken into account, and in short-term (several days) cases, water level changes due to storm surges (positive or negative) and wave set-up at the shoreline should be considered.

(4) Available long-term water level measurements suitable for the site should be reviewed for quality and continuity, and an assessment should be made of the need to separate tide and surge components to better understand water level characteristics at the site.

NOTE Some national hydrographic offices provide a wealth of information relating to astronomical tide levels, tide predictions and water level measurements.

### Water level measurements

(1) Where available and if the quality and reliability of the data can be assured, measured water level data should be preferred to synthetic data of any kind for water level information.

NOTE 1 In some locations however, the duration over which measured data is available is generally insufficient to establish the long-term extreme water level distribution.

NOTE 2 Measurements from oceanographic instruments, e.g. tide gauges, are the most accurate means of recording water levels, but are sometimes limited to a few sites, and can be reviewed for record gaps or data anomalies due to instrument malfunction.

NOTE 3 The availability of existing measured data can be researched before commissioning new measurements.

### Tides

(1) The astronomical tide levels at the site should be based on published levels from a relevant authority or an international navigation organisation.

NOTE Where published tidal levels are not available in sufficient proximity to the site, tidal levels can be calculated with the tidal constituents obtained through the harmonic analysis of tide records at the site or those estimated from a nearby tide station.

(2) The extreme (high and low) water levels at or near the site should be taken into account in evaluation of the actions from waves and currents.

(3) The vertical datum for the tidal contribution and total water levels should be clearly defined with reference to the International Marine Chart Datum and/or the national geodetic datum levels.

(4) Long-term changes to mean sea level (MSL) due to geological and/ or climate change processes shall be considered.

(5) Their effect on other standard levels such as mean high (MH) and mean low (ML) water levels, as well as highest and lowest astronomical tides should be taken into account.

### Surges

(1) The characteristics of surges at the site should be duly investigated and be taken into consideration in evaluation of the actions from waves and currents.

NOTE Surges are due to barometric pressure effects. Low-pressure weather systems, often associated with storms, can cause water levels to rise, while high atmospheric pressure can lower the water level.

(2) Other effects such as wind or wave set-up should be treated separately depending on the characteristics at the site.

(3) As surges can raise or lower the water level significantly above or below astronomical tide levels, the magnitude of surge under a range of appropriate design events should therefore be quantified.

(4) At sites where the tidal range is small relative to the design event surge magnitude, the extreme values analysis should be undertaken on the surge component only, i.e. where the astronomical tidal component is separated from a record of total water-levels and the surge records are treated independently.

(5) The extreme surge values may then be combined with a selected tide level (and other components of the design water level).

(6) At sites where the tidal range is large relative to the design event surge magnitude, careful consideration, selection and justification of whether tide and surge are treated separately or in combination should be made, e.g. to avoid a potential over-estimate of a design water level by addition of an extreme surge value and a high astronomical tide condition.

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

(7) Assessment of surge magnitude should be based on long-term continuous water-level measurements sufficiently close (so as to have the same characteristics) to the site of interest.

(8) Where measurement data is not available or of sufficient quality or duration, investigation of storm surges may include data collection and hindcasting of storm surges in the past and numerical evaluation of hypothetical storm surges in the future.

(9) In locations where actions from tsunamis are not negligible, tsunami characteristics at the site should be investigated by means of data collection and hindcasting of tsunamis in the past and/or numerical evaluation of hypothetical tsunamis in the future.

## Waves

### General

(1) A method of simplifying or representing the real wave conditions should be selected which is appropriate to the application.

NOTE Depending on the application, governing design actions from waves can depend on wave height, wave crest or single wave period exciting resonant response.

(2) The method used should take into account whether the waves are in deep, nearshore (transitional) or shallow water and the likely spectral shape and average steepness of the sea state.

NOTE The average steepness of a sea state is limited by breaking of individual wave crests.

### Wave set-up

(1) The degree of wave set-up at the site should be assessed and included as a component of the design water level.

NOTE Wave set-up is generally caused by breaking wave energy that cannot be dissipated by return flow to open water and therefore leads to a localised build-up in water-level at the shoreline or against a coastal structure.

(2) If wave set-up effects are already accounted for within design approaches to be used, the effect of wave set up on the design water level as an input parameter may be ignored.

NOTE Wave set-up is normally greater where large breaking waves impact on a constricted coastline, e.g. a narrow bay or channel, with comparatively lower set-up experienced at the open coast (for similar breaking wave conditions/ foreshore slopes). Wave set-up can be increased by the presence of long waves.

### Frequency and directional distribution of waves

(1) Wave conditions at the site should be represented by a wave energy spectrum.

(2) Both frequency and directional distribution of the wave energy should be taken into account.

(3) Contributions from locally generated wind waves as well as longer period swell should be included in the wave spectrum.

(4) Wind waves and swell may have different directions. Hence, the wave spectrum may be bi-modal and bi-directional.

NOTE Bi-modal or bi-directional sea-states can have more onerous consequences for some design situations, e.g. for floating or other dynamic structures, wave run-up/wave overtopping/transmission or beach response.

### Spectral wave description

(1) For an accurate analysis of wave conditions, a spectral approach should be used.

NOTE 1 If required, regular wave parameters for design (e.g. wave height, wave crest and wave period) can be derived from wave spectra.

NOTE 2 A wave spectrum at a particular site can be obtained either through measurements and analysis of wave surface elevation or by adopting an appropriate standard wave spectrum.

NOTE 3 Wave spectra can be established based on extreme values of *H*s and associated peak period or as curve or parametric fitting to wave spectra from a numerical wave model in storm situations.

(2) The wave spectrum should include both wind sea and swell, if relevant.

NOTE 1 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). See ISO 19901-1.

NOTE 2 The swell component of the spectrum can be approximated by the JONSWAP spectrum with the peak enhancement factor depending on the distance the swell has travelled. A value up to 10 for long distance swell can be used.

NOTE 3 When swell co exists with locally generated wind seas, two or possibly more peaks occur, and is often referred to as a “bi-modal” sea. The frequency spectrum of the sea state in such cases can be represented by a recognised bi-modal spectrum.

EXAMPLE Ochi-Hubble bi-modal spectrum.

NOTE 4 When wind waves and swell coexist, wave spectra exhibit multiple peaks. Difficulty is encountered in defining the significant wave period and the spectral peak period as well as the wave direction in case of multi-peaked wave spectra. Evaluation of the action from waves of multi-peaked spectra can be made by calculating contributions of components, which is constructed by superposing the spectra of wind waves and swell in question.

(3) If different directions of swell energy and wind sea in bi-modal sea-states is critical, such conditions should be taken properly into account, e.g. by use of hindcast data.

(4) The evaluation of actions from waves in shallow or intermediate water should take into consideration that the extent of the directional spreading of waves becomes narrower in shallow water than in deep water because of wave refraction effects.

### Storm-representative wave parameters

(1) The method for determining storm-representative wave parameters should reflect the HEA and DA levels.

(2) For analysis of structures having static or quasi-static response characteristics to wave loading, in most cases a limited number of waves, water level and current conditions, focusing on operational, extreme or accidental design scenarios, should be taken into account.

(3) Where the dynamic response of the structure to wave action is significant, account should also be taken of the possible range of wave periods, directions and associated (maximum individual or significant) wave heights that would result in the greatest dynamic magnification.

(4) Storm-representative wave parameters should be obtained taking due account of the conditions at the site.

### Wave data sources

(1) For HEA2 and HEA3 wave data should be obtained from in-situ measured and numerical model data.

NOTE Satellite remote sensing data can be used for calibration/ verification of large-scale numerical hindcast models.

(2) Where available and if the quality and reliability of the data can be assured, in situ measured wave data should be the preferred data source and should be obtained and used in the design.

(3) The reliability of published values of wave data, should be assessed for use in design, and alternatives should be sought where appropriate.

EXAMPLE For example, values of wave data are published in EN ISO 19901-1.

NOTE In many situations, the duration over which measured data is available is insufficient to allow an understanding of the long-term wave climate, particularly in relation to extrapolating extreme storm event conditions for design purposes (see also 5.2.1).

(4) If wave data (of sufficient quality and length of record) is not available at the site, offshore numerical model data or wave measurements should be obtained and reviewed, prior to transforming the data to the nearshore site.

(5) The relative water depths of the offshore data location and the site 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.

(6) Hindcast wave data should be verified against measurements for an overlapping time period, preferably including a storm event.

NOTE This is especially important for HEA3.

### Wave transformation

(1) Wave transformation shall be performed with use of one or a combination of the following methods:

* numerical methods;
* physical model tests;
* empirical methods.

(2) The selected method for wave transformation should be in accordance with Clause 4 and Clauses 6 to 11.

(3) Numerical wave models should be applied to transfer deep-water wave data to the locations of interest and should always be used to calculate transformations to wave spectra.

(4) Where relevant, it should be verified if a suitable length and quality of wind record is available for input/boundary condition purposes to provide a reliable basis on which to estimate wave conditions.

NOTE Waves undergo various transformation processes while travelling from deep water toward the shore. The following spatial/ geographical domains are commonly associated with particular wave transformation processes (as stated):

* deep-water/ offshore location (wave generation, propagation and spectral interaction);
* nearshore/ coastal domain (refraction);
* surf-zone/ shallow water/ littoral domain (shoaling, breaking);
* harbour domain (diffraction, reflection, transmission) or also where islands or reefs are present in the nearshore or shallow water domains;
* estuarine domain (current interaction).

(5) A wave transfer model should be chosen to best account for the complexity of the site (e.g. distance between the deep-water location and the coastal site, wind generated waves, realistic representation of swell).

NOTE 1 Phase averaged models are relevant for slowly varying wave conditions and are generally used for transforming waves from offshore to nearshore over large distances, typically tens of kilometres. They are based on calculating quantities such as spectral energy densities rather than the time varying details of the water surface elevation. These models are appropriate for generating local hindcast data sets, which can be used for extreme values analysis. They are generally not suitable for modelling wave steepening and breaking in the nearshore area, interaction with structures or diffraction.

NOTE 2 Phase resolving models are relevant for rapidly varying wave conditions such as diffraction around structures, regions with excessive wave breaking and non-linear wave interactions. They are currently confined to applications involving relatively small sea areas, typically a few square kilometres, due to the computation time and computer memory requirements.

NOTE 3 Computational fluid dynamics (CFD) can be used in modelling waves and wave structure interactions, and can allow a more accurate representation of the physical processes, but are computationally intensive.

NOTE 4 For HEA1, the empirical fetch method can be used where the wave climate at a site of interest is expected to be dominated by locally wind generated waves with negligible swell wave energy and where more reliable wave data sources are not available. Such an approach is generally restricted to inland or enclosed lakes or seas. There can be other cases where such an approach is technically justified, e.g. where the storm-representative wave parameters or structure performance/response are known with a high degree of confidence not to be sensitive to the contribution of swell wave energy.

(6) As no existing wave transformation model, phase-resolving or otherwise, involves a comprehensive description of all of the many processes involved, the simplifying assumptions that all numerical wave models are based on should be taken into account when interpreting model output.

(7) For HEA3 the wave transformation should be calibrated/validated with short term measured data at the site, as well as physical model tests or alternative numerical approaches as appropriate to the risks involved.

(8) If measured nearshore data of relatively short duration is used to calibrate or validate numerical models applied for offshore to nearshore wave transformations, in order to transform extreme storm events for design purposes, the calibration should ideally be based on measured data including at least one storm event of some significance.

(9) In the case of very shallow foreshores where wave breaking and re-forming occurs, bi-modal or flattened wave spectra can develop, the characteristics of which are not represented by many approaches. If necessary, the wave transformation assessment should use a phase resolving numerical model or physical modelling to better understand the resulting wave actions.

### Wave data for extreme value analysis

(1) A local hindcast data set for extreme values analysis can be based on a detailed coastal model verified against measurements.

NOTE Site measurements are usually not carried out for a duration that is long enough for extreme values analysis.

(2) An understanding of the expected wave conditions due to site characteristics should be used to inform appropriate transformation of offshore hindcast wave data to the site.

NOTE This is important for areas with complex coastlines and bathymetry, in situations where nearshore wave processes significantly modify important parameters such as wave breaking, storm duration or direction, or where coincident timing of wave conditions and water-levels is important (for some dependent/joint probability analyses). See 5.4.

(3) If the local hindcast or observation data set is not sufficiently long, the full offshore wave record may be transformed to the site by means of a relation in wave parameters between the offshore and the local data set.

(4) Alternatively, an extreme values analysis may be conducted on offshore wave data records, the results of which (individual design events) are then transformed to the site of interest, using a local numerical model.

NOTE Subject to the wave characteristics at the site, it can be necessary to separate the wave data into directional sectors which are then treated separately for probability density function fitting.

### Nearshore wave processes

(1) Nearshore effects should generally be taken into account when the depth reduces to less than one half the deep-water wavelength.

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 A.

(2) Variations in water level (due to tide, surge or wind/wave set-up/set-down) and bathymetry (due to accretion or scour) should be assessed 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 representative wave conditions.

NOTE For some nearshore structures, the representative wave height can be limited by assessing wave breaking, as described in the guidance in Annex A.

### Regular wave theories

(1) Where a random wave approach is not considered practical, a regular wave theory should be selected which is suitable for the possible wave shapes to be encountered and the response of the structure or element being assessed (see Figure 5.1).

NOTE 1 The simplest representation of a regular wave is that of a linear sinusoidal wave (Airy wave theory). Linear wave theory can be used when conditions for its validity are fulfilled and for computational reasons such as in spectral fatigue calculations and for large volume body radiation/diffraction analysis calculations.

NOTE 2 As wave steepness (*H*/*λ*) or relative wave height (*H*/*h*) increases, the wave deviates from a sinusoidal shape, as is the case, for example, in the nearshore shallow water zone or when waves meet an opposing current. In such cases, linear wave theory becomes less valid and a non-linear wave theory gives a better representation of the sharper crests and longer troughs of real waves.

(2) A non-linear regular wave theory should therefore be used where applicable and the design should take into account the fact that the velocity of a co-existing current should be combined with the fluid particle velocities caused by waves in order to calculate actions on structures, typically for slender structures or elements.

NOTE 1 Non-linear analytical wave theories include Stokes 5th order wave theory for deep and intermediate water depth where *λ*/*h* < 10 and cnoidal wave theory for long waves in shallow water (*λ*/*h* > 10). Stokes’ 5th order theory is less accurate for situations where the Ursell number (*Π*R = *H* *λ*2⁄*h*3) is greater than 26. Cnoidal wave theory may be used when the Ursell number is greater than 26. For *Π*R ~ 26, both Stokes 5th order wave theory and cnoidal wave theory have inaccuracies. For such regular waves, the stream function solution method can be used.

NOTE 2 Depending on the steepness of the regular wave, lower order Stokes waves, e.g 2nd or 3rd order may provide sufficient accuracy for design (see Figure 5.1).

NOTE 3 The stream function solution method, which is a numerical method based on Fourier analysis of the stream function, is accurate to within 1 % of the highest possible waves and is valid for all finite wave lengths. For shallow water, where Stokes 5th order theory is not valid, this method is preferred for practical applications.

(3) Breaking waves cannot be modelled by regular wave theories and actions from breaking waves shall be calculated by considering the shape and particle kinematics of the breaking wave.

NOTE A breaking wave may have different shapes, e.g spilling, plunging or surging, depending on wave steepness and slope of seabed.

### Wave shape and kinematics

(1) Depending on the application, the largest wave height and/or the highest wave crest in a sea state should be estimated from a wave height and crest height distribution for statistical modelling of waves in a short-term sea state.

NOTE For deep and intermediate water depths, a Weibull distribution (e.g Forristall crest distribution) can be applied, while for shallow water where wave breaking can occur, a Generalized Extreme Value (GEV) distribution is normally recommended.

(2) Depending on the application, regular storm-representative waves should be based on either wave height and associated wave period or crest height and associated wave period.

NOTE 1 For global response of floating structures maximum wave height can be governing, while for fixed structures being sensitive to horizontal fluid particle kinematics above still water level, maximum crest height can be governing.

NOTE 2 Although particle velocities at the surface are greatest in steep waves (relatively high wave/short period), a longer period wave can result in greater velocities at depth, because the velocities decay less rapidly.

(3) Non-linear regular wave theories and/or reliable laboratory test data should be referred to when estimating the crest elevation of storm-representative waves.

(4) Wave kinematics, e.g. fluid particle velocities and accelerations, should be evaluated by means of non-linear wave theories of high accuracy, since the linear wave theory underestimates the orbital velocities especially from the mean water level (MWL) to top of wave crest.

NOTE If relevant for time-domain simulations of hydrodynamic actions from random waves, non-linear irregular wave kinematics can be modelled by a Higher Order Spectral Model (HOSM). For shallow water, where wave breaking may occur, a Boussinesq model can be applied.

(5) When waves are expected to break at a location at which the actions from waves are to be evaluated, special consideration should be taken in evaluation of the kinematics and the shape of the waves.

NOTE For breaking waves, use of model tests and/or advanced numerical models is normally recommended for the evaluation.

(6) When currents of appreciable strength coexist with waves, the vector sum of the current velocity and the fluid particle orbital velocities by waves should be employed in evaluating the fluid kinematics.

**Ein Bild, das Diagramm enthält.

Automatisch generierte Beschreibung**

Key

|  |  |
| --- | --- |
| 1 | breaking wave limit (*H*/*h* ~ 0,8) |
| 2 | cnoidal wave theory |
| 3 | linear wave theory |
| 4 | Stokes 2nd order |
| 5 | Stokes 3rd order |
| 6 | Stokes 4th/5th order |

Figure 5.1 — Validity of regular wave theories

### Long waves

(1) The potential for long wave conditions to develop at the site and hence the likely risk of long waves affecting the structure should be assessed, e.g. inducing harbour resonance/seiching.

NOTE 1 Long-period waves with periods typically of the order of 25 s to several minutes affect nearshore sediment transport, beach morphology, harbour oscillation, wave overtopping/stability and energy transmission through breakwater structures. Local morphology, structure configuration and basin/harbour shape and size can amplify or dampen the effects of the incident or reflected long waves.

NOTE 2 There are various possible causes of long waves, including:

* moving pressure fronts (including storm surges);
* wave grouping effects and energy dissipation of certain frequency bands only;
* tsunamis.

(2) A statistically based tsunami event should be defined using a probabilistic approach accounting for historical and projected future seismic activity in the region of interest, likely fault mechanism and displacement.

(3) An appropriate hydrodynamic model should be employed for the estimation of a tsunami.

## Currents

### General

(1) Type of currents that contribute to the forces on the structure should be documented.

NOTE 1 Currents can have an effect on structures, directly and indirectly. Directly, current flows exercise drag and lift forces on the structure. Indirectly, currents interfere with waves and modify the wave kinematics and thus affect the actions from waves and currents.

NOTE 2 Currents in coastal waters can be divided into tidal currents, wind driven currents, density driven currents and wave induced currents. Current velocities are in general stronger in coastal waters than in the deeper oceans.

NOTE 3 More guidance is given in A.5.1

(2) For HEA2 and HEA3 measurements should be included when establishing current conditions at site.

NOTE 1 Current measurements include both magnitude (speed) and direction of current velocity, as well as spatial variation in vertical direction (current profile).

NOTE 2 Numerical modelling of currents can be used to document variation of current velocities around the structure.

(3) Current–wave interactions should be considered when evaluating the actions from waves and currents unless the currents are weak.

### Current data sources

(1) Numerical modelling of currents may be used when appropriate care is taken to confirm the reliability/validity.

(2) Current velocities at site should be measured for at least 12 months to account for seasonal variations.

NOTE 1 Other measurement lengths can be given in the National Annex.

NOTE 2 Typical characteristics for current measurements are:

* maximum measurement interval: 10 min;
* minimum averaging period for current velocity: 1 min;
* temporal resolution: 1 Hz.

(3) Dependent on the sensitivity of the structure, measurements should be performed at several depth levels.

### Current velocity and profile

(1) The current velocity should be expressed in vector form with the absolute magnitude (speed) and the direction or with the velocity components in a coordinate system.

NOTE The characteristics of the extreme current profile needed for the design of coastal structures can be difficult to determine. It is unlikely that any measurement programme will be sufficiently long to capture a representative number of severe events.

(2) When long-term field measurements are not feasible, numerical modelling of currents may be carried out for gaining information on extreme current velocities and current profiles.

NOTE 1 In lack of available current data, surface current in areas with negligible local topography can be taken as 2 % of 10 min averaged wind speed at 10 m above sea water level.

NOTE 2 Numerical models can be depth-averaged models or fully three-dimensional, simulating current velocity at different water depths.

(3) Numerical models should be validated against measured data at relevant depths on site.

NOTE It can be relevant to separate the tidal contribution in the measurements from the total current to find the residual current.

## Climate change

(1) Design of coastal structures should take account of the fact that climate change can cause sea-level rise and changes in storm intensity and direction (as well as other changes which could be of relevance, e.g. temperature, salinity).

NOTE Enhanced storm intensity can affect coastal wave height, surge, pluvial and/or fluvial flooding.

(2) The design 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.

(3) Effects of climate change should be assessed in relation to:

* the known purpose and design working life of the structure;
* its vulnerability and the consequences of damage;
* the potential for future adaptation of the structure.

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

(5) The design should take into account the evolving nature of published estimates due to the inherent complexity in the science of climate change.

NOTE 1 The Intergovernmental Panel on Climate Change (IPCC) provides an updated assessment every five to seven years, on which regional authorities can base local guidance.

NOTE 2 It is considered good practice to put in place measures that are robust across a range of probability levels. The range depends on the purpose of the structure, the potential risks and consequences, and the costs and benefits of allowing for different levels of uncertainty.

(6) 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, also including consideration of the magnitude and rate of change.

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

(8) Where appropriate, an allowance should be made for increased 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) Where relevant to the structure being designed, potential climate change effects on parameters other than water-level and wave conditions should be considered.

(10) Where national guidelines for considering such parameters exist, these should be followed; otherwise the approach used should be explained to the client or end user as appropriate.

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

# Wave and current actions on fixed cylindrical structures and suspended decks

## General

### Applications

(1) Actions from waves and currents on fixed cylindrical structures and suspended decks, may be derived in accordance with this Clause 6 provided that:

1. the structure falls into design class DA1, DA2 or/and DA4, see Clause 4;
2. motions and dynamic effects are sufficient small that quasi-static design methods in combination with dynamic amplification factors (DAF) are applicable in design.

NOTE 1 Additional guidance is given in Annex B.

NOTE 2 Cylindrical structures are defined as single isolated cylinders or structures composed of vertical cylindrical members or a truss structure with vertical, inclined and horizontal cylindrical members or a pipeline close to or on the seabed. Cylindrical structures can be part of a superstructure (e.g. lighthouses, jetties, suspended decks).

NOTE 3 Suspended decks are platforms which are exposed to wave action, either of a jetty or a fixed supported platform.

NOTE 4 Structural response will be determined by quasi-static structural stiffness when motions and dynamic effects are small. Example of principle of dynamic amplification factors is found in Annex B.

(2) This Clause 6 may be applied to derive actions on other types of fixed structures provided that application is technically justified.

(3) Actions from waves and currents should include effects of drag, damping, inertia, slamming, diffraction, radiation and vortex shedding (VIV, galloping etc.) where relevant.

(4) Effects of marine growth, variations in the sea level, physical properties of water (density) and structure/water interaction (friction, eddies) shall be taken into account.

NOTE 1 Stratification of the water column (circulation in layers) causing actions from internal waves propagating between the different layers is outside the scope of this Clause 6.

NOTE 2 Marine growths can be taken into account in accordance with Clause B.5.

(5) Disturbance and amplification of waves and currents by the structure and neighboring/nearby structures shall be taken into account.

EXAMPLE Such disturbances can be run-up effects, wave overtopping, reflection, particle acceleration and flow disturbance.

NOTE Guidance on numerical models and model testing is given in Clause B.12.

### Principles for assessing actions from waves and currents

(1) Methodology to assess actions from waves and currents may be chosen depending on the chosen design approach.

NOTE 1 For wave and current action assessment, see Clauses 4. For hydrodynamic conditions, see Clause 5.

NOTE 2 Further guidance for selection of design methods for actions on fixed structures is found in Annex B.

(2) Design actions on fixed structures from waves and currents may be calculated by use of a regular storm-representative wave according to 5.4.5 and 6.1.2(3).

NOTE 1 The nature and magnitude of wave actions depends on the wave height, period and spectral form, as well as the hydrodynamic force regime and the dimensions and physical properties of the structure.

NOTE 2 Storm-representative wave is a practical implementation of a rather complex sea state, used in quasi-static design.

(3) In the storm-representative wave approach, the wave parameters may be assessed by statistical methods, see Clause 4 and Clause 5.

NOTE If the structure has significant dynamic response, stochastic modelling of the sea surface is recommended. If the response is sensitive to wave kinematics above still water level, a nonlinear (e.g. second order) stochastic model of the sea is recommended. In such cases, sea state is usually assumed as a stationary random process, characterized by a significant wave height, peak period, mean direction and spreading function.

(4) In the design regular wave approach, the maximum wave height *H*max, and the corresponding wave periods shall be chosen based on a maximum crest height, *η*max.

(5) The storm-representative wave height should be taken corresponding to annual exceedance probabilities given in Clause 4, determined by the long-term statistics using a 3-hour sea-state, see Clause 5.

(6) Due care shall be taken to estimate wave crest elevation and wave kinematics in calculation of actions from waves.

NOTE For further guidance, see Annex B.

(7) The storm-representative wave shall be based on the most unfavourable combination of wave height, wave crest and wave period.

NOTE For further guidance, see B.9.6.

(8) Wave kinematics should be based on higher order wave theories, see Clause 5 and related guidance in Annex A.

(9) Provisions based on linear waves theories, and possible simplified methods may be used:

1. if recommended and technically justified; or
2. for HEA1; or
3. for wave actions below the still water level and where relevant stretching techniques are applied to account for the free surface effects; or
4. for forces on a slender cylindrical pile where linear wave theory provides a reasonable approximation of the wave kinematics, i.e. cases with small amplitude waves and greater depths where the exclusion of wave forces on the cylinder above SWL is considered to be acceptable, see Figure 5.1.

NOTE 1 The linear theory is applicable if the consequences of failure are sufficiently small or if actions from waves and currents are shown to have small influence on the reliability of the structures.

NOTE 2 For guidance on stretching techniques, see Clause 5, Annex A and Annex B.

NOTE 3 For further guidance on the applicability of different wave theories, see Clause 5.

(10) Additional action effects due to overtopping shall be taken into account according to Clause 8.

(11) For the verification of ultimate limit states of fixed cylindrical structures and suspended decks, 4.8(2) applies.

### Conditions for disregarding actions from waves and currents

(1) A minimum air gap should be taken into account for persistent design situations.

NOTE 1 A minimum air gap represents a safety margin in the persistent design situation.

NOTE 2 A minimum air gap can be set to 20 % of the storm-representative wave height *H*max, unless the National Annex gives different values.

NOTE 3 For more information, see B.9.5.

(2) The storm-representative wave should not interact with structures in the accidental design situation.

NOTE For provisions on accidental design situations, see Clause 4 and EN 1990:2023, Clause A.6.

### Current actions

(1) Actions from currents in combination with waves shall be in accordance with 6.1.1.

NOTE Current velocity can be considered a steady state flow field, where the velocity is a function of water depth. See B.5.2.

(2) Current actions on a slender structure should be calculated with a drag force formula with an appropriate drag and lift coefficient for the relevant flow condition.

NOTE For further guidance, see B.5.2.

(3) Current actions induced by adjacent structures or equivalent sources (e.g. ships or ship operations) should be considered.

### Wave and current actions on cylinders from non- breaking waves

(1) Type of wave action on cylindrical structures should be decided depending on the vortex shedding influence, see Figure 6.1.

NOTE 1 If the water particle motion across the cylinder is less or equal to the cross-sectional dimension of the structure vortex formation is not likely to occur and potential theory normally applies. Else, Formula (6.2) can be applied.

NOTE 2 The Keulegan-Carpenter number *KC = u*max*T/D* defines whether Morison’s formula or potential theory applies. If *KC* < 2 - 3 potential theory can be applied. If *KC* > 2 - 3 Morison’s formula can be applied.

NOTE 3 For further guidance, see Clause B.5.

(2) Type of wave actions on vertical structures in the vicinity of the sea surface in deep waters should be classified as shown in Figure 6.1.

NOTE In Figure 6.1, *H/D=KC*/π at SWL. See Annex C for further guidance.

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**Key**

|  |  |
| --- | --- |
| *D* | characteristic dimension |
| *H* | wave height |
| *λ* | wavelength |
| I | All inertia |
| II | Diffraction region |
| III | Large inertia |
| V | Inertia and drag |
| VI | Large drag |
| 1 | Deep water breaking wave curve |
| 2 | Small drag |
| 3 | Negligible drag |
| 4 | Negligible diffraction |

Figure 6.1 — Type of wave actions (force regimes)

(3) Actions from non-breaking waves and currents on small diameter cylinders and cylindrical structural elements where *KC* > 2 – 3 may be calculated using Formula (6.2).

NOTE For further guidance, see Clause B.5.

(4) Actions from non-breaking waves on large diameter cylinders and cylindrical structural elements where *KC* < 3 may be calculated by:

* wave diffractions theories;
* numerical methods; or
* hydraulic model tests.

NOTE 1 For further guidance on selection of methods, see Clause 4, Annex A and Annex B.

NOTE 2 The diffraction force regimes can be taken correspond to regimes II and IV in Figure 6.1, equivalent to *πD*/λ > 0,5.

### Wave and current actions from breaking waves

(1) Actions from possible breaking waves shall be taken into account with appropriate methods.

NOTE 1 For further guidance in selection of design methods, see Clause 4 and Annex B.

NOTE 2 Breaking waves will normally increase the uncertainties in calculations of actions and is normally more complex than actions for non-breaking waves.

NOTE 3 Rapid varying bathymetry and three-dimensional effects on reefs will further increase uncertainties in assessing actions from breaking waves. See Annex B.

### Slamming actions from waves

(1) Structures and structural members with a possibility to enter and leave the water surface of waves shall be considered subjected to slamming actions.

NOTE 1 For further guidance on slamming, see Annex B.

NOTE 2 Geometrical properties in the impact zone can affect the magnitude of the slamming actions.

(2) Due care should be taken of spatial variation of the wave actions, impact area distribution and simultaneous slamming actions on structures.

NOTE 1 Such variation will give local and global load effects.

NOTE 2 Both horizontal, vertical, positive and suction forces are possible.

NOTE 3 For further guidance, see Annex B.

### Wave actions on small diameter pipelines

(1) Wave and current actions on small diameter pipelines on or close to the seabed should be calculated with a Morison type formula for the horizontal force and a similar equation without the inertia term for the lift force.

### Current and wave induced vibrations

(1) Current and wave induced vortex shedding induced vibrations (VIV) of slender structures and pipelines in free spans shall be considered.

NOTE For further information, see Annex B.

### Seabed scour at cylinders due to waves and currents

(1) The scour from waves currents and operations (propeller) that can occur close to and around structures on an erodible seabed should be considered.

NOTE For further guidance, see Annex B.

## Current actions on slender structures

(1) Actions from a steady current on a single structural element may be determined by use of the following drag force, see Formula (6.1):

(6.1)

where

|  |  |
| --- | --- |
| *ρ*w | is mass density of the water; |
| *C*DS | is drag coefficient for steady flow; |
| *D* | is characteristic diameter of the structural member; |
| *l* | is characteristic length of the structural member; |
| *U* | is current velocity. |

(2) Water particle velocities should be added vectorially to calculate the drag force when waves combine with currents, preferably water particle velocities based on exact wave-current interaction theories, if available.

NOTE 1 When waves combine with currents, a non-linear increase in drag force on the structure will occur, due to its quadratic nature.

NOTE 2 For further guidance, see Clause 5.

(3) Size and diameter shall include marine growth where relevant.

(4) Drag coefficients shall be technically justified and take into account relevant geometrical effects (i.e roughness and shape) and possible marine growth.

NOTE 1 For further guidance, see Clause 5, Annex A and Annex B.

NOTE 2 For circular cross-section the drag coefficient is a function of the Reynolds number, *Re* = *u*max*D*/*υ*, *υ* = kinematic viscosity, and the roughness of the cylinder, see Annex B.

NOTE 3 Design values for marine growth for the North Sea basin can be in accordance with Table B.2 unless other values are given in the National Annex.

## Wave Actions on slender bodies

### Wave actions on single slender cylinder

(1) The wave actions on a length *dz*, see Figure 6.2, of a relatively slender member may be expressed as the sum of a drag force and an inertia force through the Morison’s load Formula (6.2):

(6.2)

where

|  |  |
| --- | --- |
|  | expresses the potential flow theory, see Annex C for more information; |
| *dF* | is actions vector normal to the member axis; |
| *dF*D | is drag actions vector; |
| *dF*I | is inertia actions vector; |
| *C*D | is drag coefficient; |
| *C*M | is inertia coefficient, *C*M = 1+ *C*a; |
| *C*a | is added mass coefficient. *C*a = *m*a/(*ρA*), where ma is the added mass per unit length and *A* is cross-sectional area; |
| *D* | is the characteristic dimension or diameter of the cylinder; |
| *U* | is local water particle velocities normal to the member axis; |
|  | is local water particle accelerations normal to member axis; |
| *ρ*w | is mass density of the water; |
| *dz* | is length of the considered member.  A black background with a black square  Description automatically generated |

**Figure 6.2 — Wave actions on a cylindrical pile**

NOTE 1 Formula (6.2) is applicable when *KC* > 2 - 3 and *D* << *λ*.

NOTE 2 Formula (6.2) can also be applicable for small volume three-dimensional bodies which falls in category *λ* > 5*D*. In such cases, in Formula (6.2), the cross-sectional area in inertia term is replaced by volume *V* and cross section dimension *D* in drag term by the projected surface area (*A*p) perpendicular to the flow of the 3D body.

NOTE 3 Drag (*C*D) and inertia (*C*M) coefficients in Formula (6.2) depends on surface roughness, Reynolds number and Keulegan-Carpenter (*KC*) number. Further information on *C*D and *C*M can be found in Annex B.

NOTE 4 See 6.3.1(9), Note 3 for further information on inertia coefficient.

(2) The short-term variability in the wave actions should be taken into account.

NOTE 1 See also Annex A and 6.1.2(3) to 6.1.2(5).

NOTE 2 Formula (6.2) assumes that the structure does not disturb the free surface. Hence for steep waves there is a loss of accuracy in Formula (6.2) due to higher order potential flow effects.

(3) In the presence of current, the local water particle velocity should be displaced with the vectorial sum of the current velocity and the wave induced particle velocity.

(4) If relevant, local velocity should include disturbance from neighboring large volume elements. Usually, the wave particle velocity is calculated based on the undisturbed wave fields.

(5) If available, total particle velocities and accelerations should be calculated based on more exact wave current interaction theories.

(6) Structural members in motion may be accounted for by a modification of the velocity and acceleration (relative velocity and acceleration) of the member (structure response).

(7) In the case of inclined cylinder, an extension of Formula (6.2) should be used to obtain the force. The water particle kinematics normal to the inclined cylinder should be substituted in Formula (6.2).

NOTE 1 See Annex C for more information.

NOTE 2 Eddies can be shed from a cylinder alternatively on either side, thus inducing a transversal force normal to the main direction of the water particle velocity. The type of vortex shedding depends on *KC* numbers. Slender structures subject to current flow may also experience inline oscillations caused by vortex shedding.

(8) Drag coefficients shall take into account all relevant geometrical effects (i.e roughness and shape) and possible marine growth.

NOTE 1 Design values for marine growth can be given in the National Annex.

NOTE 2 See Annex C for further guidance.

(9) Cross-sectional dimensions shall take into account marine growth.

NOTE 1 Marine growth will depend on the site, see Annex C for more information.

NOTE 2 The drag, inertia and lift coefficients have been obtained from analysis of results from experimental work. The values of the drag and inertia coefficients depend on the Keulegan-Carpenter (*KC*) number, the Reynolds number and the cylinder surface roughness. See Annex C for more information.

NOTE 3 For low *KC* numbers, and *D*/*λ* sufficiently small, the value of *C*M for smooth circular cylinders is 2. The value of *C*M is dependent on *KC* and surface roughness. See Annex C for more information.

(10) The wake encountered by the cylinder should be considered for low *KC* numbers (*KC* < 60). The steady state drag coefficient in such cases should be multiplied by a “wake amplification factor”.

NOTE 1 The wake effect can be neglected for large *KC* numbers. Further information can be found in Annex B.

NOTE 2 Marine growth is considered to contribute mostly to the roughness of a structural element in the water.

### Wave actions on clusters of circular cylinders

(1) Shielding may be taken into account in calculation of actions on clusters and arrays of structures.

NOTE For further guidance, see Annex B.

(2) Spatial variations in the wave field may be taken into account in calculation of actions.

NOTE For further guidance, see Annex B.

## Wave Actions on large volume bodies

(1) Wave actions on large volume bodies should be calculated on the basis of wave diffractions theory.

NOTE 1 The method is generally applicable according to Figure 6.1. See also Figure B.1.

NOTE 2 The incoming waves are reflected and scattered, and the wave potential is expressed as a sum of the potential of the incoming waves and those of the scattered waves.

(2) For simple structural shapes like a vertical circular cylinder resting on the sea bottom the MacCamy and Fuchs analytical solution may be applied.

NOTE For further guidance, see Annex B.

## Wave Impact and slamming actions

### Wave slamming on slender structures

(1) The force on slender structural members near the water surface caused by wave slamming are impulsive in nature and should be considered along with Morison type forces.

(2) Semi empirical formulae may be used to estimate the slamming forces on vertical and inclined cylindrical structures.

(3) The maximum slamming force on a vertical cylinder may be taken as Formula (6.3):

(6.3)

where

|  |  |
| --- | --- |
| *F* | is the slamming force; |
| *λ*c | is the curling factor; |
| *η*b | is the maximum elevation of the free surface; |
| *R* | is the radius of the cylinder, *R* = |*D*|/2; |
| *C* | is the breaking wave celerity. |

NOTE For further details on estimating slamming force, see Annex B. See also Figure B.1.

(4) Special care should be given to the calculation of the impact area, velocity of wave impact, slamming coefficient and impact duration.

### Wave in deck forces and air gap

(1) Relevant wave in deck analysis should be performed unless it is demonstrated that the air gap is sufficient to avoid deck forces.

(2) A positive air gap should be considered for HEA3 (see Table 4.1 (NDP)).

NOTE For guidance for sufficient air gap, see Annex A and Annex B.

(3) Horizontal or vertical forces due to wave in deck slamming should be investigated.

NOTE Slamming forces is normally critical for design. Further information can be found in Annex B.

### Dynamic amplification and vibrations

(1) Dynamic effects due to wave slamming force should be investigated.

NOTE For further details, see Annex B.

(2) Standard charts and formulae with simple mode of vibration may be used if technically justified.

NOTE For further information, see Annex B.

(3) Springing and ringing effects should be considered.

## Wave actions on pipelines and subsea structures

(1) Wave and current actions on pipelines on or close to the bottom, may be calculated with a Morison formula approach (see 6.3.1).

NOTE 1 For further information, see Annex B.

NOTE 2 For robustness of subsea structures, see EN 1990:2023, Clause A.6.

(2) Pipelines (landfalls) shall not be exposed for breaking waves.

NOTE For wave protection in the surf zone, see Clauses 7 – 10.

## Vortex induced vibration (VIV) of pipelines

(1) Characteristics of actions related to vortex shedding should be based on fluid structure interaction analysis (dynamics).

NOTE For further guidance, see Annex B.

# Wave and current actions on mound breakwaters

## Introduction and structure types

(1) This Clause 7 contains provisions for wave and current actions relevant to the verification of mound breakwaters.

NOTE 1 For further guidance, see Annex C.

NOTE 2 Conventional mound breakwaters are characterised by two sloping faces and a permeable structure. While the core is most often made of relatively small size well-graded stone material, the slope surfaces are generally armoured with larger uniformly graded rocks or concrete blocks of various shapes. Core and armour layers are separated by one or more filter layers, or geotextile. A monolithic concrete crown wall, sometimes fully or partly sheltered by armour blocks, can be used for the crest formation, especially when road access along the mound is needed. Conventional mound breakwaters allow minimal wave overtopping and present insignificant wave transmission through their body.

NOTE 3 Berm breakwaters are a special type of rubble mound breakwaters that allow a certain degree of deformation of the front slope surface under wave action in order to gain stability against further such action. A berm is formed around the mean sea level on the seaward side either at construction stage, in stable berm breakwaters, or through wave action during the initial stage of the structure’s lifetime, in static reshaping berm breakwaters. The berm and its front facing the sea absorb wave energy gradually through friction in their hollow parts permitting thus the use of smaller armour stones on the main breakwater face.

NOTE 4 Mound breakwaters can also display increased wave overtopping or permeability features that ease, in some respects, the hard-environmental intervention of conventional non-overtopped impermeable breakwaters. Main examples of such structures are the low crested (emergent) and submerged breakwaters, or those of high permeability, usually built of uniform rock material. These types of breakwaters, however, cannot be suitable for harbour but rather for coastal protection, depending on the wave transmission response of the structure. In low-crested structures this response is in general made up from the overtopped water generating new waves in the lee of the structure and the transmitted wave through the structure, if any. In submerged breakwaters the wave overtopping part is substituted by the transmitted waves over the structure’s crest.

NOTE 5 Figure 7.1 presents sketches of representative cross-sections of the above three main classes of mound breakwaters.

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**Figure 7.1 — Representative mound breakwaters: (a) conventional, (b) with berm, (c) submerged**

## Design approach for wave and current actions on mound breakwaters

### General

(1) An appropriate design approach should be implemented based on principles defined in Clauses 4 and 5, as provided in 7.2.2 and 7.2.3.

(2) After the relevant consequence class (CC) of the structure has been established (see Clause 4 and EN 1990:2023, Clause A.6), the uncertainty associated with the hydrodynamic estimation approach (HEA, see 4.6) and the tools available involving the response parameters for the specific structure should be defined.

NOTE 1 Representative cases of low to medium structure response uncertainty in mound breakwaters are those structures whose limit states are attained after two or more crossings of the design threshold, and when enough data is available from records of similar structures built under similar conditions:

* breakwaters whose slope protection is made of two armour layers;
* berm breakwaters;
* reshaping submerged breakwaters.

NOTE 2 Representative cases of high structure response uncertainty in mound breakwaters are:

* breakwaters whose slope protection is made of a single layer of armour units;
* armour units not widely used in practice;
* the crest structure is subject to impulsive loads or heavy wave overtopping;
* the toe of the structure is attacked by breaking waves;
* a breakwater head is under severe wave attack;
* non-standard build-up of layers is implemented;
* the design of berm breakwaters or submerged ones is not supported by enough data and widely accepted formulae.

### Return periods for the verification of serviceability limit states

(1) For the verification of serviceability limit states of mound breakwaters, the characteristic value of the hydrodynamic load may be determined either with the characteristic marginal or joint return periods defined in Clause 4 (see Table 4.5 (NDP)), or with the alternative case-specific characteristic marginal or joint return periods defined hereafter.

NOTE 1 Relevant serviceability limit states can be the limited displacement (LDi) of protecting armour units and the limited wave overtopping. The limited displacement serviceability limit state is similar to the “start-of-damage” state introduced in the Rock Manual (2007).

NOTE 2 Designers will make their decision to use characteristic return periods or alternative case-specific characteristic return periods by considering the sensitivity of rubble mounds’parts (toe, roundhead, number of layers etc.) as well as the level of safety of the formulae yielding allowable limit values of effect of actions in the relevant limit state conditions.

NOTE 3 Indicative damage parameters and allowable displacements from the literature can be found in Table C.1.

(2) The alternative case-specific characteristic values of the hydrodynamic loads may be defined as nominal values using conservative alternative case-specific characteristic marginal or joint return periods, without partial factor, using the importance factor *φ*I defined in Table 4.4 (NDP), which can be adjusted to the design service life of the breakwater, *T*lf, as soon as *T*lf ≥ 20 years.

NOTE 1 The alternative case-specificcharacteristic value is determined as the 400-year marginal return period, unless the National Annex gives a different value.

NOTE 2 When a two-variables joint statistical analysis of the metocean parameters is performed, the alternative case-specificcharacteristic value is determined as the 1 400-year joint return period, unless the National Annex gives a different value.

NOTE 3 This document does not propose any value of the alternative case-specificjoint return period for use in a three- or more variables statistical analysis.

(3) When the physical processes are relatively simple/benign so that statistical analyses might not be needed (HEA1), the alternative case-specificcharacteristic values may be appraised as an estimate of the values yielded by the alternative case-specificcharacteristic return periods.

(4) The combination rules using marginal or joint distributions of the metocean parameters (Marginal or Joint Deep-sea Extremes Method) are similar to those given in 4.9.

NOTE The marginal return periods of the accompanying metocean parameters (the “alternative case-specificcombination marginal return period”) are given in Table 7.1 (NDP) according to the level of dependence with the leading metocean parameter, in each high/low water level design situation.

Table 7.1 (NDP) — Synthesis of return periods and importance factors in the direct determination of the alternative case-specificcharacteristic hydrodynamic load for the verification of serviceability limit states of mound breakwaters in permanent/transient design situations

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Conse-quence class | Importance factor (φI)a | Alternative case-specificcharacteristic marginal return period (leading metocean parameter)b | Alternative case-specificcombination marginal return period (accompanying metocean parameter)c | Alternative case-specificcharacteristic joint return periodd |
| CC3 | 2,0 | 800 y | Very High dependence: 530 y | 2 800 y |
| High dependence: 140 y |
| Medium dependence: 30 y |
| Low dependence: 7 y |
| CC2 | 1,0 | 400 y | Very High dependence: 270 y | 1 400 y |
| High dependence: 75 y |
| Medium dependence: 20 y |
| Low dependence: 6 y |
| CC1 | 0,5 | 200 y | Very High dependence: 140 y | 700 y |
| High dependence: 40 y |
| Medium dependence: 13 y |
| Low dependence: 6 y |
| a The importance factor *φ*I is given in Table 4.4 (NDP).  b The alternative case-specificcharacteristic marginal return period is equal to the value given in 7.2.2(2), Note 1, multiplied by the importance factor *φ*I.  c When the degree of dependence is not known, the “medium dependence” figure can be used  d The alternative case-specificcharacteristic joint return period is equal to the value given in 7.2.2(2) Note 2, multiplied by the importance factor *φ*I. Figures are valid for a two-variable joint analysis only. | | | | |

EXAMPLE As an example of the marginal deep-sea extremes method in the verification of a serviceability limit state of a CC2 rubble mound breakwater with the alternative case-specificcharacteristic value, with medium dependence between wave height and still water level, the 400-year offshore (deep-sea) wave is considered together with the 17-year still water level as calculated in the low WL situation or in the high WL situation; the 400-year still water level calculated in the low WL situation or in the high WL situation is considered together with the 17-year offshore (deep-sea) wave. Those metocean events are transferred to the site of the (coastal) structure and the characteristic value of the hydrodynamic load can be calculated with Turkstra’s rule.

### Return periods for the verification of ultimate limit states

(1) For the verification of ultimate limit states of mound breakwaters, the design value of the hydrodynamic loads shall be determined directly as nominal values using the design marginal or joint return periods defined hereafter, without partial factor, using the importance factor *φ*I defined in Table 4.4 (NDP), which can be adjusted to the design service life of the breakwater, *T*lf, as soon as *T*lf ≥ 20 years.

NOTE 1 The design value for the verification of ultimate limit states of mound breakwaters is determined as the 2 000-year marginal return period, unless the National Annex gives a different value.

NOTE 2 When a two-variables joint statistical analysis of the metocean parameters is performed, the design value for the verification of ultimate limit states of mound breakwaters is determined as the 6 000-year joint return period, unless the National Annex gives a different value.

NOTE 3 This document does not propose any value of the design joint return period for use in a three- or more variables statistical analysis.

NOTE 4 Relevant ultimate limit states can be the severe displacement (SDi) of protecting armour units and the severe wave overtopping. The severe displacement ultimate limit state (SDi) is similar to the “failure state” introduced in the Rock Manual (2007).

NOTE 5 The severe displacement ultimate limit state (SDi) determines the time-dependent performance of the armour layers.

NOTE 6 Indicative damage parameters and allowable displacements from the literature can be found in Table C.1.

(2) When the physical processes are relatively simple/benign so that statistical analyses might not be needed (HEA1), the design values may be appraised as an estimate of the values yielded by the design return periods.

(3) The combination rules using marginal or joint distributions of the metocean parameters (Marginal or Joint Deep-sea Extremes Method) are similar to those given in 4.9.

NOTE The marginal return periods of the accompanying metocean parameters (the “design combination marginal return period”) are given in Table 7.2 (NDP)according to the level of dependence with the leading metocean parameter, in each high/low water level design situation.

Table 7.2 (NDP) — Synthesis of return periods and importance factors in the direct determination of the design hydrodynamic load for the verification of ultimate limit states of mound breakwaters in permanent/transient design situations

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Conse-quence class | Import-ance factor (*φ*I)a | Design marginal return period (leading metocean parameter)b | Design combination marginal return period (accompanying metocean parameter)c | Design joint return periodd |
| CC3 | 2,0 | 4 000 y | Very High dependence: 1 800 y | 12 000 y |
| High dependence: 370 y |
| Medium dependence: 45 y |
| Low dependence: 6 y |
| CC2 | 1,0 | 2 000 y | Very High dependence: 900 y | 6 000 y |
| High dependence: 190 y |
| Medium dependence: 30 y |
| Low dependence: 6 y |
| CC1 | 0,5 | 1 000 y | Very High dependence: 450 y | 3 000 y |
| High dependence: 100 y |
| Medium dependence: 18 y |
| Low dependence: 5 y |
| a The importance factor *φ*I is given in Table 4.4 (NDP).  b The design marginal return period is equal to the value given in 7.2.3(1), Note 1 multiplied by the importance factor *φ*I.  c When the degree of dependence is not known, the “medium dependence” figure can be used  d The design joint return period is equal to the value given in 7.2.2(1), Note 2 multiplied by the importance factor *φ*I. Figures are valid for a two-variable joint analysis only. | | | | |

EXAMPLE 1 As an example of the marginal deep-sea extremes method (in the verification of the severe displacement ultimate limit state of a CC2 rubble mound breakwater) with medium dependence between wave height and still water level, the 2 000-year offshore (deep-sea) wave is considered together with the 30-year still water level as calculated in the low WL situation or in the high WL situation ; and the 2 000-year still water level calculated in the low WL situation or in the high WL situation is considered together with the 30-year offshore (deep-sea) wave. Those metocean events are transferred to the site of the (coastal) structure and the design value of the hydrodynamic load is calculated with Turkstra’s rule.

EXAMPLE 2 As an example of the joint deep-sea extremes method (in the verification of the severe displacement ultimate limit state of a CC2 rubble mound breakwater) with medium dependence between wave height and still water level: plot the relation between wave heights and water levels at 2 000-year joint return period; consider the 30-year (combination marginal return period) still water level as calculated in the low WL situation or in the high WL situation, together with the wave height red on the 6 000-year joint return period relation; consider also the 30-year (combination marginal return period) wave height together with the water level as calculated in the low WL situation or in the high WL situation, red on the 6 000-year joint return period relation; transfer those metocean events to the site of the coastal structure and calculate the hydrodynamic load with Turkstra’s rule.

(4) Mound breakwaters should be considered for the specific design issues listed in 7.3.

(5) The cumulative damage of armour revetments of coastal structures (resilience) should be assessed through physical model testing (DA4) for structures classified in CC2 and CC3.

NOTE A qualitative idea about how the level of resilience can be obtained is presented in Clause C.6.

## Wave and current actions

### General

(1) The effects of the structure (related to, for instance, a horizontal bend of axis, wave attack at an angle, or the head) on the local storm-representative waves should be considered.

NOTE Where the weight of the armour units changes along a mound breakwater, the lengthwise transitions between typical cross-sections can produce a final seaward slope of the structure with no dent at the meeting line of the two cross-sections to prevent wave action from concentrating along that indentation and dislocating the armour units. Such transitions can be carefully implemented and given the required length. The transition zone can be designed based on the most critical wave and current conditions along its entire length.

(2) By convention the hydrodynamic loads on the armour layers of berm breakwaters are the local wave height and the square of the local current velocity.

### Wave action on the seaward slope

(1) The effect of seaward slope armour units on wave actions shall be considered.

NOTE 1 Waves can break on the sloping front of the mound due to depth limitation resulting in loading on the armour units, run-up, run-down as well as related pore pressure variations and porous flow inside the structure.

NOTE 2 The impact of waves on a breakwater depends on the stage of instability of the waves, and actions from non-breaking, breaking and broken waves will be different in each sea state.

NOTE 3 The wave action on the seaward armour layer is affected by the wave reflection from the structure. The armour stability of the front slope increases with the increase in porosity and permeability. Further, a low-crested structure where significant wave overtopping occurs experiences less wave loading on the armour units of the seaward slope than a structure with a high freely extending crown wall exposed to direct wave actions, other parameters kept equal. Commonly, the critical moment when the stability of the armour elements is most severely tested occurs during the wave down-rush on the seaward slope.

NOTE 4 Due to the complexity of the flow around each armour unit, it is not possible to evaluate the wave actions upon the individual armour units. Instead, the required mass of those units to remain stable can be determined through concepts leading to semi-empirical formulae with coefficients determined through model tests and in some cases also through prototype observations. Hence, the wave action is given in terms of parameters describing the incoming wave field.

(2) The validity range of semi-empirical formulae shall not be exceeded, unless it can be proven to be conservative, or have been verified by previous model tests, or in type (a) breakwaters by documented full-scale experience.

(3) For cases that do not meet the criteria in 7.3.2(2), hydraulic model testing should be undertaken.

NOTE For guidance on applicable formulae for this topic, see C.3.4.

### Wave actions on the seaward toe

(1) The effects of armour units forming the seaward toe on wave actions shall be considered.

NOTE 1 The toe’s integrity is mainly affected by the wave-induced flow during run-down.

NOTE 2 Where breaking waves can act directly on the toe, seabed scour can endanger its stability, see information in C.3.6.

(2) In depth limited conditions the governing water level should be determined by considering the associated wave height. Water level variations due to climate change should be accounted for.

NOTE In case of non-depth-limited waves the most critical design situation will generally be associated with low water levels.

(3) Model tests should be carried out for toe stability when no formulae are applicable.

### Wave overtopping

(1) The effect of wave overtopping water and spray should be considered.

NOTE For guidance on above issues, see C.3.7.

(2) Empirical formulae based on model tests for assessment of average wave overtopping discharge may be used within their validity range and considering the scatter in the results.

(3) Where wave overtopping is a critical factor model tests should be performed, when the existing formulae incorporate high uncertainties.

NOTE Threshold wave overtopping values are given in the EurOtop (2018).

### Wave action on the rear armour slope

(1) The effects of the rear armour layer on the wave actions shall be considered.

(2) The effect of overtopping water on the rear slope should be considered.

NOTE Overtopping water hitting the rear slope can cause damage to it and thus endanger the stability of the breakwater. Also, large pore pressure gradients can cause a push-out load on the rear side surface blocks. This effect is usually enhanced by the presence of crown wall structures. Similar instability effects on the rear armour can be observed in cases of highly overtopped, submerged or permeable structures.

(3) When the seaward slope is armoured by artificial units the rear slope should be assessed through physical model tests for structures classified as CC2 and CC3.

NOTE For structural response to hydraulic actions on the rear slope, more information can be found in Annex C.

### Wave action on geotechnical member

(1) Wave loading on the slopes should be considered when examining armour protected slopes and ground foundations

NOTE In association to wave loading it is common to assume a high level of the internal phreatic horizon as the most critical condition.

(2) The internal phreatic set-up due to wave loading should be taken into consideration since the said set-up influences pore pressure.

(3) Pore pressures in the mound and in the seabed may be estimated by using calibrated numerical models. They may also be assessed by use of model tests with due consideration of scale effects.

(4) Liquefaction potential of foundation layers under wave action should be assessed, in adverse combinations of soil properties, water depth, and wave properties.

### Wave actions on roundheads

(1) Due to the complexity of the wave actions involved, roundheads should be checked in hydraulic three-dimensional model tests for HEA2/HEA3.

NOTE 1 Roundheads are more exposed to damage due to wave action than the corresponding trunk section under similar wave loading. This is due mainly to refraction effects induced by the roundhead that tend to concentrate wave energy there. Also, the conical shape of roundheads causes higher overflow velocities and lesser lateral support of the armour units. These latter factors can induce increased damage to the rear section of the roundhead under certain conditions.

NOTE 2 For further guidance, see C.3.8.

### Wave action on breakwater crest and crown walls

(1) Conditions of severe wave overtopping shall be checked by physical model tests.

NOTE 1 The model tests are used to check armour stability at mound crests.

NOTE 2 A freely standing part of a crown wall can experience impulsive wave loads when directly exposed to waves, while a wall or part of a wall sheltered by armour blocks gets pulsating wave loads. The crown wall base will experience wave-induced forces if situated below the envelope of the phreatic surface. Uplift forces are also generated by waves hitting a recurved wall front, i.e., with an offshore nose for reduction of wave overtopping.

(2) For assessment of the crown wall, simultaneous front loading and uplift forces should be considered.

NOTE Impulsive pressures on the crown wall front are characterized by high peak values of very short duration.

(3) The influence of impulsive pressures on the crown wall shall be evaluated with due consideration of the dynamic interactions with the foundation.

(4) The influence of significant air entrained in the breaking waves hitting the crown wall shall be considered when transforming model test loadings into prototype.

(5) Actions from wave-induced pressures, wave loads transmitted through armour units resting against the wall should be accounted for. The level of the bottom of the crown element should be considered with respect to the sea level since it has an important effect on the value of the wave action upon that element.

NOTE Guidance on formulae that can be used to estimate wave forces on crown walls of simple shape can be found in C.3.10.

(6) The possibility of frontal waves hitting the wall should be considered.

NOTE 1 Formulae for wave loads on crown walls are of empirical character and based only on two-dimensional model tests.

NOTE 2 See 7.3.1.

(7) The crown wall should be evaluated through model tests when due consideration of three-dimensional effects caused by obliqueness of the waves is required.

NOTE The model tests are used to check the stability of the crown wall.

(8) For the verification of ultimate limit states of structural crests and crown walls, the most conservative provision out of 7.2 and 4.8(2) is kept for the design.

### Wave and current action on filter layers and underlayers

(1) Migration of finer mound materials into coarser mound materials and further out into the free water, caused by wave-induced pressure gradients resulting in local flow of pore water, should be avoided.

NOTE 1 Empirical formulae for the gradation of filters and the relative stone sizes between adjacent layers including the foundation soil, based on physical tests and prototype observations, are available.

NOTE 2 Relatively large stones in under-layers provide better interlocking with armour units and reduce the critical pull-out hydraulic action to such stones due to locally increased permeability.

NOTE 3 Local currents can develop eroding conditions on underwater exposed under-layers at both slopes of the structure.

NOTE 4 Further guidance on filter layers can be found in the Coastal Engineering Manual VI.5-3 and the Rock Manual (2007), 5.2.2.10.

NOTE 5 For further guidance on erosion of under-layers, see C.3.11.

### Wave action related to stresses in armour units

(1) Rocking of armour units increases forces and stresses in the units and should be avoided.

(2) The long-term abrasion and breakage performance of armour units, where wave actions cause rocking or cyclic movements of the rocks, should be considered.

NOTE 1 Wave actions on stones for building berm breakwaters can create crushing actions on the stones on the seaward side, during the reshaping phase of the structure.

NOTE 2 For further guidance, see The Rock Manual (2007).

### Wave and current actions related to local seabed scour

(1) The scour depth from waves and currents that can occur close to and around structures on an erodible seabed should be considered.

NOTE 1 Wave and current actions can cause scour close to and around structures on an erodible seabed. Currents, if present, are often, but not always, the major cause of scour.

NOTE 2 For further guidance on seabed scour, see Annex C.

# Wave and current actions on vertical face breakwaters

## Introduction and structure types

(1) This Clause 8 contains provisions for wave and current actions relevant to the verification of vertical face breakwaters.

NOTE 1 For further guidance, see Annex D.

NOTE 2 A vertical face breakwater is assumed in this Clause 8 to be a simple structure having a vertical or nearly vertical front wall extending down to the seabed and resting on top of a bedding layer. A usual type of such breakwaters is normally made of reinforced concrete caissons filled with rubble, sand, water or other material. A superstructure made of placed-in concrete is constructed on top of it. The cross section of the superstructure can have a sloped front or shape aiming at reducing wave overtopping. Variations of vertical face breakwaters can also be met, as with the composite breakwaters (Clause 9).

NOTE 3 Other types of vertical face breakwaters include sheet-pile breakwaters and breakwaters with apertures. Sheet-pile breakwaters can be constructed either as cellular batteries of various shapes or as sheet-pile walls, single or double faced. Voids are usually filled with concrete or stone material. Breakwaters with openings are usually formed by two chambers divided by a vertical wall, with the face of the offshore chamber displaying openings to absorb wave energy and decrease the degree of wave reflection by the breakwater. This special type is mostly met close to port entrances where wave reflection provides a nuisance to navigation. Floating breakwaters are treated in Clause 11.

NOTE 4 Figure 8.1 gives a typical simplified cross-section of a vertical face breakwater, considered in this Clause 8.

A black background with white lines

Description automatically generated

Key

|  |  |
| --- | --- |
| a | toe protection |
| b | vertical face |
| c | in situ construction |
| d | precast unit |
| e | bedding layer |

Figure 8.1 — Definition sketch of a vertical face breakwater

## Design approach for wave and current actions on vertical face breakwaters

(1) An appropriate design approach should be implemented based on principles defined in Clauses 4 and 5.

NOTE 1 The required reliability level is warranted by following the provisions of EN 1990:2023, Clause A.6.

EXAMPLE Examples of high structure response uncertainty in vertical face breakwaters are:

* structures under critical breaking wave conditions, in particular with respect to wave overtopping and the capping wall under such conditions;
* toe protection when the offshore face is used also for short-time berthing of large propelled vessels;
* structures built on soft clays in areas of high seismicity.

NOTE 2 For available physical and numerical modelling methods, see Clause 12. Numerical modelling methods usually refer to geotechnical aspects of the design and wave overtopping.

(2) When dynamic analysis is used in design verifications, the maximum wave height may be used associated with allowable displacements of the structure.

NOTE 1 Allowable displacements are typically due to sliding and tilting.

NOTE 2 Sliding displacement can normally be in the range of up to 10 cm.

(3) For the verification of wave overtopping ultimate limit state, 7.2.2 and 7.2.3 shall apply.

(4) For the verification of other ultimate limit states, 4.8(2) shall apply.

## Hydrodynamic loads due to waves and currents

### Types of wave actions

(1) The wave action to be used in design verifications should be a representative value of a storm based on the spectral height *H*m0.

NOTE 1 The main actions of waves on vertical face breakwaters are the wave pressure on the front face and the uplift on the bottom of the main body, if applicable. These can govern the overall stability of the main body against sliding, overturning, and foundation failure as well as the integrity of its structural elements. The wave action on vertical face structures very much depends on the type of incoming waves, i.e. unbroken, breaking, or broken waves. It is of prime importance to be able to identify the type of waves to be taken in the design, especially in a random wave environment. This is so because actions by breaking waves can be far more intense than the corresponding ones by unbroken waves. Additionally, the breaker type of waves breaking against vertical faces plays a significant role in evaluating the associated loads, taking into consideration the air pocket entrained in the wave face as well as the duration of the impact.

NOTE 2 The surf zone width assumed for design purposes is directly related to the storm-representative wave height. Thus, a structure placed outside the design surf zone can experience breaking waves if the local water depth allows this process to take place for waves higher than the design ones. This complexity, especially when linked to vulnerability, can warrant use of advanced probabilistic methods to evaluate wave actions.

NOTE 3 Under special circumstances, vertical face breakwaters can be subjected to wave actions from the lee side.

NOTE 4 The angle of wave incidence upon the breakwater plays a role in reducing the wave action on the vertical face. This reduction is also related to the structure length normalized with respect to the wavelength. Physical model tests suggest that the mentioned reduction is larger in cases of breaking rather than non-breaking waves.

NOTE 5 Wave reflection by plain vertical-face breakwaters gives normally high reflection coefficients close to unity. Hence the wave energy close to the offshore face of the structure can be quite higher than the incident one. This can present a hazard at port entrances especially for small crafts.

NOTE 6 For estimation of the bulk reflection coefficient, see information in D.3.9.

### Wave pressure, uplift, and buoyancy

(1) The horizontal impact pressure and uplift exerted upon the main body due to breaking wave action should be evaluated by means of hydraulic model tests or appropriate calculation models.

(2) The wave pressure on its rear wall should be considered if the waves diffracted from the head towards the rear of the breakwater are of appreciable height.

(3) The buoyancy of the immersed part of the main body below the appropriate design water level shall be considered in the stability analysis of the breakwater.

NOTE Under a certain combination of wave conditions, breakwater geometry, and bathymetric features, impulsive breaking wave pressure can be exerted upon the breakwater face during a period of the order of several hundredths of a second. The impulsive pressure is characterized by high peak intensity of very short duration. Such pressures can be an order of magnitude higher than the corresponding due to non-breaking waves, depending on the entrapped air by the facial shape of the impulsive wave.

(4) When the action of impulsive pressures overall or on elements of the main body needs to be considered in the design, it should be evaluated by taking its duration into account together with its peak intensity, see 8.2.

NOTE 1 Model tests under breaking wave conditions are quite demanding and difficult to evaluate the results thereof, mainly due to scale effects. Still, for many complex situations model tests can be the best tool to get any reliable answer.

NOTE 2 For guidance on estimating actions due to nonbreaking waves on vertical wall, see D.3.4. For guidance on estimating actions due to breaking waves on vertical wall, see D.3.5. For guidance on estimating actions due to uplift force, see D.3.6.

NOTE 3 Methods on physical modelling are covered in Clause 12.

### Wave overtopping

(1) Depending on the use of the area behind a breakwater, the amount of wave overtopping and the characteristics of waves transmitted behind the breakwater shall be examined.

NOTE Commonly used wave overtopping threshold values can be obtained in the EurOtop (2018). Guidance on estimating the wave transmission factor through the value of overtopped waves can be found in the EurOtop (2018) and in Clause D.3.

### Effect of wave action on geotechnical failure

(1) For the effect of wave action on geotechnical failure, 7.3.6(4) shall apply.

NOTE The main body of a vertical breakwater is subject to the wave pressure on its front wall, superstructure and the rear wall, the uplift on its bottom, the buoyancy to its immersed part.

### Wave and current actions related to local seabed scour

(1) The scour depth close to and around vertical breakwaters on an erodible seabed should be addressed by considering the reflected waves and the resulting near bottom streaming.

(2) The study of wave and current impact on the seabed should extend to a depth where scour would not have an impact. The types of wave action noted in 8.3.1 should be considered.

# Wave and current actions on composite breakwaters

## Introduction and structure types

(1) This Clause 9 contains provisions for wave and current actions relevant to the verification of composite breakwaters.

NOTE 1 For further guidance, see Annex D.

NOTE 2 The main variation of the simple vertical face breakwater (Clause 8) refers to the composite breakwaters. These are formed by vertical face breakwaters either sitting on a submerged rubble mound (vertical-composite) or protected by a rock slope at their offshore face (horizontal-composite). Vertical-composite breakwaters are placed on an engineered mound that is submerged at all tidal levels. They are usually implemented in order to optimise the dimensions and the cost of the vertical face unit in larger water depths, while horizontal-composite breakwaters are usually implemented in order to reduce the wave action on the vertical face of the breakwater. Figure 9.1 gives typical cross-sections of these two types of composite breakwaters.



Figure 9.1 — Typical cross-sectional sketches for (a) vertical-composite breakwaters and (b) horizontal-composite breakwaters

NOTE 3 When the mound in vertical-composite breakwaters is replaced by piles, see Clause 6 for wave and current action.

NOTE 4 In horizontal-composite breakwaters the front face of the main body is protected by a mound of uniform rock or prefabricated units or a combination thereof. The mound can also be formed by layers of material of varying unit size. The purpose of the mound is to reduce wave action on the vertical face of the main body as well as to reduce wave reflection from the structure. Hence the main agents of the wave action in this configuration are the wave loadings upon the mound and on the main body of the breakwater, including pressures on the vertical face, uplift and buoyancy.

(2) The provisions of Clause 8 and the related guidance in Annex D apply also to the structures of Clause 9, unless modified or substituted accordingly in this Clause.

## Design approach for wave and current actions on composite breakwaters

(1) An appropriate design approach should be implemented based on principles defined in Clauses 4 and 5, as noted in 9.2.

NOTE Representative cases of high design uncertainty in composite breakwaters, in addition to those mentioned in 8.2, are for vertical-composite breakwaters the toe protection and for horizontal-composite breakwaters when a rather impermeable protective mound is used.

(2) For the mound part of the structure, the provisions in 7.2.2 and 7.2.3 shall apply.

NOTE For available physical and numerical modelling methods refer to Clause 12. Numerical modelling methods usually refer to geotechnical aspects of the design and to wave overtopping.

(3) For the vertical face part of the structure, the provisions in 8.2 shall apply.

(4) The effects of the structure on the local storm-representative waves should be considered.

(5) For the verification of ultimate limit states of composite breakwaters, the most conservative provision out of 7.2 and 4.8(2) shall be used for the design.

## Wave and current actions on vertical-composite breakwaters

### Main types of wave action

(1) Critical wave conditions including the water level variation should be identified for non-breaking, broken, or breaking waves.

NOTE See 9.2(3).

(2) When hydraulic model tests are required for mound roundheads then three-dimensional tests should be adopted for the design of the armour units and their placement.

(3) In addition to the wave action on the main body of the breakwater, associated actions upon the mound shall be considered.

(4) The berms covered with armour-units on both sides of the main body of the mound (usually a rubble mound) for protection against wave and current actions, should be considered either by semi-empirical formulae or by appropriate stability model tests in cases of DA4 design method, see Table 4.3.

NOTE 1 For guidance on avoiding wave breaking on the breakwater face, see D.4.4.

NOTE 2 For further guidance on wave action on the mound armour, see D.4.5.

### Wave overtopping

(1) 8.3.3 shall be followed regarding wave overtopping in a composite breakwater on an insignificantly influencing mound.

(2) If the mound is significantly influencing the wave regime, model tests should be performed.

NOTE For further guidance on wave overtopping, see D.4.7.

### Wave action on mound filter layers

(1) 7.3.9 should be followed regarding wave actions relevant to the mound filter layers.

### Wave action on prefabricated armour units

(1) When the armouring of the offshore slope of the mound by prefabricated units is subjected to extreme conditions, 7.3.10 shall be applied.

### Effect of wave action on geotechnical failure

(1) In vertical-composite breakwaters the effect of wave action, when examining geotechnical slip failure, shall be considered on both the main body of the breakwater and the submerged mound. Hence 7.3.6 and 8.3.4 shall be applied.

NOTE In common cases the resulting action on the main body under wave attack can be transferred to the mound crest through the associated stresses at the base of the vertical face body.

### Wave and current actions at the vertical face toe

(1) 8.3.5 shall be applied when estimating the wave and current actions at the mound crest elevation instead of the seabed.

NOTE 1 Wave induced flow in front of the main body can cause instability of armour units of the mound crest of vertical composite breakwater.

NOTE 2 For guidance on this aspect, see D.4.9.

### Wave and current actions on the seaward toe of the mound

(1) For wave and current actions on the seaward toe of the mound, the provisions in 7.3.3 should apply where relevant.

NOTE Wave-induced flow, combined with ambient current when applicable, can cause scour of the seabed at the offshore toe of the mound. This can cause damage to the mound and consequently to the main breakwater body, such as tilting.

## Wave and current actions on horizontal-composite breakwaters

### Main types of wave action

(1) The methods applied for the estimation of the actions on the protective mound and on the vertical face unit shall take into account the interrelation of these actions.

NOTE For guidance on evaluating wave action on this type of breakwaters, see D.5.4.

(2) Stability testing in hydraulic models should be undertaken to assess both the main body and the mound units, i.e. design approach DA4 (Clause 4 and Clause 12).

### Wave overtopping

(1) Wave overtopping in horizontal-composite breakwaters shall be estimated by considering the higher crest elevation between the mound and the main body.

NOTE 1 It is common that the mound crest is higher than or at the same level with the main body crest.

NOTE 2 For information on wave overtopping rates, see C.3.7.

### Effect of wave action on geotechnical failure

(1) The wave actions shall be considered in geotechnical analyses, either pertaining only to the mound, as slope or foundation ground effects, or to both components of the composite structure, as foundation ground effects.

(2) In these analyses the critical water level should be considered as modified by waves, since that level depends on the pore pressures developed in mound and seabed material that can be modified by waves; also, on whether the analysis refers to mound-only or mound-body cases.

### Wave action on roundheads

(1) For wave action on roundheads, the provisions in 7.3.7 shall apply.

### Wave action on breakwater crest

(1) As the crest of a horizontal-composite breakwater can experience severe wave action in overtopped structures, the action distributions should be considered. Loads should include wave-induced pressures acting directly on the wall and wave action transmitted by armour units resting against the wall.

(2) The stability of the crown wall should be determined from model tests with due consideration of both three-dimensional effects caused by obliqueness of the waves and of scale effects.

NOTE Formulae for wave loads on crown walls are of empirical character and based only on two-dimensional model tests.

(3) Armour and crown wall stability in heavily overtopped structures shall be checked by hydraulic model tests.

NOTE 1 For DA4 approach, see Clauses 4 and 12.

NOTE 2 For guidance on design formulae that can be used to estimate wave force on crown walls of simple shape, see C.3.10.

### Wave action on filter layers

(1) If the protective mound contains layers made of different size material, the wave-induced pressure gradients resulting in local flow of pore water should be considered.

NOTE 1 Local wave induced flow is responsible for eventual migration of fine material into the pores of coarser material and eventually out into the free sea water.

NOTE 2 The presence of the reflective face of the main body of the breakwater enhances the pressure gradients and subsequently the intensity of the local flow.

(2) When using formulae for the gradation of the layer materials based on models or prototypes other than those directly related to the horizontal-composite breakwater, account should be taken of the increase of the local flow intensity due to the increase of the driving pressure gradients by the reflective face of the main body of the breakwater.

NOTE For further guidance, see the corresponding formulae used in the conventional rubble mound breakwaters in Clause C.3.

### Wave action related to stresses in armour units

(1) For wave action related to stresses in armour units, the provisions in 7.3.10 shall apply.

(2) Calibrated semi-empirical formulae for stresses can be used, but the presence of the impermeable face of the main body of the breakwater should be considered, as far as the reflected waves from it modify the hydrodynamic loading on the prefabricated units, see also 7.3.10.

### Wave and current actions related to local seabed scour

(1) Due consideration should be given to the impact of wave and current actions with respect to local seabed scour.

NOTE Waves and currents can cause scour close to and around a horizontal-composite breakwater on an erodible seabed. The study of wave and current impact on the seabed normally extend to a depth where scour, under the most critical design conditions, would not threaten the structure. In particular, the toe of the vertical face, the foundation and the toe of the mound are typically examined for scour prevention.

(2) In case of wave–induced flow during run-down of non-depth-limited waves, account should be taken of the most critical situation generally associated to low water levels.

NOTE The wave–induced flow during run-down can affect the stability of the seaward rubble mound toe.

(3) Special consideration should be taken where breaking waves can act directly on the toe.

NOTE 1 Seabed scour can endanger the stability of the breakwater.

NOTE 2 Empirical formulae exist for assessment of toe block stability, based on model tests.

(4) When assessing the mound toe, the wave reflection by the main body in a horizontal-composite breakwater should be considered.

NOTE A starting point for the evaluation of toe protection may be the corresponding issue for conventional rubble mound breakwaters, see C.3.6.

(5) Wave and current actions should also be examined in relation to seabed erosion at the toe of the vertical face and the foundation layer of the protective mound.

(6) Model tests should be undertaken to check the mound toe and foundation ground stability including the vertical face toe, when no applicable formulae exist.

# Wave and current actions on coastal embankments

## Introduction and structure types

(1) This Clause 10 contains provisions for wave and current actions relevant to the verification of coastal embankments.

NOTE 1 For further guidance, see Annex E.

NOTE 2 Coastal embankments are linear structures built along the shore in order to protect the “banks” of the land edge alongside the coastline from coastal erosion if required. They are often protecting the seaward slope of a coastal road, a land reclamation or other utility. The influence of the foreshore level is crucial upon the waves attacking the embankment. The longshore variability of this level tends to concentrate wave energy at places; it also governs toe and structure foundation levels along the embankment.

NOTE 3 Embankments can be divided in two structural types: revetments and seawalls. Typically, revetments display a sloping offshore face and seawalls a nearly vertical one.

NOTE 4 Pore pressures in coastal embankments is an important issue that plays a significant role in assessing the potential of geotechnical failures and the stability of the embankment face. These pressures are controlled by the quality of the internal material of the structure and the phreatic gradient between the seawater and the underground water level.

NOTE 5 Coastal revetments are man-made sloped structures parallel to the shore to protect the hinterland against erosion. Revetments function mainly through absorbing wave energy by inducing wave breaking controlled by water depth. In several cases waves can break before reaching the structure, during a certain percentage of time. Revetments are generally armoured by different materials such as grass, asphalt, stone, or concrete. They can be placed along coastal shorelines or around estuaries. These structures are characterised by mild slopes and quite often by a berm which provides access for maintenance, reduces wave run-up and wave overtopping. Their cover layers can range from quite permeable, e.g. formed by natural rocks, through to impermeable, e.g. covered by concrete or masonry. Underneath the cover layer filter layers and/or geotextiles can be placed.

NOTE 6 Seawalls are onshore or foreshore structures generally parallel to the shoreline. They are built as vertical or nearly vertical face structures, such as gravity concrete walls, steel or concrete sheet pile walls, stone filled cribworks, etc. The principal function of seawalls is to reinforce a part of the coastal profile and to protect land and infrastructure from the erosive action of waves and wave overtopping. This is done primarily through wave energy reflection rather than absorption as done in revetments.

NOTE 7 Following the definition given above, three main structural components can be distinguished in a seawall:

* the body, which includes the front face structure and the material behind it;
* the toe;
* the crest or capping beam.

## Design approach of wave and current actions on coastal embankments

(1) An appropriate design approach should be implemented based on principles defined in Clause 4, Clause 5.

(2) The framework described in 7.2.2 and 7.2.3 can be applied to revetments, and the framework described in 8.2 can be applied to seawalls.

(3) The effect of waves at the shoreline upon the phreatic gradient between the seawater and the underground water level should be considered, in particular for waves close enough to the embankment face.

EXAMPLE Examples of structures with high design uncertainty, excluding environmental loading, are embankments in areas of very high underground water table, or near the shoreline resulting in waves breaking against the structure during a significant part of time.

## Revetments

### Type of wave and current actions

(1) The following wave and current actions on coastal revetments should be considered:

* wave loading on the seaward slope of the structure including outward forces on the cover layer;
* hydraulic stability of revetment filters;
* wave run-up and run-down on the seaward slope;
* wave overtopping over the crest;
* scouring effects by waves and currents on the embankment toe.

(2) Wave actions on coastal revetments shall be evaluated considering parameters associated to the local tidal variations, such as:

* wave height;
* wavelength;
* water level.

(3) If relevant, the influence of the shape of wave spectra should be accounted for.

### Wave action on seaward slope

(1) Wave regime and type of protection shall be examined together, and armour material shall be selected accordingly.

NOTE The wave actions on the seaward slope of a revetment are to a large extent dependent on the type of cover layer used for its protection.

(2) Hydraulic model tests may be undertaken to assess the seaward slope.

(3) Empirical or semi-empirical formulae may also be used to assess the seaward slope.

(4) Wave-induced run-up and run-down, as governed by the permeability of revetment slope, should be estimated since excessive run-up leads to severe wave overtopping over the wall crest and high run-up and run-down velocities can cause erosion damages on the seaward slope.

NOTE 1 The run-up height depends on the wave breaker type.

NOTE 2 Guidance on calculating wave actions, including run-up, on revetments is provided in E.3.4.

(5) The crest elevation should be determined based on the allowable wave overtopping.

(6) The run-up and run-down flow, wave induced uplift forces underneath the revetment or cover layer, and wave impact loads on the seaward side of the embankment shall be considered when relevant.

### Wave action on seaward toe

(1) The seaward toe in a coastal revetment should be examined by hydraulic model tests or empirical formulae where wave-induced run-up and run-down during lower water levels are considered.

NOTE 1 The seaward toe in a coastal revetment can be important for the stability of the structure.

NOTE 2 For further guidance on addressing wave and current actions on the seaward toe, see E.3.5.

### Wave overtopping

(1) Wave-induced overtopping over the revetment crest, as influenced by the permeability of the seaward slope of the revetment, should be assessed.

NOTE Wave overtopping can damage the shoreward side and the operations it supports.

(2) Empirical formulae may be used to predict mean and peak wave overtopping discharges.

NOTE Allowable wave overtopping rates considering local conditions of the hinterland can be used to avoid damage the embankment.

(3) Where wave overtopping is a critical factor hydraulic model tests should be undertaken for its evaluation.

(4) The crest elevation of the embankment should be sufficiently high to prevent wave overtopping rates higher than the allowable rate.

NOTE 1 Values of allowable wave overtopping rates can be found in the EurOtop (2018).

NOTE 2 For information on tools that can be used to predict wave overtopping discharges, see E.3.6.

### Effect of wave action on geotechnical failure

(1) Semi-empirical and analytical models may be used to account for wave actions on the revetment that can cause geotechnical failures.

(2) For the effect of wave action on geotechnical failure, the provisions in 7.3.6(4) apply.

### Wave and current actions related to local seabed scour

(1) The scour depth close to and around structures on an erodible seabed should be considered.

(2) The toe susceptibility not only to direct wave action (see 10.3.3) but also to undermining due to wave and current actions should be considered.

## Seawalls

### Types of wave and current actions

(1) Wave actions on seawalls shall be evaluated according to Clause 5, considering the water depth and tidal variations in front of the seawall, through:

* wave height;
* wavelength;
* water level.

(2) The crest elevation should be verified for the allowable wave overtopping rate or volume and/or the wave run-up height.

(3) The run-up height may be evaluated, in cases of significant wave reflection and non-breaking waves, through the kinematics of the corresponding standing wave formed at the seawall face.

(4) The following factors associated with the stability of the structure components including body, toe and crest shall be considered:

1. horizontal wave forces including impact loading by breakers;
2. wave up-lift forces;
3. internal water pressure in the embankment and eventual seepage flow;
4. erosion effects undermining the structure toe.

### Wave reflection

(1) The influence of wave reflections from seawalls should be investigated.

NOTE Seawalls will reflect a rather high proportion of the incident wave energy. These reflected waves can have significant impact on the wave pattern and the sediment transport in the coastal zone in front of the seawall.

(2) Reflection coefficients may be estimated from empirical and semi-empirical formulae for perpendicular wave attack.

NOTE Further guidance on wave reflection by seawalls can be found in E.4.3.

### Wave actions on seaward toe

(1) Wave actions on the toe should be assessed by hydraulic model studies or through empirical formulae.

NOTE The main purpose of the toe structure of seawalls is to prevent undermining of the body of the seawall with shallow foundation. Failure of the toe can lead to total collapse of the structure.

### Wave overtopping

(1) Wave overtopping over seawalls should be examined in detail.

NOTE Wave overtopping can induce significant damage to the structure itself or any infrastructure behind the wall or can generate hazard for people living, travelling, or working immediately behind the structure.

(2) Due to the complexity of some seawall geometries where no wave overtopping formulae are available hydraulic model tests or numerical modelling should be used to assess the mean wave overtopping rate as well as individual wave overtopping volumes.

(3) Semi-empirical formulae based on hydraulic model tests and prototype investigations may be used to quantify wave overtopping in seawalls.

NOTE For directions on the use of wave overtopping formulae, see E.4.4.

### Wave-induced forces

(1) The type of wave action should be identified in such a wave that critical load combinations of horizontal wave loads, uplift forces and water levels are considered.

NOTE 1 For further relevant issues such as duration of wave impact forces, uplift, and buoyancy of the seawall, see 8.3.2.

NOTE 2 Seawalls with a vertical or steep front face can be exposed to wave loadings that might endanger the overall stability of the wall. Depending on the foreshore geometry, seawalls can experience both pulsating and impulsive wave loadings.

(2) Wave-induced forces may be assessed using semi-empirical formulae for two-dimensional conditions and simple geometries, i.e. vertical walls with or without berms.

(3) Wave forces should be assessed by hydraulic model tests for more complex geometries.

NOTE For guidance on estimating wave-induced forces on seawalls, see E.4.5.

### Seabed scour due to waves and currents

(1) The scour depth should be considered, especially when the exposed face of the structure is close to vertical.

NOTE 1 Waves and currents can cause scour close to and around structures on an erodible seabed.

NOTE 2 Since seawalls normally reflect wave energy, they induce significant offshore velocities close to the foundation of the structure body. These water particle velocities can weaken an erodible soil close to the structure’s toe and induce tilting and damage to the toe.

NOTE 3 Sloped structures are often preferred for coastal embankments if seawalls are not protected by a sloping face as in horizontal-composite breakwaters (see 9.4.8).

# Wave and current actions on floating structures

## Definitions and types of floating structures

(1) For coastal applications, a floating structure may be defined as a structure that is buoyant in water and has motions that are limited by mooring equipment.

NOTE 1 Different types of floating structures can be found in coastal areas depending on functionalities, for instance:

* floating breakwaters;
* floating system for aquaculture (e.g.fish farms);
* large floating quays for offshore berthing and mooring of vessels;
* moored floating structure supporting miscellaneous structure (e.g. support pile as part of floating bridge).

NOTE 2 Floating breakwaters are moored floating objects installed to reduce the height of waves approaching harbours, marinas etc. The cross-sectional shape of the object can vary, the most common one being the rectangular shape. Several such objects can be placed in a row thus composing a breakwater of the required length.

NOTE 3 The wave height reduction through floating breakwaters is mainly a function of the ratio wave length/width of the breakwater. Generally, floating breakwaters allow the long period waves to pass through while reducing the heights of the short period waves (typically period of 3s to 5s). Floating breakwaters are thus mainly used in relatively sheltered areas where only relatively short period waves are present.

(2) Generally, the performances of a floating structure (in terms of motions, forces, etc.) shall be evaluated taking into account the force-motion characteristics (stiffness) of the mooring system.

NOTE 1 Two main types of mooring devices are used for floating structures in coastal areas:

* spread mooring (e.g. catenary lines mooring);
* pile guided mooring.

NOTE 2 In the catenary lines mooring configuration, the floating structure is held in position with different mooring lines (composed of wires and/or chains, or fibre ropes) which feature sufficient length to make the lines quite flexible and to make the system compliant. Intermediate sinkers or buoys can be added to modify the stiffness of the mooring lines. Catenary lines can be fixed at the seabed with anchoring devices (drag anchors, piles or concrete units).

NOTE 3 Pile guided mooring method consists on vertical pile structure used to maintain a floating structure in position. In this method, the horizontal motions of the floating structure are restrained, leading to large mooring forces on the piles. An interface between floating structures and piles can require the use of appropriate fender system to contribute to the global mooring stiffness.

NOTE 4 Other specific mooring configuration can be used, based on a combination of various mooring devices. For instance, some floating structures can be moored by catenary lines at one end and connected to the shore (or to a fixed structure) with a huge ball joint or with other mechanical device at the other end. Specific mooring configurations can be governed by specific site constraints on a case by case basis.

NOTE 5 The mooring stiffness of floating structures are often non-linear i.e. the effect of the action will not follow a linear behaviour with reference to action whether in terms of floating structure motion or in terms of mooring resistance (pile or line).

(3) An Eigenmodes analysis should be systematically carried out, taking into account the stiffness of the mooring device.

(4) The following Eigenmodes should be analysed:

* hydrostatic Eigenmodes (with reference to heave, pitch and roll motions);
* mooring Eigenmodes (with reference to surge, sway, and yaw motions);
* hydrodynamic Eigenmodes (hydrodynamic resonance or sloshing between the different parts of the floating structure or with other structures in the vicinity (e.g. quay).

NOTE Identification of Eigenmodes contributes to assess the moored floating structure hydrodynamic behaviour.

(5) The response characteristics of the system shall be assessed for correct treatment and inclusion of all relevant load effects.

NOTE 1 The response itself can also be important for the loads (e.g. hydroelastic effects and coupled effects between floater and mooring/risers).

NOTE 2 A floating structure responds mainly in its six rigid modes of motions including translational modes, surge, sway, heave, and rotational modes, roll, pitch, yaw.

(6) The following three responses shall be taken into account for the analysis of the behaviour of floating structures:

* wave-frequency response (peak period and higher harmonics);
* low-frequency response;
* mean wave and current drift response.

NOTE 1 The response of a moored floating structure subject to wave actions consists in the above three contributions.

NOTE 2 The response of moored structures to waves is highly dependent on the wave direction, on the wave period and wavelength. For example, maximum response can occur when a wave period coincides with the natural period of motions of a floating structure, which also corresponds to the situation when a wavelength coincides with structure characteristic length of the floating structure.

NOTE 3 Time scales associated to the type of responses in (6) are typically:

* for wave-frequency response: a range from 5 s to 30 s;
* for low-frequency response: a range from 1 to 10 min;
* for mean wave drift response: steady loads for the duration of interest.

NOTE 4 The natural periods of the floating structures can be determined by the hydrostatic stiffness for the natural periods of vertical motions (heave, pitch and roll) while natural periods of horizontal motions can be determined by the stiffness of mooring system.

NOTE 5 The primary wave-induced forces on a moored floating structure are oscillatory and have, in general, the same frequency characteristics as the waves themselves. The motions of structures at the frequency of the waves are an important contribution to the total mooring system loads (amplification). Wave-induced forces on a floating body are usually defined as “first order” forces, which are proportional to wave amplitude. Significant wave action will cause large first order forces. These forces can be reduced by providing a soft system e.g. chain or wire catenary mooring such that the structures moves freely.

NOTE 6 Low frequency motions are induced by the low frequency component of the second order wave forces, which in general are small compared to the first order forces. Because of this, in general the low frequency forces do not play a significant role in the motions in the vertical plane (i.e. roll, pitch and heave motions) where large hydrostatic restoring forces are present. However, in the horizontal plane (i.e. surge, sway and yaw motions), where the only restoring forces are due to mooring, motions produced by the low frequency can be significant. This is particularly true at frequencies near the natural frequency of the mooring system. The motion amplitude is highly dependent on the stiffness of the mooring system. The motion amplitude is also highly dependent on the system damping. However, low frequency response for other motions such as roll, pitch and heave can also be significant for some specific floating structure types (such as deep draught floating platforms).

NOTE 7 These slowly varying drift forces which act on the moored floating structures are primarily due to non-linear interaction effects between the waves and the floating structures. These drift forces are proportional to the square of the wave amplitude and have a much lower frequency than the first order forces.

NOTE 8 The mean wave drift force is induced by the steady component of the second order wave forces.

(7) Wave-induced motions of a floating body should be considered to be dependent on mooring stiffness for floating systems with natural period less than 30 s.

(8) When several floating structures are close, their interaction in terms of hydrodynamic behaviour shall be taken into account.

(9) Determination of wave actions may be performed with use of one of the following approaches:

* analytical approach;
* numerical modelling approach;
* physical modelling approach.

(10) For the verification of ultimate limit states of floating structures, 4.8(2) shall be applied.

## Wave actions on floating structures

### General

(1) Actions from waves should include, where relevant, effects of:

* drag;
* damping;
* inertia;
* diffraction;
* radiation.

(2) Depending on wave characteristics (wave length, wave height) and on the floating structure main dimension, some contributions may be neglected for the estimate of wave loads on floating structures.

NOTE With regard to wave load estimation and wave force regimes acting on floating structures, three reference lengths are defined:

* wave height *H;*
* wavelength *L* (or noted *λ*) (in metres);
* characteristic dimension of the floater *D*.

(3) Depending on the ratio between wave height and floating structure characteristic dimension (*H*/*D*) and the ratio between floating structure dimension and wavelength (*D*/*L*), contributions of drag loads, inertia loads and diffraction loads are more or less significant, as illustrated in Figure 6.1 presented in Clause 6 and which is also relevant for floating structures.

NOTE Some very large floating structures can be flexible and exhibit significant deformations in presence of waves. Hence, hydroelastic response of such floating structures is to be considered is an important contribution. Some reference documents are listed in Annex F on that subject.

(4) Where relevant, the following actions on floating structures shall be analysed and taken into account:

* action of wave overtopping (green water);
* actions from local wave impact;
* actions due to sloshing in tanks.

### Analytical approach

(1) Analytical approach may be used for a very limited amount of applications.

NOTE Morison Formula can be used to estimate wave loads exerted on slender structure featuring simple geometry (and a geometry for which the ratio of characteristic dimension on wave length allows to neglect diffraction loads, with reference to Figure 6.1). In addition, added mass, radiation damping, wave excitation force and mean wave drift can be estimated for some simple geometries of floating structure by explicit formulas. Relevant reference documents are provided in Annex F.

### Numerical modelling approach

(1) Numerical modelling approach may be used to study different types of floating structure geometries as well as different types of mooring.

(2) Linear and non-linear wave actions shall be considered when relevant.

NOTE Some hydrodynamic load effects can be linearized and included in a frequency domain approach, while others are highly non-linear and can only be handled in time-domain.

(3) A time domain analysis should be performed when non-linear effects are significant.

NOTE A time domain analysis involves numerical integration of the equations of motion. This consists in computing the linear and non-linear motions of the floating body, induced by waves (wave frequency, low frequency 2nd order drift, and mean drift), taking into account the (non-linear) stiffness of the mooring system (which can include catenary lines, fenders, etc.) for all relevant combinations of wind, waves and current as well as other relevant actions. Time histories of all main action and response parameters (e.g. structure motions, mooring line tensions, anchor forces, etc.) are obtained from the simulation and the resulting time histories are then processed to provide statistical values (e.g. maximum displacements reached over the simulation duration or maximum tension in mooring lines).

(4) Prior to this time domain analysis, a preliminary step is necessary which consists in computing the hydrodynamics surrounding the floating body and the corresponding wave diffraction/radiation loads in frequency domain.

NOTE 1 For this purpose, a current practice is to use software based on the potential theory, which main hypothesis is to neglect viscous effects. Such approach allows to compute, from a mesh-model of the submerged part of the floating structure’s surface, the hydrodynamic characteristics of the floating structures in terms of diffraction (disturbance of the wave field incident to the fixed structure) and radiation (as generated by the wave field induced by the movement of the structure). Radiation is evaluated in the form of added masses and radiation damping. Diffraction is evaluated in the form of transfer functions of wave loads.

NOTE 2 Since the main assumption of the potential theory is to neglect viscous effects, additional models can be used to reproduce the effects due to the viscosity, especially for those degrees of freedom where radiation provides very little damping, as in roll. A good estimate of damping is also critical in computing low frequency motions (for relevant assessment of surge, sway and yaw motions). References regarding numerical mooring dynamic analysis and the associated design standards focusing on moored floating structures are presented in Annex F.

(5) Estimate of viscous damping should be considered crucial in order to provide accurate motions of the floating structure in roll direction (wave-frequency motions response to waves loading).

(6) Since the main assumption of the potential theory is to neglect viscous effects, it is, therefore, additional models should be used to reproduce the effects due to the viscosity, especially for those degrees of freedom where radiation provides very little damping, as in roll.

NOTE Viscous damping in roll are added based on coefficients found in the literature (or preferably from basin model tests with dedicated decay tests or forced oscillation tests).

(7) In order to provide accurate motions of the floating structure in roll direction (wave-frequency motions response to waves loading), estimate of viscous damping should be considered as crucial.

(8) Frequency domain analysis may be used in some cases as an alternative to the time domain analysis to estimate dynamic mooring loads.

NOTE 1 Methods for approximating non-linearities in the frequency domain and related limitations can be investigated to ensure acceptable solutions for the intended application.

NOTE 2 In the time domain method, all nonlinear effects (e.g. line stiffness, seabed interaction, etc.) can be modelled, but full time domain simulation can require a noticeable computational effort. On the other hand, the frequency domain approach is only linear.

NOTE 3 Regarding the numerical assessment of low frequency second-order wave force, a common practice is to use the Newman approximation (which enable to reduce significantly calculation time) instead of evaluating the full Quadratic transfer function (QTF) of the floating system. However, the Newman approximation can be used only in some specific conditions (for instance for floating systems featuring large natural periods and preferably in deep waters).

(9) Numerical models should be validated to establish whether the model reproduces the physical phenomena to be investigated with an acceptable level of accuracy.

### Physical modelling approach

(1) Physical modelling may be carried out to assess wave action on floating structures when assumptions in the underlying numerical modelling approaches are not met.

NOTE For further guidance, see 11.4.

## Current actions on floating structures

(1) In presence of currents, action of currents shall be taken into account.

(2) Current action should be computed with a drag force formula, as presented in Formula (11.1).

(11.1)

where

|  |  |
| --- | --- |
| *F*c | is current drag force (N); |
| **w | is density of seawater; |
| *A*P | is projected area of the submerged part of the floating structure as viewed from the direction of the current (m²); |
| *U*C | is velocity of the current (m/s); |
| *U* | is velocity of motion of the floating structure (m/s), if any; |
| *C*DC | is drag coefficient with respect to the current. |

NOTE In areas where there are currents (such as tidal currents for instance), these currents exert a force on the submerged part of the floating body. This force is referred to the current drag force. It is proportional to the square of the flow velocity (velocity of the current relative to the velocity of motion of the floating structure).

(3) The value of the drag coefficient *C*DC should be appropriately selected by taking the draft/ water depth ratio into consideration as well as the Reynolds number.

NOTE 1 For most floating structures with complex geometry or specific configurations (such as floating structures in the vicinity of fixed structures such as vertical quay or close to seabed), numerical modelling or physical modelling can be useful.

NOTE 2 In the case of numerical modelling approaches, some computational fluid dynamics (CFD) software can be used, which can be based on RANS (Reynolds Average Navier Stokes) calculations.

(4) Model test data from towing tank, hydrodynamic tunnel or wind tunnel tests may be used to estimate current loads for the design of moored floating structure systems. These tests may be performed when a representative underwater model for the unit is tested and that the contribution to current load made by appendages (e.g. bilge keels) is accounted for.

(5) Care should be taken to assure that the character of the flow (laminar or turbulent defined by a representative Reynolds number) in the model test is the same as the character of the flow for the full-scale unit.

NOTE Related references regarding force evaluation methodology can be found in associated Annex F.

## Physical modelling approach

(1) Physical modelling should be used for:

* calibration or validation of numerical modelling;
* for instance, the viscous damping in roll direction (which is defined in numerical model) needs to be calibrated from results of physical model tests;
* modelling of some specific configurations for which numerical modelling is not sufficient, for instance:
* areas featuring complex bathymetry, e.g. with steep slopes;
* areas with complex waves pattern featuring large spatial variations over the length of the floating structure;
* shallow waters areas with non-linear waves;
* shallow waters areas with low under keel clearance (of the floating structure);
* presence of one or several partially (e.g. rubble-mound breakwater) or fully (e.g. vertical quay) reflective structure close to the floating structure.

NOTE Some recommendations and guidance on how physical model tests of floating structures are to be performed are presented in Clause 12.

# Wave and current action assessment assisted by physical model testing

## General

(1) Physical model testing in hydraulic laboratories is a dedicated approach (DA4) which should be used in the design approach of coastal structures.

NOTE 1 For design assisted by testing, see EN 1990:2023, Clause 7.

NOTE 2 For uses of numerical simulations and field measurements applicable to coastal structures, see Clause 5 and Annex A.

NOTE 3 For further guidance, see Annex G.

## Purposes of testing

(1) Scaled physical modelling shall be used for coastal structures when:

* empirical equations or numerical modelling are too uncertain or out of their range of application;
* the numerical modelling is not economically feasible (e.g. CFD modelling of breakwaters).

(2) In particular, new and novel structures shall be model tested.

NOTE 1 Relevant guidelines are found in Clause 4 and Clause 6.

NOTE 2 Physical model tests are usually undertaken for the following purposes:

1. assessing stability of rubble-mound breakwaters components (e.g. armour, toe, crown wall, rear side);
2. assessing loads and pressures exerted by waves on fixed structures (e.g. vertical breakwater, gravity based structures, suspended-deck structures);
3. assessing movements and mooring forces of floating structures exposed to waves and currents
4. identifying wave transformation where complex bathymetry in front of a structure causes significant variations in sea state;
5. quantifying wave interactions with the structure such as wave overtopping, run up, reflections, wave breaking, forces and pressures on structural elements;
6. assessing wave penetration into harbours for the design of port facilities (quays, pontoons, jetties, locks) where wave disturbance patterns cannot be modelled numerically with sufficient confidence (e.g. wave transmission through complex structures).

NOTE 3 Physical modelling of sediment transport and movable bed tests are out of the scope of the present document, see Clause 1.

(3) Physical model testing is based on a simplification of reality and therefore should be used with an understanding of those simplifications and of inherent limitations of the use of the model tests results.

NOTE 1 Information on advantages and disadvantages of physical modelling are given in Annex G.

NOTE 2 Physical modelling can be used at different phases of the design process or at different stages of maritime projects depending on the objectives (e.g. design validation or optimization, maintenance programme definition).

NOTE 3 Physical modelling can be used for validation of numerical models as well as for optimization or validation of designs.

## Organization of a physical model study

(1) Physical model testing shall be conducted in a suitable hydraulic laboratory by experienced and qualified personnel in a facility with the appropriate specialized equipment.

(2) The modelling study shall be organized in three steps:

* step 1: physical model concept and layout;
* step 2: model testing;
* step 3: reporting of test results.

## Physical model concept and layout

### Input data

#### List of required information

(1) Prior to the start of the study, the following information shall be provided:

* study objectives (what phenomena will be examined) and scope of tests;
* structures to be tested (including water depths and materials composing the structure);
* seabed bathymetry;
* type of waves to be tested;
* range of hydraulic conditions to be simulated (waves, water levels and currents) at given reference points and a test programme.

(2) When verifying hydraulic design situations of armoured cover layers of coastal structures with physical model tests (DA4), the offshore (deep-sea) wave heights should be multiplied by a majoring factor *ρ*l, and offshore wave steepness retained.

NOTE 1 The values of *ρ*l are 1,20 for double layer systems, 1,35 for single layer systems, unless the National Annex gives a different choice.

NOTE 2 The displacement and the wave overtopping design situation are verified with calculations and semi-empirical formulae. Implicit safety margins for action, geometric and material parameters have been introduced in some semi-empirical formulae. With model tests, the increase of safety on the action side introduced by the present rule (majoring factor *ρ*l) is the counterpart of the drop-out of the semi-empirical formulae’s safety margins.

#### Test programme

(1) The test programme shall define each test precisely, i.e. each combination of:

* one water level,
* one wave condition,
* one current condition, where relevant,
* a duration,

and specific measurement objectives.

(2) Type of waves to be tested shall be defined, including:

* regular waves;
* irregular waves;
* multidirectional waves;
* other waves (transient waves, bichromatic waves, etc.).

(3) The targeted incident wave conditions shall be defined at a relevant reference location with information which include:

* the spectral significant wave height (*H*m0);
* the peak or the spectral period of the frequency spectrum (*T*p or *Tm*-1,0);
* the peak direction of the frequency spectrum (Dirp);
* possibly the type of wave spectrum (frequency, direction).

NOTE In flumes (vertically 2D model), the wave direction is perpendicular to the tested structure and no information on direction is needed.

(4) The reference point should generally be located at the toe of the structure or nearby at a distance of up a half wavelength in order to properly correlate incident wave conditions and response of the structure.

(5) In many cases, a foreshore is present on which waves can shoal and break and by which the wave height is reduced. In such a case, incident wave conditions at a point located further offshore, outside the surf zone, in intermediate or deep waters, should be provided in order to guide the definition of wave conditions at the wavemaker.

(6) The duration of each individual wave condition should be between 1 to 3 hours prototype scale. The number of waves should be greater than 500 to be statistically relevant.

NOTE 1 The number of waves can be greatly reduced for regular, transient and bichromatic wave tests.

NOTE 2 Shorter duration can be acceptable when the tidal exposure is short. Longer durations (6 to 12 hours) can be required to provide reliable estimates of the load/response.

(7) The water level should be fixed for one wave condition.

(8) An incremental loading procedure, up to characteristic wave conditions, shall be used in structure stability testing.

(9) For stability models of rubble mound structures, these tests should be followed by an overload test for testing reserve stability, typically 110 % to 120 % of the storm-representative wave height.

(10) For rubble-mound section stability model, at least two tests should be relative to the design condition:

* one with the low water level (testing of the toe);
* one with high water level (testing of the crest and the rear side).

(11) Repeatability tests should be performed when significant scatter of the results is expected (e.g. very low wave overtopping rates and extreme forces).

(12) A sample extreme from e.g. a 3 hour or 6 hour storm simulation shall be interpreted as just one sample among many.

NOTE 1 Sample extremes from a random realization are subject to sampling variability. This is a basic phenomenon, not a laboratory effect.

NOTE 2 For studies of extreme events and response statistics, a number of different realizations of the actual spectrum can be run.

### Contents of the modelling methodology (test plan)

(1) Prior to the start of testing, a model concept and layout and a test plan shall be agreed upon based on input data provided.

(2) An inception report shall be setup to inform on the following aspects:

* scaling laws which apply and proposed model scale;
* selected test facility;
* model layout;
* construction methodology;
* measurement equipment;
* procedure for the installation and calibration of the instrumentation;
* validation methodology of input conditions;
* model testing procedures.

### Scaling laws and model scale

(1) Because the force of gravity predominates over the other forces, Froude’s scaling law without any distortion between vertical and horizontal scales, shall be used for modelling of the action of waves and currents on coastal structures.

NOTE Coastal hydrodynamic processes are influenced by inertia/gravity, viscosity, surface tension (and sometimes even electro-chemical properties). It is not possible to reproduce wave/current processes at a reduced scale with a true similarity for all these all factors, therefore it is important to identify predominant factors.

(2) When modelling waves, inertia/gravity should predominate over surface tension (capillary forces) and viscous effects. In practice, the following parameters should be above a certain magnitude:

* water depth over 5 cm;
* wave periods over 0,4 s;
* storm-representative wave height over 1 cm.

(3) When the stability of rocks and concrete armour blocks are tested, the flow should remain turbulent within the armour layer and the associated Reynolds number (*Re)* should be larger than around 30 000, see Formula (12.1):

(12.1)

(4) Additional correction factors should apply to account for the difference between prototype and model water and rock/concrete mass densities, see Table 12.1.

Table 12.1 — Froude and stability scaling ratios (*L*r = model scale, *Δ* = (*ρ*arm - *ρ*w)/*ρ*w)

|  |  |  |
| --- | --- | --- |
| Physical quantity | Unit (SI) | Scaling ratio (prototype value / model value) |
| Dimension (height, length) | m | *L*r |
| Time | s |  |
| Water mass | kg | (*ρ*w/*ρ*mod)*L*r3 |
| Rocks or concrete armour units mass | kg | (*ρ*w/*ρ*mod)(*L*r*Δ*mod/*Δ*pro)3 |
| Rocks or concrete armour units diameter | m | *L*r(*Δ*mod/*Δ*pro) |
| Discharge | m3/s | *L*r3/2 |
| Pressure | Pa | (*ρ*w/*ρ*mod)*L*r |
| Force | N | (*ρ*w/*ρ*mod)*L*r3 |
| Moment | Nm | (*ρ*w/*ρ*mod)*L*r4 |

(5) The choice of the model scale should be guided by:

* the need to obtain satisfactory hydraulic and hydrodynamic similarity for the observed phenomena;
* the dimensions and performance of the wave flume, wave basin, wave generator and instrumentation;
* the size and geometry of the structure;
* the need to ensure that the floating structure hull form, draught, trim and mooring properties can be represented in case of a floating structure.

(6) The model scale should be as large as possible to minimise distortion of the prevailing hydrodynamic forces (scale effects) as well as to allow for accurate measurements. Potential limitations and uncertainties should be identified, and the model results should be interpreted accordingly.

NOTE For more guidance on minimizing scale effects, see 12.7.3.

### Choice of a facility

(1) The choice of a test facility shall be made to allow for a correct reproduction of the phenomenon to be modelled.

NOTE Standard facilities are wave flumes (open channel with a wave maker at one end and in some cases pumps to generate collinear currents) and wave basins (water tanks in which a wave field can be modelled in two horizontal dimensions).

(2) The chosen facility should be equipped with appropriate equipment to ensure the more appropriate representation of wave and/or current actions on the structure:

* wave and/or current generators;
* wave absorber systems (passive and/or active);
* dissipating beach;
* wave and/or current guides.

(3) Vertically two-dimensional tests in a narrow wave flume should be performed on typical cross-sections only subjected to near normal incident waves, away from peculiar three-dimensional geometry of the structure and/or seabed.

(4) The width of the wave flume should be large enough to minimize unwanted viscous effects in wave propagation along the lateral sides of the flume.

(5) Large three-dimensional physical modelling in wave basins should be built:

* whenever the structure is three-dimensional; or
* the wave action at the structure is significantly oblique (*B* > 30°, where *B* is the angle from the perpendicular), short crested or focused; or
* in situation with very irregular seaward bathymetry.

NOTE 1 Facilities are often categorized as flume (2D) and basin (3D).

* A wave flume is an open channel with a wave maker at one end, and, in some cases, pumps to generate a collinear current. Range of dimensions available in Europe are: width between 0,3 m to 5,0 m, water depth between 0,1 m to 6 m and length between 10 m to 324 m. Larger flumes could accommodate 3D model setups.
* A wave basin is a water tank in which a wave field and a coastal structure can be modelled in two horizontal dimensions. Typical dimensions of these facilities are about 20 m by 30 m and with variable water depths up to about 1 m. Wave basins are equipped with wave generators – typically fixed on one side or consisting in movable elements which can be placed for testing at different wave directions relative to the structure subject to testing and/or current generators. Fixed or portable wind generators are sometimes available in some facilities.

NOTE 2 Vertically two-dimensional models can be used to check/optimize typical cross sections at preliminary design stage(s) and three-dimensional models to verify /optimize 3D features during detailed design.

### Model layout

(1) The model layout shall present the model positioning in the proposed facility, including:

* facility boundaries and measuring area;
* bathymetry at the test area;
* position of hydraulic actions generators, guiding walls, wave absorbing systems;
* the location, geometry and composition of the tested structures (cross-sections and layout) in the facility;
* instrumentation equipment.

(2) A professional drawing plan shall support the detailed description of the layout.

NOTE A model and a prototype scale can be shown on all drawings to facilitate the change.

(3) Bathymetry contours (3D model) and slope gradients (vertically 2D model) shall be modelled to ensure that wave transformation is correctly reproduced at the points of interest of the structure.

(4) The bathymetry to consider shall be designed in accordance with the sedimentary environment of the coastal structure to take into account possible scour and active morphodynamic features.

(5) The wave generator should be located in the deepest section of the model where the water depth is superior to minimum of 3 times the spectral significant wave height and the distance to the structure superior to around 3 wave lengths at peak period.

(6) The transition slope between the wave generator and the modelled bathymetry should not be:

* too flat to avoid unrealistic wave damping;
* not too steep to cause unrealistic wave breaking.

NOTE The transition slope is usually chosen between 1:20 and 1:10.

(7) The tested part of the structure and the measuring area shall be correctly exposed to wave and current actions.

(8) Wave dampers should be installed in order to reduce reflections that propagate from the basin boundaries into the measuring area.

(9) Waves and current guides shall be placed parallel to the wave direction, so that wave reflections from the guide walls do not distort the wave field.

NOTE Waves and current guides can be used in 3D models to control energy spreading/diffraction effects.

### Construction of the model: bathymetry and tested structure

(1) The seabed shall be built as a non-erodible surface.

EXAMPLE The seabed can be made of wood or cement mortar.

(2) A movable bed bathymetry may be considered if active scour is considered as a major problem, taking into account the different uncertainties and scale effects inherent to such a morphodynamic model.

(3) The structure shall be built accordingly to the plan view and sections provided, and the selected scaling law.

(4) In case of a structure made of rock, the chosen rock grading shall be provided.

EXAMPLE For armour, toe, underlayer, etc.

(5) Finest materials should be washed to avoid dust and fines suspended in the water.

(6) Floating structures shall be constructed to correctly reproduce:

* the exterior geometry of the floating structure;
* the total mass of the structure (as well as relevant position of centre of gravity);
* the mass moment of inertia about the different axes.

(7) In addition to (6), the following model characteristics shall be verified:

* geometry, mass, mass distribution, metacentric heights, waterline;
* mooring stiffness;
* natural periods.

(8) Depending on the level of accuracy required, mooring lines as well as fender stiffness properties may be simplified and considered as featuring a linear load-deflection behaviour.

NOTE Actions of current and wind combined with action of waves can be applied on coastal structures (mainly floating structures) although it is sometimes not simple to implement. References of relevant documents regarding this aspect are provided in Annex G.

(9) A quality check of the construction including record drawings and reporting of precise measurement of key dimensions shall be realised.

(10) Photos and videos should illustrate the construction works.

### Measurement equipment

#### General

(1) The employed instrumentation shall:

* provide adequate resolution;
* be adapted to laboratory environment;
* be stable under varying temperatures.

(2) The employed instrumentation shall be calibrated following the normative GUM (Guide to the expression of Uncertainty in Measurement) and related documents published by the Joint Committee for Guides in Metrology (see Annex G for references).

(3) Photographic and/or video equipment shall be employed for every aspect of testing.

#### Equipment for wave measurement

(1) Waves shall be measured using appropriate wave probes (single or in network) deployed at different locations.

(2) The number and location of the different wave probes shall be carefully defined in order to correctly assess the wave parameters of interest), notably at the reference point where design conditions are known.

NOTE The wave parameters of interest are normally incident, reflected and transmitted wave heights, periods and directions.

#### Equipment for current measurement

(1) Currents shall be measured with appropriate current meters located at points of interest.

NOTE Current can be measured locally or globally (mean current) over the vertical.

#### Equipment for pressure and load measurement

(1) Load measurements on fixed and floating structures include pressure sensors and load cells which shall be chosen following the appropriate range of pressures and loads to be estimated beforehand and shall exhibit high natural frequency compared to load frequency.

(2) Particular care should be taken with pressure and force sensors which are generally fragile and often restricted in their applicability (pressure range, temperature, Eigen- frequency range).

(3) When direct response measurements is required, one should ensure of a proper scale of structure properties (elasticity, natural frequencies and modes).

NOTE 1 Pressures can be measured using pressure sensors installed within the structure.

NOTE 2 Load measurements (forces and moments) can be conducted by strain gauges or by averaging pressure sensor readings over the given area.

NOTE 3 For load measurements it is often necessary to use suspended/independently anchored sensors or sensor arrays (force frames) to produce reliable estimates. Load sensors are usually only able to resolve global forces.

#### Equipment for wave overtopping measurement

(1) Wave overtopping measurements should include mean discharge and instantaneous volumes.

(2) When measuring wave overtopping of the structures using a collecting tray, this tray should be fixed and sufficiently large to collect wave overtopping waves during the entire duration of the test.

#### Equipment to assess the stability of rubble-mound structures

(1) No specific equipment is requested for the assessment of the stability of rubble-mound structures.

NOTE The use of profilers or photogrammetry technologies can be interesting for an enhanced accuracy or for specific purpose (berm breakwater, scour model etc.).

#### Measuring equipment relevant to floating structures models

(1) Motions of floating structure in the six degrees of freedom shall be measured by a system that does not impose forces on the model that could significantly affect its response, such as non-contact optical motion tracking and capture system.

(2) Loads in mooring equipment (mooring lines as well as fenders) should be measured with strain gauges by load cells attached to the mooring line.

### Installation and calibration of the instrumentation

(1) Water levels shall be measured before each test and it shall be checked if the measured water level is in compliance with the target level for the test.

(2) Wave probes shall be calibrated prior each testing day and the zero-level shall be adjusted before each test.

(3) The collector tray (for wave overtopping rate measurement) shall be empty prior to each test or the level within shall be measured and reported before each test.

### Validation of input conditions

(1) The calibration of the waves shall be carried out at the reference point(s) where design conditions have been provided and repeated till comparison with the target values is satisfactory (for a precision of less than 3 % in terms of incident wave heights).

(2) In presence of low-reflective structures or inside the surf zone, the methodology should consist in calibrating all the input wave/water levels conditions before implementing the coastal structures to be tested.

NOTE This methodology results in a more accurate reproduction of the target waves, minimizing any impact of reflection.

(3) In presence of highly-reflective structures and outside the surf zone, the calibration of the waves may be carried out during specific calibration tests with the structure being built in the facility in order to take into account residual re-reflected waves not absorbed by passive or active wave absorption systems.

NOTE During these short duration tests, the methodology for measuring and validating input wave conditions can be as follows:

* measurement of the total (incident and reflected) wave spectra at the reference gauges;
* determination of the total significant wave height *H*m0, as well as the peak period *T*p;
* assessment of the incident significant wave height *H*m0 at the reference point and comparison with the given input target conditions;
* if needed, the previous steps are repeated till comparison with the target values is satisfactory.

## Model testing

### General

(1) The model testing procedure shall include the relevant items from the following list:

* wave and current generation procedure;
* data acquisition and processing;
* analysis of hydraulic measurements;
* validation of input conditions;
* assessment of stability;
* wave overtopping measurements;
* analysis of force and pressure measurements;
* analysis of motions (floating structures), and forces on mooring equipment.

### Wave and current generation procedure

(1) Wave generator shall be employed to generate regular/monochromatic and irregular/random waves. The wave generator shall be controlled by an integrated system for wave generation.

(2) In wave flumes, the wave generator shall be equipped with an active reflection compensation system when a highly reflective structure is tested.

NOTE 1 This means that the motion of the wave board compensates for the reflected waves, preventing them to re-reflect towards the breakwater model.

NOTE 2 Wave energy spectra can be prescribed by using standard or non-standard spectral shapes or by a specific time-series of wave trains.

NOTE 3 When the objective is to study the response of a few groups of high waves that create the highest loads on a structure, it can be more efficient to apply sampling schemes to isolate the most important wave sequences for structural response, so that shorter time series can be run. Wave focusing techniques can also be applied.

NOTE 4 Current generation can be performed in a variety of laboratory facilities having their own limitations in the ability to reproduce environmental conditions such as onset flow non-homogeneity or turbulence levels. Choice of a facility will depend upon many factors, including proximity of the device to the free surface and/or the seabed. Where floating devices are being tested, it is naturally beneficial to use a facility with a free surface.

(3) Depending on the objectives of the tests, proper inflow representation and the characterization of the inflow conditions should be achieved.

NOTE The characterization of the inflow conditions can be flow speed, direction, uniformity, steadiness and turbulence characteristics–small and large scale turbulent structures and combined wave-current effect.

### Data acquisition and processing

(1) Data acquisition and processing should be performed with a specialised wave analysis software.

(2) The data acquisition rates should be adapted depending on the studied phenomena:

* for waves recording: rates 25 - 2 000 Hz;
* for pressure and load cell recordings: 1 000 – 20 000 Hz (with a reduced number of channels).

(3) For pressure and load cell recordings, the sampling frequency shall be chosen far from the eigen frequency of the sensors.

(4) The wave measurements shall be carried out simultaneously with the different gauges for the total duration of the tests.

NOTE The measured time series of free surface displacement will be stored and processed using a specialised wave analysis software.

(5) Signals should be filtering during acquisition using low, high or band pass analogue filters to filter out noise or to narrow the frequency range used in the analysis.

(6) Care should be taken of phase shifts differences induced by different type and characteristics of filters when comparing several responses.

(7) Digital filters should be employed in the processing of the data.

(8) When using digital filters, care should be taken to prevent aliasing, taking into account the eigen frequency response of both structure and measuring equipment.

NOTE Depending on the analysis requirements, filtering of data after acquisition can facilitate data interpretation. For example, short waves can thus be separated from long ones or turbulent fluctuations of the current can be filtered out.

### Analysis of hydraulic measurements

(1) Data analysis should be performed with a specialised wave and/or current analysis software.

(2) The removal of spurious data is an important prerequisite for an accurate data analysis/interpretation and shall be realised (in particular in the case of load and pressure measurements), including the removal from the data of the following:

* ‘spikes’ due to instrument problems or data acquisition methods;
* offsets due to the instrument or analogue/digital conversion;
* slowly varying trends due to instrument drift and changes in water level.

NOTE Both statistical (time domain) and spectral (frequency domain) analysis can be carried out to derive the main wave parameters of interest, in particular wave height and period parameters.

### Analysis of wave overtopping

(1) Wave overtopping shall be analysed qualitatively through visual observations and potential overtopping-induced damage to the rear side of the structures shall be reported.

(2) Wave overtopping discharge should be analysed quantitatively by dividing the total wave overtopping volume by the test duration.

(3) Instantaneous volumes should be measured for the largest waves during overtopping tests.

### Assessment of stability of rubble mound structures

(1) Observation shall be the first method used for assessing damage in rubble-mound structure models through direct observation of the model during and after the tests.

(2) All observations of interest related to the stability of the structures under wave action shall be reported.

EXAMPLE Observations can be movements in the profile, extraction out of the layer, reshaping of the slopes, overtopping wave impacts and wave propagation effects influencing the stability of the structures.

NOTE Damages occur to rubble-mound structures when individual armour units on the structure are dislodged or settle. This can lead to further unravelling of the structure or loss of underlayer material.

(3) Strips of colours should be used (the rock layers will be painted) so as to allow an easier observation of the rock behaviour under wave action.

(4) No repairs should be done during one test series, in particular when evaluating resilience.

NOTE This methodology allows being closer to nature, acknowledging cumulative damages.

(5) Damage evolution should be documented by photographs taken before and after each test.

NOTE The photographs can be used in support to the observations and can help clarifying the type and level of damage undergone by the model structures under examination.

(6) A video footage should be realised during each test to show the behaviour of the structures during the tests.

(7) Damage in rubble-mound structure models should be quantified by following a commonly used method either by:

* counting the number of units that have been dislodged; or
* determining the volumetric change in areas where armour units have been displaced.

(8) Quantifying damage by volumetric change requires that pre-test and post-test profiles of the armour slope are measured in a consistent manner for comparison.

(9) In that case, the test section should be surveyed over a set grid with sufficient resolution to determine profile change with reasonable accuracy.

### Analysis of pressure and load measurements

(1) Pressure and load measurement analysis shall distinguish quasi-static from impulsive actions. Possible air effects (air entrapment) during impact events shall be identified.

(2) When air effects are identified during impact events, impulsive measurements should be corrected of the distortion of the Cauchy similitude law when extrapolated to prototype scale (see 12.7.3).

### Assessment of floating structures motions and of forces on mooring equipment

(1) For a coherent analysis of results, time series of motions and time series of tensions in mooring lines and forces in fenders shall be synchronized when they are recorded.

(2) Special attention should be paid to low-frequency response of a floating structure subjected to short-wave action.

## Reporting of test results

(1) Test results shall be reported into a physical modelling report including:

* project description and purpose(s) of the modelling;
* project methodology;
* description of the test facility, model set-up, method of construction, and instrumentation;
* testing programme;
* description of test results;
* conclusions: achievement of the objectives of the tests.

(2) Test results should be presented in dimensional form and prototype values in design studies.

NOTE Dimensionless analysis of the most important parameters as basis for interpretation of results and their presentation can give valuable insights into the model behaviour. This is especially useful if compared with other relevant tests or design guidelines or if data is exchanged between varying partners/facilities.

(3) The additional safety margin on metocean events for structures (or elements thereof) undergoing physical model tests in DA4 should be explicitly stated, depending on the implemented test range.

NOTE An additional safety margin close to unity is commonly applied when many long time series of random environmental parameters are used in the tests.

## Miscellaneous

### Inherent model uncertainty and model setup effects

NOTE 1 It is not possible to reproduce precisely and measure accurately wave/current processes and their effects on maritime structures at a reduced scale with a true similarity because of unrealistic effects due to the model setup and model scaling, and because of instrument accuracy.

NOTE 2 Model setup effects are introduced by the model set-up in the laboratory, such as:

* the absence or limited effectiveness of wave dampers or wave absorber causing unrealistic wave reflection in the study zone;
* the absence or misplacement wave guides leading to unrealistic wave dispersion or wave focusing;
* the unrealistic change of roughness at walls or seabed leading to fictive instabilities.

NOTE 3 For guidance on model uncertainty and statistical uncertainty, see EN 1990:2023, Annex D.

### Minimizing model scale effects

NOTE Scale effects occur when scaling laws do not correctly reproduce the site conditions at model scale. In a Froude model, scale effects occur when viscosity, surface tension, bottom friction, air compressibility (wave impulsive forces) are not negligible factors compared to inertia and gravity in the studied phenomenon.

(1) Common situations with distortion of the Reynolds number (turbulence) that require particular attention are:

1. Internal flows within rock mounds can be underestimated by Froude scaling as viscosity can become a preponderant parameter (in case of small rocks/ units in the core). This can lead to an underestimation of underside pressure loads or stability of solid structures (i.e. crown wall).
2. Drag forces on solid bodies (bridge piers, etc.) are depended on Reynolds numbers and roughness of the cylinder surface. Thus, the drag force coefficients can be influenced by scale effects.

Froude scaling tend to overestimate impulsive wave loads including air entrapment, leakage and entrainment.

NOTE Elasticity of the structure could play a significant role in its response to wave and current actions.

(2) Scaling corrections of the core grading should apply when internal flows within rock mounds are underestimated, see (1) a).

(3) A correction of the measured pressures and efforts should be applied when provided results at prototype scale for the case in (1) c).

NOTE Further guidance is given in Annex G.

(4) For floating structures, it should be ensured that the forces of viscosity are sufficiently representative in model tests.

NOTE Some guidance on Reynolds number values (with relevant references) are provided in Annex G.

### Instrument accuracy

(1) Measuring instruments should be calibrated in the range of values expected during the experiment.

NOTE 1 The manufacturer specification can indicate the instrument accuracy (usually less than 1 %).

NOTE 2 Variations in repeat calibration coefficient can give a measure of the instrument errors.

# Wave and current actions in reliability analysis

## Introduction

(1) Reliability analysis of structures should estimate the probability that a structure meets predefined safety and performance levels, with due consideration of all uncertainties related to actions, resistance and design tools.

NOTE 1 Reliability analysis is considered essential in pursuing full probabilistic or risk-informed design of coastal structures that requires evaluation of the structure response under a range of environmental actions, i.e. in DA2 and DA3 approaches.

NOTE 2 Reliability-based design (DA2) is recommended in Table 4.3 to be associated to any one of the hydrodynamic levels HEA2 and HEA3, presented in Table 4.2 (NDP) and Clause 5.

NOTE 3 Further guidance on application of reliability-based methods is given in Annex H.

## Probability models for wave and current actions on coastal structures

(1) The failure modes dealt with in this document yields a reliability function (of time) involving, under the present DA2, a fully probabilistic methodology. To apply this methodology a reliable limit state function should be employed.

NOTE The failure modes applicable to each type of coastal structures dealt with in this document are described in Clauses 6 to 11 and illustrated in Annex B to Annex F.

(2) The core variables of the limit state function should be examined with relevance to their dependence on time.

NOTE 1 A good approximation is to associate time variables with environmental loads.

NOTE 2 The environmental loads can be defined by their probability distributions commonly expressed through the joint probability density function (pdf) of significant wave height *H*s, and a selection among:

* mean wave period *T*m;
* directional spreading *Sp*;
* mean wave direction *θ*m;
* current velocity *U*C;
* the part of sea level *SL* correlated to the wave field.

NOTE 3 *C* denotes a representative current velocity, excluding tidal currents but including storm surge effects.

NOTE 4 *SL* refers mainly to the sea level change due to storm surge that can usually develop a correlation with the wave field through a common driving wind field.

NOTE 5 Typically *SL* includes wave set-up and storm-surge driving parameters. The latter are mainly the atmospheric pressure gradient and the wind set-up.

(3) The sea-level variation Δ*SL* due to the astronomical tide variable, although of a deterministic nature, should combine in a random fashion with waves under a uniformly distributed in time probability since the phase relationship between the two processes cannot be determined.

NOTE 1 Sea-level variation (Δ*SL*) due to the astronomical tide is the most common unrelated to the (time-variant) wave field environmental parameter present around coasts.

NOTE 2 The sea-level variation Δ*SL* has a time-variation with a cycle about 19 years regarding engineering application. This time period is, in the vast majority of coastal structures, well below the design service life of the structure. Hence this variable does not modify the failure probability in both stationary and nonstationary approaches, the latter including climatic modifications.

(4) The effect of the atmospheric pressure in a storm surge upon the sea level is loosely correlated to the wind wave field. Such correlation may be ignored in engineering applications.

(5) Currents display site-specific characteristics, and long-term measured data, or equivalent techniques, should be required to determine the pdf(*U*C).

NOTE Further guidance on probability of failure estimation can be found in EN 1990:2023, C3.3.2, and in H.3.2. An easy-to-use method is also presented in H.3.4.

## Extrapolation of exceedance probability

(1) Under the assumption of statistical independent annual extremes, the annual probability of exceedance associated to the basic environmental time-variant variables may be approximately extrapolated to larger time periods, say the design service life of the structure *L* year, through Formula (13.1):

(13.1)

NOTE 1 Similar formula hold for other extrapolations in time.

NOTE 2 A useful formula, based on Formula (13.1), involving the return period *T* = *N* year of the time-variant action is Formula (13.2).

(13.2)

(2) Formula (13.1) can be usedfor the estimation of failure occurrences with the additional condition that these are, like loads, time-variant variables.

NOTE 1 It is assumed that failure events from one year to another are independent.

NOTE 2 Time-invariant uncertainties include resistances, load coefficients, model uncertainties and statistical, parameter uncertainties.

NOTE 3 Formula (13.1) does not incorporate variability of actions over long-term time scales pertinent to climate change.

(3) When DA1 format is applied eventual modifications of environmental actions due to climate change should be accounted for externally by following recommendations from credible sources.

## Target reliability

(1) Reliability requirements applicable to design approach DA2 should be as prescribed by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties based on the guidelines given herein.

(2) When a DA3 (Clause 4, Table 4.3) is being followed then target reliability shall be determined by a risk assessment exercise, in which failure consequences should be addressed along with all costs associated to building and operating the coastal structure under consideration.

(3) Reliability requirements shall apply for all relevant failure events including partial structure failure.

NOTE Reliability requirements do not account for human errors.

(4) The target values of safety levels given in EN 1990:2023, Annex C, pertaining to structural integrity should correspond to the consequence classes CC1-CC2-CC3.

NOTE The reliability analysis presented in this Clause 13 can be applied to both procedures, i.e. selected extreme and unconditional failure probability based on all sea states.

(5) In cases where the minimum level design tool (see Table 4.3) is DA1 (partial factors) eventual use of DA2 approach shall warrant the safety level attained by DA1.

(6) Coastal structures designed in the framework of DA2 should be verified for performance at least at two limit states: serviceability limit state and ultimate limit state (see 4.2.3 and 4.7.2).

NOTE 1 An outline example of reliability analysis is given in H.6.

NOTE 2 Examples where the SLS can be applied include the calculation of a port breakwater crest based on the disruption of port basins due to wave overtopping, similarly in an embankment protecting a coastal road, the direct use of a floating pontoon by humans, etc.

## Resilience

(1) For a full resilience analysis the functional curve between reliability level and probability of failure occurrence should be established under the conditions developed through the generation and matureness of a severe storm.

NOTE 1 Introduction to resilience is given in 7.2.3(8) for mound breakwaters.

NOTE 2 A first step toward implementing a resilience analysis is to establish the critical environmental actions within severe storms, mainly in terms of wave height *H*, period *T* and angle *θ*.

NOTE 3 In the short-term hydrodynamic framework *θ* denotes the angle from the principal wave direction of the wave energy components in a short-crested sea.

(2) Until the safety level of a marine coastal structure subjected to increasing environmental actions leading to a final state above the serviceability limit state, without repairs in the meantime, cannot be credibly calculated by widely accepted formulae or numerical tools, physical model experiments should be undertaken for a calibration of such level of a coastal structure under wave fields of increasing intensity, see Clause 12.

NOTE Ways to gain a qualitative view of the resilience of some coastal structures are given in 7.2.3(5).

(3) Target resilience values should be as prescribed for CC3 and CC4 structures by the relevant authority or, where not specified, as agreed for a specific project by the relevant parties.

1. (informative)  
     
   Additional guidance on environmental sea conditions
   1. Use of this annex

(1) This informative annex provides supplementary and additional guidance to Clause 5 for hydrodynamic conditions.

NOTE National choice on the application of this informative annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

* 1. Scope and field of application

(1) This informative annex includes informative guidance supplementing Clause 5.

* 1. Water levels
     1. Tide levels

(1) The range of tide levels should be chosen as one of the following:

* mean high water springs (MHWS) or mean low water springs (MLWS);
* highest astronomical tide level (HAT) or lowest astronomical tide (LAT);

NOTE 1 During these conditions, all harmonic components causing the tides, are in phase.

* mean monthly highest or lowest tide level.

NOTE 2 The water level at a given location can vary beyond the range of astronomical tide levels due to occurrence of storm surges, tsunamis, and others. Both storm surges and tsunamis can raise the water surface to very high elevations, but they can also cause the water surface to drop below the lowest astronomical tide.

* + 1. Design water levels

(1) Depending on the available data, design practices in use, local situationsetc. one of the following methods of selecting the design water levels may be used.

1. Use of the highest recorded water level: This method is sometimes used where tsunamis can be expected with addition of a certain allowance, because the statistical analysis of tsunami amplitudes is difficult owing to rare occurrences of tsunamis.
2. Statistical extreme value analysis of storm surge levels: This can be done for both the absolute level of the highest water level above the datum level and the residual water level above the astronomical tide at the times of storm surges. The extreme value analysis can yield the water level corresponding to a designated return period. When the *T*-year storm surge is evaluated, it is added to the near highest high-water level or some other high-water levels. The designation or return period is the matter decided by the owner of the facilities.
3. Numerical simulation of the worst storm surge or tsunami in the past around the locality: The storm surge or tsunami that inflicted worst damage to the locality is selected, and the temporal and spatial variations of water level along the coastline are computed for design consideration. Sometimes, the strongest storm is chosen as a design storm and its track is shifted so as to yield the worst storm surge at the design site.
4. Probabilistic analysis: All the basic variables related to design of structures including the water level are given the statistical variability and various combinations of variables are analysed with their occurrence probabilities. Monte Carlo simulation techniques are often employed and economical and/or risk optimisation can be made.
   1. Waves
      1. Short-term wave condition
         1. Marginal short-term distribution of individual wave heights

(1) The individual wave heights shall be modelled by a short-term probability distribution relevant for the water depth at site.

NOTE 1 The Weibull distribution can be used as a general wave height distribution for sea states in deep and intermediate water depth, as where the cumulative distribution is expressed in Formula (A.1)~~:~~

(A.1)

NOTE 2 The widely used classical Rayleigh distribution theoretically valid for a narrow-banded wave process, is a special case of Weibull distribution with shape parameter *β*s = 2 and scale parameter . For shallow water the Battjes–Groenendijk distribution (Battjes & Groenendijk, 2000) can be used with a higher shape factor *β*s or wave heights above a threshold value for wave breaking.

(2) A modified Rayleigh distribution may be applied for broad-banded wave processes by modifying the scale parameter *α***.**

(3) When applied to a sea state characterized by a Jonswap wave spectrum (A.2.2.2), the scale parameter given by Formula (A.2) should be used:

(A.2)

where

|  |  |
| --- | --- |
| *γ* | is the peak enhancement factor (1 ≤ *γ* ≤ 10). |

(4) With an assumption of Rayleigh distributed wave heights, the significant wave height should be related to the zero-th spectral moment as given in Formula (A.3) and Formula (A.4):

(A.3)

(A.4)

where

|  |  |
| --- | --- |
| *ση* | is the standard deviation of the surface elevation; |
| *m*0 | is the zero-th moment of frequency wave spectrum; |
| *S*(*f*) | is the frequency spectral density function of the wave elevation *η*. |

NOTE The notation is often used for the representative wave height defined in terms of the zero-th spectral moment. For the Rayleigh distribution, the significant wave height *H*s is approximately equal to .

* + - 1. Highest wave height in relation to significant wave height

(1) The mean value of the ratio between maximum and significant wave heights in a group of waves depends on the number of waves within the duration of a given sea state and may be expressed as in Formula (A.5):

(A.5)

where

|  |  |
| --- | --- |
| *N* | denotes the number of waves. The value given by the first term of the right-hand side of Formula (A.5) can be used as a good approximation for high enough *N*. |

NOTE 1 A specific value of the highest wave height *H*max in a group of waves is subject to random variation governed by its own probability of appearance.

NOTE 2 Extremely large waves have been observed and reported with heights beyond what might be expected from the statistical distribution of *H*max/ *H*s. These waves are called freak waves.

NOTE 3 It is a question whether the probability of the freak wave appearance remains within a range predicted by the classical wave height distributions of wave heights or if they are caused by some effects not accounted for in presently available theories of wave statistics.

* + - 1. Short term distribution of individual wave periods

(1) The distribution of individual wave periods shall be examined in the light of the short-term joint distribution between wave heights and periods, because the representative wave periods such as the significant wave period are defined in association with the representative wave heights ranked by their magnitudes.

NOTE Theoretical models are available for the short-term joint distribution of wave heights and periods. Waves of sharply-peaked spectra exhibit a narrow spread of period distribution around the spectral peak period, while a combined sea state of wind waves and swell shows multiple peaks in the distribution of wave periods.

(2) For wind waves with single-peak spectra, the empirical relationship in Formula (A.5) holds for the wave periods of individual waves defined by the zero-upcrossing or zero-downcrossing method, and the significant wave period in Formula (A.6) may be used:

(A.6)

where

|  |  |
| --- | --- |
| *T*s | is significant wave period; |
| *T*z | is the zero up-crossing wave period which is the mean value of the wave periods of individual waves. |

(3) The relationship between the significant wave period and the spectral peak period is affected by the spectral shape. For fully-grown wind waves, the relationship of *T*s ≌ 0,9*T*p may be used.

NOTE Further information can be found in Goda (2000), Table 2.4.

(4) The zero up-crossing period may be estimated from the zero-th and second moments of the frequency wave spectrum according to Formula (A.7):

(A.7)

where the spectral moments *m*0 and *m*2 are calculated by Formula (A.4).

NOTE Formula (A.7) is based on the condition that the wave spectrum is comprised by an infinite number of free linear frequency components. Because actual waves contain a certain amount of non-linear spectral components in the high frequency ranges, *T*m0,2 estimated by Formula (A.7) becomes shorter than the zero up-crossing period *T*z calculated from the time history of wave elevation; the difference is enhanced in shallow water and can reach up to 20 %.

* + - 1. Directional wave spectra

(1) A detailed structure of wind waves and swell is represented with the directional wave spectrum, which may be expressed as in Formula (A.8):

(A.8)

where

|  |  |
| --- | --- |
| *f* | is the frequency; |
| *θ* | is the azimuth measured from some fixed axis of direction; |
| *S(f)* | is the frequency wave spectrum or the frequency spectral density function; |
| *D*(*θ*∣*f*) | is the directional spreading function. |

NOTE 1 The frequency wave spectrum, *S*(*f*), expresses the distribution of wave energy density (per unit weight of water) in the frequency domain and has the dimension of m2s or equivalent units.

NOTE 2 The directional spreading function, *D*(*θ*∣*f*), expresses the distribution of wave energy density in the directional domain at a specific frequency relative to the spectral density at that frequency. Thus, the directional spreading function has no dimensions and its integration over the full range of azimuth is set at unity for every frequency.

(2) For the sea state of coexisting wind waves and swell, the respective spectral densities may be linearly superposed so as to yield the directional wave spectrum of the combined sea state.

(3) Use of wave spectra in very shallow water should be made with due caution for the effect of non-linear components on wave actions.

NOTE Spectral analysis of wind waves and swell is based on the concept of linear superposition of component waves. When the representative wave height becomes large compared with wavelength and/or water depth, non-linear interactions between component waves are enhanced and a wave spectrum begins to include an appreciable amount of non-linear spectral components.

* + - 1. Frequency spectra of wind waves and swell

(1) For evaluation of the actions from waves, a frequency spectrum may be expressed as the function of representative wave height and period.

NOTE Some frequency spectra are expressed as the function of wind speed. They include the Pierson-Moskowitz, the JONSWAP, and the TMA spectra.

(2) For fully-grown wind waves, the Bretschneider-Mitsuyasu frequency spectrum of the following may be employed, see Formula (A.9):

(A.9)

NOTE The functional dependence of the Bretschneider-Mitsuyasu spectrum with respect to frequency is the same as that of the Pierson-Moskowitz spectrum. Its constants have been set to satisfy the condition of and the relationship of *T*p = 1,1 *T*s, which was derived by Mitsuyasu based on his field measurements of wind waves with presence of some low frequency components.

(3) For versatile functional shapes of single-peak wave spectra, the modified JONSWAP of the following may be employed, see Formula (A.10) and (A.11):

(A.10)

where

|  |  |
| --- | --- |
| *fp* | denotes the frequency at the spectral peak, *fp* =1/*T*p; |
| *γ* | is peak enhancement factor; |
| *B*(*γ*) | is normalizing factor, *B*(*γ*)= 0,0625/(0,065*γ* 0,803+0,135); |
| *σ*s | is spectral width parameter: |

(A.11)

(4) The significant wave period is related to spectral peak period as given in Formula (A.12):

(A.12)

* + - 1. Directional spreading functions of wind waves

(1) Two functional forms are introduced in (2) and (3) for directional spreading of wind waves, but other functional forms may also be used in the analysis.

NOTE Mutual comparison between the Mitsuyasu type directional spreading function (dictates the frequency dependency of directional spreading) and the wrapped-normal directional spreading function (assumes directional spreading independent of frequency) is possible by means of the angular standard deviation *σ*θ. When the overall directional energy spreading of the Mitsuyasu type directional spreading function is calculated by the integration of the directional spectrum with respect to the frequency, the spectrum with *s*max = 10 approximately yields *σ*θ = 33°, while *s*max = 75 yields *σ*θ = 14°. For more details of mutual comparison of several directional spreading functions, see Goda (1999).

(2) The Mitsuyasu type directional spreading function in Formula (A.13) shall be based on field data:

(A.13)

where

|  |  |
| --- | --- |
| *D*0 | is the normalizing constant given in Formula (A.14): |

(A.14)

where

|  |  |
| --- | --- |
| *θ*m | is the mean wave direction measured from a given axis of direction; |
| *s* | Is the spreading parameter that varies with the frequency as in Formula (A.15): |

(A.15)

NOTE According to Mitsuyasu et al. (1975), the peak value of spreading parameter, *s*max, varies depending on the state of wind wave growth. A representative value for fully-grown wind waves is about *s*max = 10. With an increase in the *s*max value, the extent of directional spreading becomes narrow. Goda (2000, p. 34) and OCDI (2002, p. 39) suggest the value of *s*max = 25 for swell with short decay distance and that of *s*max = 75 for swell with large decay distance for the cases of deep-water waves.

(3) The wrapped-normal type directional spreading function in Formula (A.16) employed in some multi-directional model tests may be used:

(A.16)

where

|  |  |
| --- | --- |
| *σ*θ | is the angular standard deviation defined by Formula (A.17): |

(A.17)

(4) When using the wrapped-normal type directional spreading function in Formula (A.16), the number of serial terms *N* should be sufficiently large to ensure convergence of the finite series.

(5) Owing to wave refractions effects, directional spreading of wind waves and swell becomes narrow in shallow water compared with that in deep water. In evaluation of the actions from waves, such a change of the spreading parameter should be appropriately estimated and a corresponding value in shallow water should be employed.

* + - 1. Representative height and period of combined sea state

(1) When a sea state is composed of wind waves and swell and the information of the heights and periods as well as the principal directions of these wind waves and swell is known, the respective directional spectra of these wind waves and swell may be estimated in accordance with A.4.2.3 as follows.

* The directional wave spectrum of the combined sea state is obtained by linearly superposing the directional spectral densities of the wind waves and swell.
* The actions from these waves are then analyzed by calculating contributions of components of the directional wave spectrum thus obtained.

NOTE The representative period of the combined sea state is difficult to define, because the joint distribution of wave heights and periods exhibits multiple modes that correspond to the modes of periods of respective wave groups. However, there is a formula for estimation of the significant wave period of the sea state composed of two wave groups as listed in OCDI (2002, p.70). It was proposed by Tanimoto et al. for the purpose of evaluating wave loading on vertical face breakwaters.

(2) The representative height of the combined sea state may be estimated as in Formula (A.18):

(A.18)

where

|  |  |
| --- | --- |
| *H*s | denotes the representative wave height of combined sea state; |
| *Hn* | is the representative wave heights of wave group *n*. |

NOTE Any definition of representative wave height such as the significant wave height or the highest wave height is applicable to Formula (A.18) because the distribution of wave heights of the combined sea state is approximated by the Rayleigh distribution as given in A.4.1.

* + 1. Wave climate (long-term) statistics
       1. Statistical representation of wave climate

(1) Wave climate at a site may be described in many ways:

* time-history diagrams of height, period and direction over a month, a season, or one year visualize a general trend of wave climate at a site.;
* the means and standard deviations of height and period over months, seasons, and years provide basic statistics of wave climate;
* marginal and joint distributions of wave height and period are utilized in the analysis of long-term effects of wave actions on structures;
* joint distributions of wave direction with wave height or period are important in assessing tranquillity of a harbour basin and littoral sand transport rate along a coastline;
* duration statistics of calm seas and rough seas are examined for analysis of the workability and operational efficiency of maritime facilities.
  + - 1. Marginal distributions of representative wave height and wave period

(1) Marginal distributions of representative wave height and periods may be:

* Weibull distribution for wave height;
* log-normal distribution for wave period.

NOTE 1 The total sample method, or initial distribution method, is a simple extrapolation of a marginal distribution of significant wave height for estimation of extreme wave height such as 100-year or more wave height.

NOTE 2 The data set for the marginal distribution of wave height or period does not constitute a sample of statistically independent data since the wave heights and periods measured over several hours are mutually correlated. The upper tails of the marginal distributions often exhibit the trends different from the main parts, because the data in the upper tails are samples from the population of storm waves being different from the population of medium to calm sea state.

* + - 1. Joint distribution of representative wave height and period

(1) Since the pattern of the joint distribution of representative wave height and period is highly dependent on the nature of wave climate at locality, caution should be taken against simple application of theoretical models such as double log-normal distributions to the joint distribution data of representative wave height and period. See A.4.1.3.

NOTE In the area in which wind waves are predominant throughout a year, a close correlation between wave height and period is observed and the scatter of data points in the joint distribution of representative wave height and period is relatively small. In the area in which swell activity is strong, the data points are scattered over a broad area and the correlation between wave height and period is weak.

(2) The marginal distribution of whole individual wave heights or actions during the design service life of a structure may be made from a joint distribution of representative wave height and period over many years convoluted with the distribution for individual wave heights:

* For each class of joint histograms of wave height and period, the number of individual waves expected in the time interval between successive measurements is calculated with the representative wave period.
* The individual waves are given respective heights according to the wave height distribution, and the numbers of waves in respective classes of the height are counted and tabulated.
* If some information is available on the joint distribution of individual wave heights and periods at the site, further refinement can be achieved.
  + 1. Extreme wave statistics
       1. Data set for extreme wave analysis

(1) The database for extreme wave analysis should be:

* a long record of wave measurement data; and/or
* results of wave hindcast data.

(2) A wave hindcasting method that has been verified to yield predictions in good agreement with instrumental wave records for several large storm waves obtained around the site of interest shall be employed.

NOTE The accuracy of wave hindcasting is affected by the reliability of both the hindcasting model itself and the meteorological information for wind field estimation.

(3) The length of data record should be 30 years or longer.

NOTE A long record is needed to reduce the effect of sample variability and to minimize the influence of wave climatic changes on the prediction of extreme wave heights for a long return period such as 100 years or more.

(4) The measured and/or hindcasted extreme waves should be classified according to the types of wave populations so that the data sets can be constructed for respective storm types, e.g. due to:

* meteorological disturbances (weather patterns);
* sea-states (steepness and other parameters); or
* directional sectors/ origin.

(5) Extreme wave data of respective storm types may constitute samples from different populations of extreme waves.

NOTE When an extreme wave analysis is made on a data set of mixed populations, the prediction of extreme wave height can be unreliable.

(6) A set of extreme wave data may be prepared by two methods:

* the annual maximum method, i.e. to take the maximum significant waves in every year;
* the peak-over-threshold (POT) method, i.e. to take the waves at a peak of every storm event that is defined with exceedance of the significant wave height above a preset threshold level.

NOTE Because the currently available databases of extreme waves in the world do not cover a sufficiently long time span, the sample size of extreme wave data by the annual maximum method is rather small and the confidence interval of extreme wave analysis becomes relatively large.

(7) The peak-over-threshold (POT) method should be the technique of data analysis.

NOTE A set of extreme wave data by the POT method does not belong to the category of extreme data in the strict sense of the statistics, because a peak height is not a maximum data among a subset of independent data such as required in the extreme statistics.

(8) The average number of storm events, or the mean rate, is an important parameter in the extreme wave analysis when the POT method is employed. The mean rate should preferably be calculated for respective wave populations/storm types.

* + - 1. Extremal distribution functions for storm wave heights

(1) The following distributions may be employed as the candidates for fitting to the data set of extreme wave heights:

* double exponential distribution;
* Gumbel distribution;
* three-parameter Weibull distribution.

NOTE No consensus has been established on the population distribution of storm wave heights. The POT wave data is not the extreme data in strict sense.

(2) Other distributions such as the Generalized Extreme Value and the log-normal distributions may also be used.

NOTE See Clause 11.

* + - 1. Data fitting and selection of extremal distribution function

(1) Appropriate criteria of best fitting and/or rejection should be chosen and applied when fitting a data set of extreme wave heights, or a sample, to a candidate distribution for parameter estimation, depending on the methodology of data fitting.

EXAMPLE Examples of fitting techniques are:

* the least squares method (LSM);
* the maximum likelihood method (MLM);
* the method of moments (MOM);
* probability weighted moments (PWM).

(2) When using the least squares method (LSM), care should be taken to employ the non-bias plotting position formulas for respective distribution functions.

NOTE When applying the least squares method (LSM), the shape parameter of the Weibull distribution is often fixed at one of predetermined values to transform it into a two-parameter distribution.

* + - 1. *T*-year wave height and confidence interval

(1) The wave height corresponding to a given return period, or *T*-year wave height, should be estimated based on the distribution assumed to represent the population of extreme wave heights at the site by a standard procedure of extreme statistics.

(2) A range of confidence interval should be estimated and indicated for every estimate of return wave height, since because of the sample variability of data set, each sample will yield different estimates of *T*-year return period wave height.

NOTE 1 A data set of extreme wave heights obtained through wave measurements and/or hindcasting represents one sample from the population of storm waves at the site. Even with absence of climatic changes, a data set covering different but equal length of time will constitute a sample of the same distribution but with different statistical characteristics. This is called the sample variability of data set.

NOTE 2 A misfit of an extreme wave data set to a distribution different from the true population of storm wave heights will yield a bias in the estimate on *T*-year return period wave height. Analysis of storm wave data sets at multiple stations in a region of same storm characteristics can yield information on the population of storm waves (see Goda et al., 2000).

* + - 1. *T*-year height of highest wave

(1) The highest wave height corresponding to a given return period may be estimated from the *T*-year return significant wave height by multiplying it with a certain factor based on the wave height distribution.

NOTE In offshore engineering, efforts are often made to estimate the *T*-year return height of highest wave from the marginal distribution of whole individual wave heights, which is constructed from the wave climate data being convoluted with the short-term wave height distribution.

* + - 1. Wave period associated with *T*-year wave height

(1) A joint distribution of storm wave heights and periods should be established to model the correlation between wave height and wave period.

NOTE The information of wave period associated with the T-year return period wave height is often needed when evaluating actions from waves.

(2) In lack of a joint distribution model, the wave period associated with the *T*-year return period individual wave height *H*max may be taken as .

* + 1. Wave kinematics
       1. Crest elevation

(1) Nonlinear theories of regular waves such as the Stokes 5th wave theory and the stream function theory may be used to calculate the crest elevation and the profile of large regular waves.

NOTE 1 The crest elevation of highest wave is one of the key factors in designing pile-supported structures such as piers and oil drilling platforms, because it determines the upper limit to which the actions from waves are exerted.

NOTE 2 The theory of nonlinear random waves has not developed yet to accurately calculate the crest elevation of highest wave among random waves.

NOTE 3 The ratio of the crest elevation above the still water level to the wave height increases from 0,5 for the infinitesimally small waves toward a limiting value at wave breaking as the wave height increases.

NOTE 4 The upper limit of the crest-to-height ratio at wave breaking is a function of the water depth relative to the wavelength.

* + - 1. Water particle velocities

(1) For estimation of wave kinematics of large waves one of the following approached should be used:

* the storm-representative wave method using nonlinear theories of regular waves, which is applied to individual waves defined by the zero-crossing method;
* the spectral computation of wave kinematics by converting the directional wave spectrum of surface elevation to that of wave kinematics by means of the transfer function from the surface elevation to the kinematics;
* the hybrid storm-representative wave method in which the prediction of wave kinematics by nonlinear wave theories is reduced by taking into account the effect of the directional spreading of wave spectra, which is estimated by the linear transformation theory.

NOTE As to non-linear wave theories the Stokes 5th order wave theory and the stream function method by Dean (1965) are often used by practioners in the oil industry. There is another numerical (user-friendly) method for non-linear wave kinematics by Rienecker and Fenton (1981), which employs Fourier expansion series of the stream function and applies Newton techniques to solve a system of non-linear simultaneous equations. The latter can also solve the case of waves in water flowing with a specified speed. The Fenton method is user-friendly and applies well to regular shallow water waves.

(2) When the storm-representative wave method using nonlinear theory of regular waves is employed, Figure 5.1 should be used to confirm that the theory can predict the horizontal velocity at the crest of breaking wave being equal to the wave celerity.

NOTE Laboratory measurements of wave kinematics by Chakrabarti and Kriebel (1997) among others have demonstrated the applicability of the storm-representative wave approach in the mid-water zone. Measurements of horizontal velocities around the wave crests have been made by Skjelbreia (1987) with laser doppler velocimetry and by Lader (2002) with particle image velocimetry. Skjelbreia reported the approximate equivalence of the maximum horizontal velocity to the celerity of breaking solitary wave, while Lader obtained the maximum velocity in the range of 0,7 to 0,8 times the celerity of transient breaking waves by extrapolation of measured velocity profiles.

(3) When using the approach with the spectral computation of wave kinematics, a simple application of the linear wave theory for the transfer function overpredicts the wave kinematics near the wave crest in relatively deep water, because the spectral components of wave kinematics at the high frequency range are excessively amplified, and some stretching of the vertical coordinate should be employed.

NOTE 1 Wheeler’s method (1970), for example, transforms the vertical coordinate z into z’ = (z – *η*)/(1 + *η*/*h*), where *η* denotes the instantaneous surface elevation. Gudmestad (1993) has presented a review of measured and predicted wave kinematics in deep and intermediate water, including a number of field measurement reports.

NOTE 2 When the hybrid storm-representative wave method is used, reduction of wave kinematics from two-dimensional wave theories up to 15 % has been observed in several field measurement projects. See for example Forristall et al. 1980.

NOTE 3 The ratio of the directional wave induced water particle velocity to the unidirectional wave induced water particle velocity is sometimes called the spreading factor. Forristall and Ewans (1998) have presented the value of spreading factor for various wave conditions in the field.

* + - 1. Wave kinematics

(1) When calculating actions from waves and currents on a slender structure, kinematics of waves and currents shall be combined.

EXAMPLE The Morison type loading, vortex induced vibrations, etc.

NOTE 1 For the simple case of the following or opposite currents in water of uniform depth, see the horizontal and vertical water particle velocities and accelerations for the stationary frame, assuming linear wave theory in Hedges (1985).

NOTE 2 Hedges (1987) give for the similar conditions the spectrum of the water article velocities and accelerations by a transfer function approach when the scalar water elevation spectrum is given by *S*ηη(*f*a,*U*), where *f*a is the frequency corresponding to the period observed by a stationary observer and *U* is the current velocity.

NOTE 3 For shortcrested seas for which there will be varying degrees of wave refractions caused by currents, depending on initial wave direction and frequency, as well as on current direction and strength, see methods referred to in Hedges (1987).

(2) Since there have been no investigations on the effect of directional waves and currents on the Morison type force, the scalar wave spectrum together with the angle between the mean direction of the waves and the current may be used when evaluating the spectrum of the water particle velocities and accelerations.

NOTE It is common practice to simply to add the current velocity, stretched to the instantaneous water level, to the water particle velocities from the waves as obtained from a proper wave theory, e.g. API RP 2A WSD (1993) and NORSOK Standard (1999). The Fenton Fourier series theory, see A.3.8.2, is also useful and more exact than the engineering approach taken by the oil industry.

* + 1. Wave transformations

(1) Wave attenuation by bottom friction shall be taken into account.

NOTE 1 Reliable evaluation of the amount of wave attenuation by bottom friction is difficult, because there is a wide scatter of the data of the friction coefficient estimated from the field measurements of wave decay, but the term of energy dissipation due to bottom friction has been incorporated into some spectral models for wind wave generation and propagation in shallow water.

NOTE 2 Where a nearly flat and shallow sea extend over a long distance or the seabed is inclined with a slope gentler than 1/300 or so, waves are gradually attenuated owing to the bottom friction; i.e., the loss of wave energy flux by the orbital motion of water particles working against the bottom turbulent shear stress.

NOTE 3 Another source of possible wave attenuation is the wave-induced motion of soft subsoil layers and associated visco-elastic energy dissipation. There are reports that an appreciable degree of wave damping takes place in coastal waters with the seabed composed of very soft clay. Several theories have been presented and laboratory tests have been made for their verification. No established methodology is available however for quantitative evaluation of wave damping by this mechanism in the field.

* 1. Currents
     1. General

(1) For bottom founded structures, the total current profile associated with the sea state producing extreme or abnormal waves should be specified.

NOTE 1 The total ocean current velocity is the vector sum of tidal and non-tidal or residual currents. The components of the residual current include circulation and storm generated currents, as well as short and long period currents generated by various phenomena, such as density gradients, wind stress and internal waves. Residual currents are often irregular, but in many locations, the largest residual current to be considered is the extreme storm surge currents.

NOTE 2 Tidal currents are regular and predictable and their daily maximum velocities are approximately proportional to the tidal range of the day. They are generally weak in deep water past the shelf break. They are generally stronger on broad continental shelves than on steep shelves. Tidal currents can be strengthened by shoreline and bottom configurations such that strong tidal currents can exist in many inlets and coastal regions.

NOTE 3 Circulation currents are relatively steady, large scale, features of the general oceanic circulation. Examples include the North Atlantic Current. While relatively steady, these circulation features can meander and intermittently break off from the main circulation feature to become large scale eddies or rings, which then can drift at a speed of some few miles per day. These circulation features occur mainly in deep water beyond the shelf break and generally do not affect coastal sites. But they can affect wave refractions from deep to shallow water.

NOTE 4 Wind generated currents are caused by the wind stress and atmospheric pressure gradients through the storm. These current velocities are a complex function of the storm strength and meteorological characteristics, bathymetry and shoreline configurations, and water density profile. In deep water along open coastlines, surface storm currents can be estimated to up to 3 % of the 1 hour sustained wind velocity during storms. As the storm approach the coastline and shallow water, the storm surge and current can increase.

NOTE 5 In the surf zone, there exist special currents called the near-shore currents induced by waves. Longshore currents run parallel to the shore as a result of waves breaking at an angle on the shore, also referred to as littoral current. Because the near-shore currents are generated within the surf zone, they transport suspended sediments in areas where sediments are present and cause topographical changes of beaches.

NOTE 6 Rip currents are strong, localized and narrow masses of surface water moving directly away from the shore, compensating for the average mass transport in steep breaking waves. An undertow current is a similar type of current moving away from the shore close to seabed.

NOTE 7 At river outlets and in estuaries the currents can be complex due to the interactions of fresh and salt water.

NOTE 8 In inlets and fjords along mountainous coastlines, strong current-like flooding due to melting of snow and ice can occur during spring time.

* + 1. Stretching of current to wave surface

(1) The effect of waves on the variation in current profile with water depth should be accounted for.

(2) In the wave zone, the current profile may be stretched or compressed vertically, with the current velocity at any proportion of the instantaneous depth constant, see Figure A.1.

NOTE 1 By this method the surface current component remains constant.

A diagram of a curve

Description automatically generated

Key

|  |  |
| --- | --- |
| 1 | linear |
| 2 | non-linear |
| 3 | input current profile |

Figure A.1 — Stretching of current profiles

NOTE 2 Stretching is expressed formally by introducing a stretched vertical coordinate *z*s such that the current speed *U* at depth *z* in the still water profile. Linear stretching is defined by Formula (A.19):

(A.19)

where

|  |  |
| --- | --- |
| *η* | is the wave surface elevation; |
| *h* | is the still water depth. |

NOTE 3 This is the essentially the same as Wheeler stretching used for wave kinematics above z = 0 for linear waves.

NOTE 4 Non-linear stretching is defined by relating *z*s and *z* by Formula (A.20):

(A.20)

where

|  |  |
| --- | --- |
| *k*nl | is the non-linear wave number corresponding to the wavelength *λ*nl of the regular wave under consideration for water depth *h* and wave height *H*, calculated using non-linear wave theory and the intrinsic wave period. |

NOTE 5 Non-linear stretching provides the largest stretching at the sea surface where the wave orbital motion is largest.

NOTE 6 If the current is not in the same direction as the wave, both in-line and normal components of the current may be stretched. For irregular waves, the stretching method applies to each individual crest-trough.

(3) In most cases linear stretching produces accurate estimates of hydrodynamic loads, but if the current profile has very high speed at the sea surface, with a strong shear to lower speeds just below the surface, the non-linear stretching should be used.

* + 1. Numerical simulation of current flows – current hindcast

(1) Current hindcast models may be used to estimate current velocity profiles in locations where current measurements are not available.

NOTE 1 Current hindcast models are based on numerical ocean flow models covering a limited ocean area, and atmospheric and ocean force terms are input to the models.

NOTE 2 Atmospheric forcing includes wind speed and direction, air temperature and atmospheric pressure provided from atmospheric hindcast models.

NOTE 3 Ocean forcing along ocean boundaries include current speed and direction, sea surface height, sea temperature, density and salinity from large-scale ocean circulation models.

NOTE 4 Current hindcast models are calibrated against current measurements at a set of locations where the measurements cover the entire water column and typically sampled every 10 minutes.

NOTE 5 The output from current hindcast models is typically current speed and direction every 1 hour, typically regarded as 10-minute mean values.

(2) Current hindcast models should be validated by measurements before they can be used with confidence.

NOTE While hindcast models can give quite reliable predictions for some areas, this is generally not the case for other locations.

* + 1. Current properties

(1) The properties of the extreme current profile may be obtained by:

* current measurement surveys;
* current hindcast numerical modelling;
* extrapolation of data sets.

NOTE The properties of current profiles in different parts of the world depend on the regional oceanographic climate, in particular the vertical temperature structure and the advection of water into or out of area. Both these controlling aspects vary from season to season. Typically, shallow water profiles in which tides are dominant can often be characterised by a logarithmic profile or a simple power laws of velocity versus depth, whereas deep-water profiles are more complex and can even show reversal of the current direction with depth.

(2) Site-specific current measurements should extend over the water profile, depending on the water depth, and over a period that captures several major storm events that generate large sea states.

NOTE Site-specific measurements of currents at the location of a structure can be used either as the basis for independent estimates of likely extremes or to check the indicative values of the various components of the total current.

(3) Current models may be used in lieu of site-specific measured data.

(4) The period over which the current model is run should be adequate to allow tidal decomposition to be carried out and the residual current to be separated out of the total current.

(5) Efforts should be made to ensure that the output of a current model is validated against nearby measured data.

(6) When extrapolation of data sets is performed account should be taken of the three-dimensional nature of the flow.

(7) The logarithmic current profile given in Formula (A.1) or the simple power law in Formula (A.2), may be used where appropriate as given in Formula (A.21):

 (A.21)

EXAMPLE It can be used in areas dominated by tidal currents in relatively shallow water as in most coastal waters.

(8) When the velocity at a certain elevation below the still water line, *z*1, has been measured, Formula (A.22) may be used:

 (A.22)

where

|  |  |
| --- | --- |
| *z* | is vertical coordinate (*z* = 0 at the still-water line); |
| *u*\* | is friction velocity; |
| *z*0 | is bed roughness length; |
| *κ* | is van Karman’s constant, *κ* = 0,4; |
| *α* | is coefficient, approximately 1/7; |
| *h* | is water depth. |

1. (informative)  
     
   Additional guidance for fixed cylindrical structures and suspended decks
   1. Use of this annex

(1) This informative annex provides supplementary and additional guidance to Clause 6 for wave and current actions on fixed cylindrical structures and suspended decks.

NOTE National choice on the application of this informative annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

* 1. Scope and field of application

(1) This informative annex includes informative guidance supplementing Clause 6:

1. additional guidance on classification of structures within the scope of this document and selection of methods to assess actions (related to Clause 4 and EN 1990:2023, Clause A.6);
2. supplementary guidance in application of methods outlined in Clause 6, where additional information in practical design is needed or can be useful;

EXAMPLE Alternative methods, more sophisticated methods, conservative methods not fully technically justified in coastal engineering practice (e.g. offshore methods) or methods that are felt still not to cover a widely accepted design approach).

1. guidance in the selection of coefficients in formulas similar as above.
2. general discussion of suitable methods that gives more accurate and less conservative approached to assess actions on fixed structures.
   1. Classification

(1) HEA («Hydrodynamic Estimate Approach») level should be defined, based on the consequence class and expected hydrodynamic uncertainties, as described in Clause 4.

(2) Actions of fixed structures with a recommended design approach should be assessed according to Clause 6.

NOTE For design approaches (DA), see Table 4.3.

(3) Fixed cylindrical structures and suspended decks subjected to wave and current actions in the coastal area should be classified as structures with «low to medium design uncertainties» according to Table 4.3 unless:

* structures containing complex geometrical cylinders, and arrays of structures that will provide reflections and high degree of disturbance of waves can fall in the design class «high uncertainty». Also, for structures where effects of ringing and springing is expected to influence the structural reliability;
* structures like vertical cylinders on exposed reefs and shoals subjected to breaking waves can fall in the design class «high design uncertainty»;
* structures where the superstructure design is classified unproven and the «Hydrodynamic uncertainties» are classified «medium» or «high» according to Table 4.6 can fall in the design class «high design uncertainty».

NOTE Justification of «proven» is associated with newness. Such require that the structural system and reliability is based on proven design solutions or if assessment of actions is based on methods proven by testing (DA4), design approach according to Clause 4).

(4) Structures where new technology is used, should be classified with «high design uncertainty.

EXAMPLE Structures limited in number and where no relevant experience exists, structures made of innovative materials/substructures/geometry, or where new execution technology is used.

* 1. Principles of design
     1. General

(1) Calculation of actions from wave and currents on fixed structures should take appropriate care of the interaction with the structure geometry.

NOTE 1 Nature of the waves is dynamic, and the variation in time and space is significant. Currents is in principle a steady flow but is also characterized by variation in time and space. Current can have large seasonal variations, it is on a larger scale influenced by water density, wind, bathymetry and disturbance by structures and land, all in addition to tidal variations. See Clause 5.

NOTE 2 Waves and currents passing any submerged or surface piercing structure induce dynamic pressures in addition to the hydrostatic pressures on the wetted surface of the structure. These pressures when integrated over the wetted surface of the structure results in a net force. Clause 6 and this Annex B aims to guide in calculation of forces on fixed structures.

NOTE 3 Design that will fall outside the scope of Clause 6 and this Annex B is design of structures with elastic flexural behavior, and natural period within the wave frequency domain. Also structures subjected to higher order wave loads that result in high-frequency resonance.

NOTE 4 Forces from wave and currents on fixed structures can be assessed with a variety of design methods, from simplified methods to more sophisticated, numerical simulations and physical model testing. The most simplified methods will have limitations in their application to calculation of actions.

(2) Maximum storm-representative wave approach may be applied.

NOTE 1 To calculate the maximum wave height, see 6.1.2 and A.4.1.2.

NOTE 2 Wave actions are characterized by a number of parameters. Actions on fixed structures in combined sea states (fetch generated wind waves and ocean waves) are not always governed by the maximum wave height approach and corresponding periods.

(3) Wave periods will, in addition to structural dynamics, also influence the spatial distribution of particle velocity, such effects should be investigated.

NOTE Screening of the sea state based on *H*s-*T*p contour plots can be appropriate.

(4) When linear wave theory is applied to calculate the wave actions, due consideration shall be given for the wave actions above SWL.

NOTE Linear wave theories imply that the free surface conditions are linearized and satisfied at the still water level. This will normally make an oversimplification that is non-conservative and not applicable in normal design.

(5) As linear wave theory is applied up to the still water level, the effects of the waves above the free surface shall be accounted separately.

(6) The linear wave theory should be modified to take into account for the free surface effect.

(7) The wave kinematics between the free surface and still water level may be obtained by applying stretching or extrapolation methods.

NOTE A comparison of different stretching methods has been presented by Stansberg et al. 2013. Further reading on stretching techniques can be found in NORSOK N-003.

(8) The Wheeler’s method should be used along with second order surface record and second order correction of wave kinematics.

NOTE The Wheeler stretching method (Wheeler, 1970) is widely used in offshore structure design. In this method the wave kinematics are transferred from the still water level to the free surface by stretching the vertical coordinate.

(9) When the structure is sensitive to drag forces, the applicability of Wheeler’s stretching should be verified, as it can results in underestimation of wave kinematics, especially for steep waves, when used along with a linear surface process.

* + 1. Storm-representative wave approach

(1) The storm-representative wave approach may be used for fixed offshore structures whose load effects are primarily quasi-static in nature.

(2) The storm-representative wave height should be established based on relevant data from the location of interest and the suitable range of wave periods need to be specified.

(3) The data should be corresponding to an annual exceedance probability of *q*, determined by a long-term analysis.

(4) The storm-representative wave height and period shall then be combined in such a way that the load effects on the structure is established with acknowledged design methods.

NOTE In the absence of site specific data on storm-representative wave height, NORSOK N003 suggests the extreme storm-representative wave height can be taken as 1,9 times the significant wave height *H*s,q, corresponding to an annual exceedance probability *q* determined by the long-term statistics using a 3-hour sea-state.

(5) The storm-representative wave period should be determined as the most conservative of pre-defined values from the site when no specific data are available.

(6) For the North Sea, the range of values in Formula (B.1) may be used:

 (B.1)

where

|  |  |
| --- | --- |
| *H*q | is storm-representative wave height corresponding to an annual exceedance probability of *q* in meters and *T* in seconds. |

NOTE Appropriate values of the return periods *T* corresponding to annual exceedance probabilities *q* are given in Clause 4.

* 1. Wave and current actions on structures
     1. General

(1) Appropriate wave theories should be selected according to Clause 5.

NOTE 1 For calculating the actions from waves and currents on a single structural element, see Clause 6.

NOTE 2 This subclause B.5 is written according to design actions on fixed structures from waves and currents calculated by use of a regular “storm-representative wave” according to Clause 5, and a steady state current field not varying in time or space.

(2) The nature of the wave–structure interaction may be defined by determining the relationship between:

* characteristic dimension of the submerged part of the structure *D* and wave length *λ;* or
* wave height *H* and the characteristic dimension of the submerged part of the structure *D*.

NOTE 1 The characteristic dimension is related to the *KC* number (*KC* = *u*max*T*/*D*).

NOTE 2 The characteristic dimension can be the diameter of a submerged member of the structure.

(3) For deep water waves and vertical cylindrical elements located in the vicinity of the sea surface *KC* number may be chosen as *KC* ≈ *πH/D*.

* + 1. Waves and current actions on slender structures
       1. Steady flow – no waves

(1) The flow-induced force *dF* per unit length depends on the dynamic pressure *ρ*w*U*2/2 and the projected area of the cylinder exposed to the flow. This force may be expressed by the non-dimensional force coefficient, see Formula (B.2):

 (B.2)

NOTE 1 The coefficient corresponding to the maximum of fluctuating in-line forces (drag forces) is denoted by *C*’D, and the time-averaged values by *C*D. Subscript S in *C*DS indicates steady flow.

NOTE 2 The coefficient corresponding to the maximum of transversal forces (lift forces) are denoted by *C*’L, and the time-averaged values by *C*L. Subscript S in *C*LS indicates steady flow.

NOTE 3 Steady in-line current force coefficients (*C*DS), depend mainly on the Reynolds number *Re* = *u*max*D*/*ν* and surface roughness of the cylinder, and are shown in Figure B.1.

NOTE 4 In Figure B.1 there is a distinct drop in the drag coefficient in a certain Reynolds number range. This is referred to as the critical flow regime and is very pronounced for a smooth circular cylinder.

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**Key**

|  |  |
| --- | --- |
| 1 | smooth |

Figure B.1 — Drag coefficients (*C*DS) for fixed circular cylinder in steady flow in the critical flow regime, for various roughness

(2) The surface roughness height *k* and thickness *t* may be defined as given in Figure B.2.

(3) As guidance for the surface roughness used for determination of the drag coefficient on steel and concrete pipes, the values in Table B.1 may be used.

(4) The nominal diameter should be increased from *D*c to *D* in case of heavy marine growth.

NOTE Natural marine growth on piles and platforms will generally have *e* = *k*/*D* > 10-3. Marine growth will depend on the location.

(5) Site-specific data should be used to reliably establish the extent of the hydrodynamically rough zones.

NOTE 1 In the absence of accurate data, NORSOK N003 provides guidelines on marine growth thickness and density at different water depths for Norwegian Sea and North Sea (Table B.2).



Key

|  |  |
| --- | --- |
| 1 | hard growth |
| 2 | pipe |
| *e* | = *k*/*D* |
| *D* | = *D*C + 2*t* |

Figure B.2 — Definition of surface roughness height and thickness (from EN ISO 19902:2020)

Table B.1 — Surface roughness of pipes

|  |  |
| --- | --- |
| Material | *k*  m |
| Steel, new uncoated | 5 x 10-5 |
| Steel, painted | 5 x 10-6 |
| Steel, highly corroded | 3 x 10-3 |
| Concrete | 3 x 10-3 |

Table B.2 — Indicative thickness and density of marine growth and biofouling. The water depth refers to mean water level (MWL)

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Depth m | 56° to 59° N | | 59° to 72° N | |
| Thickness 𝒕  mm | **Density 𝝆mg**  kg/m3 | Thickness 𝒕  mm | **Density 𝝆mg**  kg/m3 |
| Above +2 | 0 | - | 0 | - |
| -15 to +2 | 100 | 1300 | 60 | 1325 |
| -30 to -15 | 100 | 1300 | 50 | 1325 |
| -40 to -30 | 100 | 1300 | 40 | 1325 |
| -60 to -40 | 50 | 1300 | 30 | 1100 |
| -100 to -60 | 50 | 1300 | 20 | 1100 |
| Below -100 | 50 | 1300 | 10\* | 1100a |
| a Cold water corals can build up local colonies with no limitation regarding size in water depths between 100 and 800 m. Cold water corals are assumed not to occur for temperatures below 2° C, i.e. for the Norwegian continental shelf it can be assumed that these occur in water depths between 100 m and 450 m. The density of the marine growth may be taken as 1300 kg/m3 in the whole water depth range where cold water corals can be found. | | | | |

NOTE 2 For further information, see NORSOK N-003:2017.

(6) For high Reynolds number *Re* > 106 and large *KC* number (*KC* > 14), the dependence of drag coefficient on roughness may be taken from Formula (B.3):

(B.3)

NOTE 1 Drag coefficient for non-circular members can be found in DNVGL-RP-C205.

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Key

|  |  |
| --- | --- |
| 1 | Jones (1989) |
| 2 | Blumberg (1961) |
| 3 | Wolfram (1985) |
| 4 | Miller (1976) |
| 5 | Szecheny (1976) |
| 6 | Achenbach (1971, 1981) |
| 7 | Want (1988) |
| 8 | Roshco (1961) |
| 9 | Norton (1983) |
| 10 | Nath (1987) |
| 11 | Rodenbusch (1983) |

Figure B.3 — Dependence of steady flow drag coefficient (*C*DS) for circular cylinders on relative surface roughness (*e* = *k*/*D*) for post-critical Reynolds numbers

NOTE 2 Fluctuating forces are strongly related to the presence of vortex shedding and the time-varying asymmetric low-pressure distribution on the lee side of the cylinder. In general, *C*’LS > *C*’DS. These time-varying forces are the driving mechanism for the generation of vortex induced vibrations, see Clause B.11.

NOTE 3 In the subcritical flow regime (5 103 < *Re* < 5 105) Zdravkovich (1997) shows rms values of *C*’LS up to 0,8, while rms values of *C*’DS < 0,1. In the subcritical range the fluctuating forces are quite deterministic with a dominant sinusoidal variation in time with a frequency *f* = *f*v for the lift forces, and frequency *f* = 2 *f*v for the in-line forces, *f*v given by the Strouhal number *St*= *f*v *D*/*U* ≈ 0,2.

NOTE 4 In the fully turbulent regime (*Re* > 106), rms *C*’LS is shown up to 0,2, while rms *C*’DS < 0,1. In the fully turbulent flow regime the time variation of the force is quite stochastic with a Strouhal number varying between 0,15 and 0,5 (Lienhard, 1966).

NOTE 5 This paragraph (6) is valid as long as disturbance from influencing parameters such as, wall blockage, aspect ratio (*l*/*D*), transversal or inline oscillations, wall proximity, free end etc. have negligible significance.

NOTE 6 More information on force coefficients discussing influencing parameters, can be found in Sumer and Fredsøe (1997), Zdravkovich (1997) and Zdravkovich (2003).

* + - 1. Waves – no currents

(1) The decision schemes to be used for calculating inline forces on circular cylindrical elements in waves may be assessed based on Figure B.4.

NOTE 1 In the case of the load on surface piercing vertical cylinders, Figure B.4 is not in conflict with the statement in 6.1.5(2) as long as the Keulegan-Carpenter number and Reynolds number are evaluated at the SWL for the determination of *C*M and *C*D and replacing *H*/*D* in Figure 6.1 with *KC*/*π*.



Figure B.4 — Decision scheme – how to calculate in-line forces on a vertical cylinder in waves

NOTE 2 For inclined cylinders, see Chakrabarti, 1987 for more information.

(2) Structural members in motion in line with the wave heading may be accounted for by a modification of the velocity and acceleration of the member (structure) where the wave actions are given by Formula (B.3):

 (B.3)

where

|  |  |
| --- | --- |
|  | is the velocity of member normal to its axis positive in the wave direction; |
|  | is the acceleration of member normal to its axis positive in the wave direction. |

(3) For the accurate estimate of the wave forces appropriate values of drag (*C*D) and inertia (*C*M) coefficients shall be used.

(4) Most of the variation in *C*D and *C*M may be expressed as function of relative surface roughness (*e*), Reynolds number (*Re*) and Keulegan-Carpenter (*KC*) number.

(5) For a non-moving cylinder , values of *C*D may be taken from Figure B.5 and values of *C*M may be taken from Figure B.6.

(6) For low *KC* numbers (*KC* < 30), the steady state drag coefficient (*C*D) is modified by the wake effects. The value of wake amplification factor (*Ψ*) as a function of *KC* may be taken from Figure B.5.

(7) The drag coefficient may be calculated as given in Formula (B.4):

(B.4)



Figure B.5 — Wake amplification factor *Ψ* as function of *KC* number for smooth (*C*DS = 0,65 - solid line) and rough (*C*DS = 1,05 - dotted line)

(8) Wake effects may be neglected for large *KC* values as the wake travels farther and decays before it reaches back the cylinder.

NOTE Further information can be found in DNVGL RP C205.

(9) The value of *C*M depends on *KC* and surface roughness, and the variation in *C*M as function of roughness and *KC* can be found in Figure B.6.

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**Figure B.6 — Inertia coefficient as function of *KC* for smooth (dashed line) and rough cylinder (solid line)**

NOTE The effect of *KC* on *C*D and *C*M (see Figure B.5, Figure B.6 and DNVGL RP C205) are valid for vertical and near vertical cylinders. For horizontal members, the effect is different. The *C*D and *C*M values valid for horizontal members can be found in DNVGL-RP-F105.

(10) Values of lift forces on smooth cylinders may be taken from Figure B.7.

NOTE 1 Sarpkaya (1976) measured the transverse force acting on fixed circular slender cylinders in a sinusoidal oscillating flow for a wide range of Reynolds numbers and Keulegan-Carpenter numbers, and relative roughness.

NOTE 2 The curves in Figure B.7 act also as upper limits for *C*L for roughened cylinders. The coefficients presented represent maximum values measured.



Figure B.7 — Lift coefficient as function of Reynolds number for various *KC* for smooth circular cylinder (Sarpkaya, 1976)

* + - 1. Wave-current interaction

(1) The current effects on wave loading should be included by vectorially superimposing current profiles (*U*0) over the wave particle velocity (*u*) generated in the absence of a current and calculated from an appropriate wave theory.

NOTE Further information of this method is given in Sarpkaya (2010).

(2) If doing as described in (1), the horizontal acceleration *a*x and velocity *u*x to be used in the Morison formula to calculate the inertial force and the drag force should be written as in Formula (B.5):

 (B.5)

(3) The Reynolds number and the Keulegan-Carpenter number may be defined as in Formula (B.6):

 (B.6)

(4) The drag and inertia coefficients may be determined from the experimental investigations in waves with no current (see Formula B.4 and Figure B.6).

* + 1. Waves actions on large volume bodies

(1) When the diameter of the vertical cylinder is sufficiently large compared to wave length, the diffraction theory should be used taking into account both incident and scattered waves.

NOTE Many closed form solutions for diffraction theory are available in literatures (Chakrabarti, 1987) for several special but practical structures. A linear diffraction theory for a surface piercing vertical cylinder in a finite water depth was derived by McCamy and Fuchs (1954). McCamy and Fuchs solution is based on linear wave theory.

(2) In order to obtain the wave force calculations using diffraction theory, complete boundary value problem should be solved.

(3) According to MacCamy and Fuchs solution, the horizontal force per unit length (*dF*h) may be equivalently represented as the inertia part of the Morison Formula (B.7):

 (B.7)

where

|  |  |
| --- | --- |
|  | is the water particle acceleration at a given elevation z; |
| *C*M | is effective inertia coefficient. |



Figure B.8 — Variation of effective inertia coefficient *C*M and lag angle *α* of maximum force with parameter *D*/*L*, Dean and Dalrymple (1984)

(4) The effective inertia coefficient for calculating the wave actions may be obtained from Figure B.8.

NOTE This solution shows that there will be a phase lag *α* between the maximum force and the wave as the wave passes the cylinder. This phase lag is depending on the *D*/*λ* ratio (see Figure B.8). MacCamy and Fuchs (McCamy and Fuchs, 1954) analytical solution provides only linear wave force on the structure.

(5) In the vicinity of large bodies, the free surface elevations can be increased due to structural motions, diffractions, radiation, and other non-linear wave effects, e.g. shallow water effects. These should be accounted for in the wave actions calculations.

NOTE Higher order solutions for diffraction theory can be found in Chakrabarti (1987).

(6) An analytical solution for wave forces on large structures may be obtained by Froude-Krylov method.

NOTE 1 This method is limited to structures which are neither too small compared to wave length nor too large to have reflection from the structure.

NOTE 2 Closed form force expressions for basic shapes of the structure are given in Chakrabarti (1987).

(7) For general structures consisting of several large volume components, boundary element methods (BEM) or finite element methods or Computational Fluid Dynamics (CFD) should be used.

EXAMPLE Isaacson and Sarpkaya (1981), Faltinsen (1990).

(8) Care should be taken in such methods so that convergence of the result is checked.

(9) Wave forces on vertical walls is a result of both linear reflection and nonlinear wave interaction. At a first approximation the linear forces based on linear wave theory for a regular wave and complete reflection off the wall may be calculated as given in Formula (B.8):

(B.8)

(10) In addition, a nonlinear mean force appears. This force may be calculated as given in Formula (B.9):

(B.9)

where

|  |  |
| --- | --- |
| *H*i | is the height of the incoming wave of normal incidence. |

NOTE For storm-representative wave forces on vertical structures, see Clause 8 and related guidance in Annex D.

* 1. Seabed scour at cylinders due to waves and currents

(1) The assessment of local scour should be documented.

(2) The assessment of local scour may be based on experience and literature sources.

NOTE Validated semi-empirical formulae for sediment transport theory can be found in US Army Corps of Engineers Coastal Engineering Manual (2002), Whitehouse (1998), Sumer and Fredsøe (2002), OCDI (2002).

* 1. Clusters of cylinders

(1) The effects of the wake formation on the upstream and side of cylinders shall be taken into account.

NOTE 1 The particle velocity in the vicinity of the cylinder is modified by the wake and the wave forces is influenced.

NOTE 2 Reed et al. (1990) give drag coefficients for single cylinder and a cluster of cylinders in a unidirectional back and forth motion and with different orientations. Chakrabarti (1990) gives information on the maximum wave forces on cylinders in single row for different cylinder spacing and different wave direction and different *KC* numbers.

NOTE 3 Hildebrandt et al. (2008) investigated the influence of neighbouring cylinders on the wave loads of single isolated cylinder. They tests were performed for tandem and side by side cylinder configurations. The key points to be noted are:

* there was a considerable increase in wave loads when the cylinders were arranged side by side. The drag coefficients obtained ranges from 1,15 - 1,55, exceeding the single cylinder drag value by 60 % in average. The inertia coefficients values also exceeded single cylinder values by 35 % to 45 % in average;
* for tandem arrangement, reduction in wave forces were observed. For *KC* < 23, there was no definite pattern of variation in *C*D and *C*M values. However, for *KC* > 23, there was a reduction in *C*D values 25 % - 10 % in average.

(2) A reduction factor should be applied on the drag and inertia coefficients to account for the wave forces on closely spaced slender structures that are reduced by the shielding effect from neighbouring ones.

NOTE API RP 2A-WSD (2002) gives additional information on shielding factors.

(3) In the case of cylindrical structures supporting platform (e.g. Jetty), the global force on the group of piles should be obtained considering the summation of forces on the individual piles at various phase angles.

(4) It may be assumed conservatively that the piles are subjected to long crested waves.

(5) The breaking wave forces on the piles may be considered individually depending upon the spacing between the piles.

NOTE The slamming force due to breaking waves on the piles are impulsive in nature and are of short duration.

(6) In the global force calculations, the breaking forces on the piles should be considered such that the wave doesn’t break on all the piles at the same time. i.e., the spatial position of the breaking waves needs to be taken into account.

(7) Numerical models may be used for more accurate estimation of the global forces on the pile groups.

NOTE Such models can use higher order wave theories and can also represent the actual waves in time and space, based on spectral density and directional spreading. They can also account for the mutual interactions between the piles.

* 1. Long-crested and short-crested wave action

(1) In most of the situations, wave field may be assumed to be two dimensional for calculating wave actions on the structures.

NOTE 1 Such a description doesn’t provide complete information on real ocean surface. The occurrence of short crested waves are common in nature. They can occur due to strong winds which has spatially varied turbulence intensity, cross propagation of two incoming waves at different directions and lateral modulation of long crested waves.

NOTE 2 Halliwell and Machen (1981) concluded that the short crested waves travels faster and attains higher breaking height than the component waves. Hsu et al (1979) propose a nonlinear wave theory to obtain kinematics of short crested waves. The proper estimation of wave forces by Morison formula relies on accurate description of water particle kinematics. Zhu (1993) proposed exact solution for the diffraction of short crested waves on a circular cylinder.

NOTE 3 Many numerical models are available to simulate short crested wave kinematics. Higher Order Spectral method (Ducrozet et al, 2016) is one of the robust numerical model for simulating nonlinear waves.

* 1. Wave impact and slamming actions
     1. General

(1) The hydrodynamics of wave slamming is complex and simplified methods may be used to calculate slamming actions on structures.

(2) For circular cylindrical members, the slamming force per unit length should be calculated as in Formula (B.9):

 (B.9)

where

|  |  |
| --- | --- |
| d*F*s | is slamming force per unit length in the direction of the velocity; |
| *ρ*w | is mass density of water; |
| *C*s | is slamming force coefficient; |
| *D* | is member diameter; |
| *V* | is relative velocity of water surface to the surface of the member. |

NOTE The most investigated parameter with respect to Formula (B.9) is the slamming coefficient. The value of *C*s is found to be in the range of *π* - 2*π*.

(3) For a smooth circular cylinder the slamming coefficient may be taken as *C*s = 2 *π*.

* + 1. Slamming actions on vertical and inclined cylinders on uniformly sloping or horizontal bottoms

(1) The total wave force on vertical and inclined cylinders in the event of wave breaking may be calculated as in Formula (B.10):

(B.10)

where

|  |  |
| --- | --- |
| *F*D+*F*I | is the Morison force; |
| *F*s | is the slamming force. |

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**Key**

|  |  |
| --- | --- |
| *η*b | is maximum elevation of the free surface |
| *C* | is wave celerity |

Figure B.9 — Sketch of a plunging wave hitting a vertical pile

NOTE 1 The wave slamming forces are large impact forces with a very short impact duration of the order of milliseconds.

NOTE 2 For more information, see Wienke and Oumeraci (2005).

NOTE 3 The drag and inertia components of the force are quasi-static in nature, vary in time accordance with the wave surface elevation. In addition to these forces, there will be dynamic amplification of these forces, governed by the structural properties of the cylinder.

(2) Formula (B.9) represent slamming force per unit length of the cylinder, and the total slamming force may be obtained by integrating the line force along the impact length (*λ*c *η*b) where *λ*c is the curling factor, which describes the part of the surface elevation *η*b which contributes to the impact.

(3) A general formula for calculating wave slamming force, applicable for vertical and inclined cylinders is given in Formulae (B.11) and (B.12):

(B.11)

(B.12)

where

|  |  |
| --- | --- |
| *β* | is (*V* cos*γ*)/*R;* |
| *R* | is the radius of the cylinder; |
| *V* | is the impact velocity; |
| *γ* | is the angle between the direction of the impact velocity *V* and the normal to the cylinder axis, which is 0 for a vertical cylinder; |
| *λ*c | is the curling factor. |

NOTE For more guidance on application of Formula (B.11) and Formula (B.12), see Wienke and Oumeraci (2005).

(4) The impact duration may be calculated by Formula (B.13):

(B.13)

where

|  |  |
| --- | --- |
| *γ* | is the angle between the direction of the impact velocity V and the normal to the cylinder axis, which is 0 for a vertical cylinder; |
| *R* | is the radius of the cylinder; |
| *V* | is the impact velocity; |
| *T* | is the impact duration. |

NOTE The curling factor is obtained based on experimental data (see Figure B.10). The maximum curling factor for vertical cylinder is *λ*c = 0,46. The curling factor increases with increasing negative inclination of the cylinder.

(5) For a vertical cylinder, the maximum slamming force occurs when wave breaks in front of the cylinder (*t* = 0) and impact velocity reaches the wave celerity C at the breaking location may be obtained from Formula (B.14):

(B.14)

where

|  |  |
| --- | --- |
| *C* | is the wave celerity.  A group of screws on a black background  Description automatically generated |

Figure B.10 — Curling factor

(6) For calculating the slamming forces on horizontal cylinder, a method given by Kaplan (1992), may be used.

NOTE Further guidance on the Kaplan method can be found in DNV GL C205.

* + 1. Wave actions, including slamming actions, on vertical cylinders on reefs and shoals

(1) Special attention should be given to vertical structures erected upon reefs subjected to breaking waves (classified as high design uncertainty).

NOTE 1 For more information, seeGoda (1973), Hovden and Tørum (1991), Kyte and Tørum (1996), Hanssen and Tørum (1999).

NOTE 2 The height of breaking waves over shoals, Lie and Tørum (1991), and the wave kinematics (Goda (1973), Hanssen and Tørum (1999), differs considerably from the uniformly sloping bottom. No numerical wave program on the wave heights and wave kinematics is yet available to cover the breaking wave conditions on steep shoals.

(2) General results from laboratory wave flume investigations should be used with care and not for conditions deviating too much from the shoal configurations used during the laboratory tests.

(3) Alternatively, computational fluid dynamics (CFD) may be used to estimate the distribution of wave forces on the structure, alone or in combination with physical model testing if required.

(4) In accordance with Table 4.3, site specific hydraulic model tests should be carried out in some cases.

* + 1. Wave-in-deck forces

(1) The deck structure adjacent to the platform columns should be designed against possible wave actions on the deck due to run up along the columns.

(2) The deck impact analysis should be performed unless it is demonstrated that the air gap is sufficiently large enough to avoid wave in deck impact.

(3) Wave in deck load assessment may be either global approach or detailed component approach.

NOTE Based on this philosophy there are many load assessment models. Global approach (API RP2A model, the Shell method (Tromans and Graaf, 1995), the Statoil method (Dalane and Haver, 1995) provides an overall assessment of wave in deck loads on the platform whereas the detailed component approach (Kaplan model, the Chevron model (Finnigan and Petrauskas, 1997) and the Amoco model (HSE, 1998) provides detail assessment of the force on a component level.

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Key

|  |  |
| --- | --- |
| a | overtopping |
| b | overall drag |
| c | structure slam |
| d | pile slam |
| e | uplift slam |
| f | pulsating pile load |

Figure B.11 — Waves loads on piers

(4) The vertical and horizontal wave in deck forces may be obtained using Formulae (B.14) and (B.16).

NOTE 1 Kaplan (1992), Murrey et al. (1995) and Murray et al. (1997) developed a set of equations to calculate the wave actions on almost any form of the platform. According to HSE (2002), Kaplan’s approach is state-of-art in predicting impact loads on platform decks. Kaplan’s model is based on Morison formula, in which the wave induced forces on the deck is considered as combination of momentum (inertia + added mass) and drag force.

NOTE 2 In Kaplan model, the diffraction effect due to the deck is not considered. In the case of large volume structures, the added mass calculated by Kaplan model is underestimated. The comparison of the model test results showed that the Kaplan model underpredicted the wave forces on the structural components.

NOTE 3 Details on assessing wave in deck forces can be found in DNV GL RP C205.

(5) Assuming the wave propagation direction along the length of the cylinder, the wetted area of the deck may be approximated by a flat plate of length L and width B, and the vertical force time history may be calculated by Formula (B.15):

 (B.15)

where

|  |  |
| --- | --- |
| *Ma*z | is the vertical added mass for the rectangular flat plate deck given by Formula (B.16): |

 (B.16)

where

|  |  |
| --- | --- |
| *w* = *w*(*t*) | is the vertical velocity underneath the deck; |
| *L*=*L*(*t*) | is the wetted length. |

(6) The horizontal force time history may be calculated by Formula (B.17):

(B.17)

where

|  |  |
| --- | --- |
| *c* = c(*t*) | is the instantaneous wetted height; |
| *u* = *u*(*t*) | is the instantaneous horizontal particle velocity in undisturbed wave. |

(7) Assuming the maximum value of c is much smaller than the horizontal width *B*, the lateral added mass may be calculated by Formula (B.18):

 (B.18)

(8) The drag coefficient *C*D may be taken as 2,0.

(9) The model proposed by Cuomo et al. (2007) for calculating wave in deck loads. may be used to estimate the wave actions on underneath slabs and decks.

NOTE The model is based on the physical model tests performed at HR Wallingford (McConnell et al. (2004)).

(10) For a general deck of (arbitrary shape, computational Fluid Dynamics (CFD) may be used to access the wave actions on the structure members.

NOTE The effects of compressed air can be included in the analysis.

* + 1. Air gap calculations and recommendations

(1) When slamming actions is not considered, a positive air gap shall be maintained to minimize wave in deck forces.

NOTE Air gap considerations are relevant for the deck structures of both fixed and floating platforms. A negative air gap means there is impact between wave and structure.

(2) Normal implementation may be to assume a positive air gap in an accidental load scenario.

NOTE For more guidance, see EN 1990:2023, Annex E, and EN 1991-1-7.

(3) Alternatively, a positive air gap of 5 % of the wave amplitude according to Table 4.3 may be assumed.

(4) According to DNV, the air gap requirement should be taken as 1 m for 100-year storm-representative wave crest or 20 % of the 50-year significant wave height, whichever is larger.

(5) Installation tolerance and global sea level rise shall be included in the air gap assessment.

(6) Accurate estimation of wave crest height is important. Higher order waves should be considered when and where required.

(7) Depending on HEA class and design uncertainty, model tests should be performed to validate the calculations.

NOTE Installation tolerances and sea level rise can influence air gap assessment.

(8) Instantaneous air gap may be calculated by Formula (B.19):

(B.19)

where

|  |  |
| --- | --- |
| *a*0 | is the distance between highest water level and bottom of the deck; |
| *z* | is the vertical motion of the structure; |
| *η* | is the free surface elevation. z is zero in case of fixed structures. |

* + 1. Dynamic amplification and vibrations

(1) Dynamic effects shall be considered while evaluating the structural design.

(2) For structures whose dynamic effects are small, the DAF may be calculated using a simplified single degree of freedom system approximation (Biggs, 1964).

(3) For an inertia dominated structure, DAF’s may be obtained from a representative dynamic equivalent model.

NOTE This approach is usually combined with an environmental contour method.

(4) Screening of the global responses of the structure may be performed along the environmental contour in order to tune the dynamic model with respect to the most severe sea state.

(5) The following steps may be performed:

* obtain the candidates of *H*s - *T*p along the environmental contour corresponding to annual exceedance probability of *q*;
* perform the screening of the sea states obtained in step 1 based on the global response of the structure (typically, OTM and base shear) and obtain the most severe sea state;
* tune an equivalent hydrodynamic model (simplified model with beam elements), to correspond to the inertia forces obtained from quasi static response;
* DAF is obtained as the ratio of the dynamic response and the quasi-static response. DAF’s established from the hydrodynamic model is combined with the quasi-static response from the storm-representative wave approach.

(6) This method may be used to establish the DAFs by a simplified model, rather than through the full global analysis model where the computational effort will be significant.

NOTE Further information on this method can be found in NORSOK N-003.

(7) A structure should be investigated for the dynamic response to slamming actions.

NOTE When actions characterized with a short duration compared with the resonance period of the structure, depending on the ratio (*td*/*T*), the response force can be amplified. Slamming actions are characterized by high peak intensity and a very short duration. Depending on the elasticity and damping characteristics of a structure, the effect of slamming actions depends on the dynamic response of the structure and its foundation, which may amplify or dampen the peak force. The dynamic amplification greatly depends on the shape of the impact force and duration (Biggs,1964).

(8) Standard charts and formulas may be used for the preliminary analysis, assuming simple mode of vibration.

NOTE In Figure B.12 maximum response of one-degree elastic undamped system subjected to different impulse load from Bergan, Larsen & Mollestad, 1986 are given.

(9) For structures with larger natural periods, ringing effects shall be considered.

NOTE Ringing and springing are two high frequency response effects. Ringing occurs when the vertical members of the structure is encountered by high steep nonlinear waves. Springing is rather steady state response often associated with 2nd order waves. Ringing is transient in nature and decays to steady state depending on the system damping. Ringing is relevant for TLP and GBS structures. Due to strong nonlinear nature of these phenomena, numerical models often lack accurate predictions and demands physical model testing (see DNV GL RP C205, 2019).

(10) Model tests should be performed in order to quantify effects of ringing and springing.

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Figure B.12 — Maximum response of one-degree elastic undamped system subjected to different impulse load

* 1. Subsea pipelines

(1) For simple analytical design the horizontal action on a pipe length *dl* for smaller structural vibrations compared with the fluid particle motion may be calculated by Formula (B.20):

 (B.20)

and the lift actions by Formula (B.21):

 (B.21)

where

|  |  |
| --- | --- |
| *α* | is the angle between the wave direction and the pipeline axis. |

NOTE 1 Wave and current actions on pipelines are most extensively dealt with by the oil industry and are treated in different standards and guidelines, see DNVGL-ST-F101, DNVGL-RP-C205 and DNVGL-RP-F105 for further details and guidance.

NOTE 2 The model by Formulae (B.20) and (B.21) predicts that the vertical actions are in phase with the water particle velocities. But measurements show that the vertical actions are ahead of the undisturbed water particle velocities above the pipeline. There has been developed more refined methods to obtain the horizontal and vertical actions that predicts more correctly the phases, Lambrakos et al. (1987), Verley et al. (1987). However, these methods are cumbersome to use and are not employed unless it is very critical for the pipeline stability considerations.

NOTE 3 The drag, inertia and lift coefficients *C*D, *C*M and *C*L are depending on the Reynolds number, the Keulegan-Carpenter number, the gap to diameter ratio, *e*/*D*, the current flow velocity ratio *β* and the pipe surface roughness.

NOTE 4 DNVGL-RP-F105 gives a detailed recommendation of the *C*D and *C*M coefficients including the effects of the current flow velocity ratio, the seabed proximity, pipe surface roughness, a pipe in a trench and cross flow vibrations.

NOTE 5 Sumer and Fredsøe (1997) gives values of *C*L as a function of seabed proximity, pipe roughness and Keulegan-Carpenter number *KC*.

(2) For larger structural vibrations, Formulae (B.20) and (B.21) should be modified by replacing the absolute velocity, *u* with the relative velocity between the structural element and the surrounding fluid and including an added mass term due to structural vibrations (see Formula. (B.3)).

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**Figure B.13 — Pipeline on the sea bottom (a) or in a trench (b)**

(3) Detailed numerical or experimental analyses should be considered for estimating hydrodynamic force coefficients for open pipeline bundle systems.

NOTE 1 In addition to the horizontal and lift actions, load effects due to instability oscillations such as in-line, cross-flow oscillations and galloping and other effects, e.g. due to buoyancy variations and marine growth, can be relevant.

NOTE 2 For more information on estimating hydrodynamic loads on pipelines, see DNVGL-RP-F105 and DNVGL-RP-C205.

(4) The current flow velocity ratio, *β* may be calculated by Formula (B.22):

 (B.22)

where

|  |  |
| --- | --- |
| *U*c | is the mean current velocity; |
| *U*w | is the wave velocity. |

* 1. Vortex induced vibration of pipelines

(1) Assessment of the risk of vortex induced vibration (VIV) in pipelines should be done according to acknowledged literature and standards.

NOTE 1 Water or any fluid past a slender member can cause unsteady flow patterns due to vortex shedding. At certain critical flow velocities, the vortex-shedding frequency can lock-in with or be a multiple of the natural frequency of motion of the member, correlating the vortices along the cylinder and resulting in harmonic or sub-harmonic excitations, either in-line with the flow or cross flow.

NOTE 2 The oscillation amplitudes are affected by several environmental and response parameters, such as the reduced velocity *V*r, the stability parameter *K*s, the relative flow angle, the current flow velocity ratio, the turbulence intensity and the seabed proximity.

(2) The vortex shedding frequency may be calculated using Formula (B.23):

(B.23)

where

|  |  |
| --- | --- |
| *f*v | is vortex shedding frequency; |
| *St* | is Strouhal number; |
| *U* | is flow velocity normal to the slender member axis; |
| *D* | is diameter of the member. |

NOTE For circular cylinders the Strouhal number (*St*) is a function of the Reynolds number (*Re*). In a wide range of Reynolds numbers, the Strouhal number is *St* ≈ 0,2.

(3) For determination of the velocity ranges where vortex induced vibration (VIV) can occur, the reduced velocity *V*r defined in Formula (B.24) may be used:

(B.24)

where

|  |  |
| --- | --- |
| *fj* | is natural frequency of vibration for the *j*th mode of the structural member. |

(4) The stability parameter, *K*s, representing the damping. may be calculated using Formula (B.25):

(B.25)

where

|  |  |
| --- | --- |
| *m*e | is effective virtual mass per unit length; |
| *δ* | is generalized logarithmic decrement (2*πξ*) of total damping defined by Formula (B.26): |

*δ* = *δ*s + *δ*soil + *δ*h (B.26)

where

|  |  |
| --- | --- |
| *δ*s | is structural damping; |
| *δ*soil | is soils damping or other damping |
| *δ*h | is hydrodynamic damping. |

(5) The amplitude of vibration ratio, *A*/*D*, where *A* is the vibration amplitude, is primarily a function of the reduced velocity and may be reduced by increasing value of *K*s.

NOTE For further guidance, see DNVGL-RP-F105 and DNVGL-RP-C205.

* 1. Tools to support design
     1. Numerical models
        1. Linear models

(1) The boundary element method (BEM) may be used for both linear and the second order problems for solving potential flow:

* in the boundary element method, geometry of the structure is defined with panels. Depending on flat or curved panels used, it can be either lower order or higher order method respectively;
* BEM requires large number of panels to describe the body of the structure. The accuracy of the results directly depends on the number of panels. The higher order method gives higher accuracy with less number of panels than lower order method;
* higher number of panels are required near the free surface. It is important to perform sensitivity study on the number of panels in order to ensure the accuracy of the solutions;
* special care should be given to remove the discrepancies related to irregular frequencies while using boundary element method for surface piercing bodies (Refer DNVGL RP-N103 for more details);
* advantage of BEM method is that it takes into account the geometry of the structure and also interactions from neighbouring structures;
* the solution is limited to linear events though second order solutions are available in advanced models.

NOTE For large structures, the presence of the structure alters form of incident flow over its vicinity. In such cases, flow can be well described by the potential flow theory.

* + - 1. Computational fluid dynamics

(1) Computational fluid dynamics (CFD) may be used when calculating wave actions on offshore structures.

NOTE 1 The solution involves solving the Naiver Stokes equations in a discretized fluid domain:

* mesh convergence study carried out in order to ensure the mesh independency for the accuracy of the solutions;
* simplification on the geometry is recommended to avoid complex meshing;
* solutions from CFD verified preferable with physical model test results;
* potential flow theory is limited to calculate radiation damping. In the cases of structures where viscous damping is important, CFD can be used to access viscous damping.

NOTE 2 CFD demands higher computation time and resources.

* + 1. Model tests

(1) Hydrodynamic model tests should be performed where the analytical and numerical tools is not proven or fail to produce accurate solutions.

(2) Model tests should also be performed in order to verify the operational aspects for new and innovative design concepts.

NOTE See also Clause B.3 and Clause 4.

(3) Model tests should also be performed when calculating for instance:

* viscous wave drift forces;
* higher order wave loads (ringing);
* moonpool dynamics and sloshing;
* vortex induced vibrations;
* wave slamming.

NOTE See Clause 12.

1. (informative)  
     
   Additional guidance for mound breakwaters
   1. Use of this annex

(1) This informative annex provides supplementary guidance to Clause 7 for wave and current actions on mound breakwaters and in particular for:

* wave action on seaward slope (7.3.2) and toe (7.3.3);
* wave overtopping (7.3.4);
* wave action on breakwater crest and crown wall (7.3.8);
* wave and current action on filter layers and underlayers (7.3.9);
* wave and current actions related to local seabed scour (7.3.11).

(2) This informative annex provides also supplementary guidance to 9.4 for wave and current actions on horizontal-composite breakwaters and in particular for

* wave overtopping (9.4.2);
* wave action on breakwater crest (9.4.5);
* wave action on filter layers (9.4.6);
* wave and current actions related to local seabed scour (9.4.8).

NOTE National choice on the application of this informative annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

* 1. Scope and field of application

(1) This informative annex covers issues related to wave and current actions on mound breakwaters, i.e. topics covered in Clause 7. It applies to structure types presented in 7.1:

* conventional mound breakwaters with natural or artificial armour units;
* berm-breakwaters of the non-reshaped static or reshaped static stable type;
* low-crested and submerged rubble mound breakwaters.

(2) This informative annex can also be used, in combination with other tools, in issues related to wave and current actions on horizontal-composite breakwaters, covered in Clause 9.

NOTE Wave and current actions on coastal structures depend on the actual structure considered and they cannot be determined by existing knowledge in most types of such structures. Instead, there is ample information on the structure response and the stability level associated to those actions per structure type. Given the above this annex provides the user with reference to the most appropriate information on the structure’s response, e.g. which formulae and where to find them, along with rest design data and approaches.

* 1. Conventional mound breakwaters
     1. Failure modes

(1) Due care should be taken when selecting the material, size and shape of the armour units.

NOTE Damage to conventional mound breakwaters includes several modes of failure as illustrated in Figure C.1. The major failure mode is the erosion of the seaward slope by removal or breakage of armour units by wave action. If the armour layer is removed at a certain area the material of the underlying layer is subject to direct wave action. This can lead to its damage, loosing of core material and finally to extended failure. Other key causes of major damage induced by wave and current action are armour toe deformation (modes g and i in Figure C.1) and severe wave overtopping (mode a) that can cause crest erosion (mode c1) and leeside armour erosion (mode d).

(2) Geotechnical-type failures are quite common and should be carefully assessed.

NOTE 1 Failure mode c1, i.e. crest erosion, may cover a larger area of potential damage in the absence of capping wall; otherwise it is concentrated at the toe area of the reflecting wall.



Key

|  |  |
| --- | --- |
| 1 | armour |
| a | crest instability due to overtopping |
| b | erosion of armour |
| b1 | fracture of armour units |
| c | breakage, sliding, tilting of capping wall |
| c1 | crest erosion |
| d | erosion of leeside armour |
| e | slip failure |
| e1 | inner slip |
| f | venting |
| g | berm instability |
| h | core settlement |
| i | seabed scour and toe erosion |
| j | filter permeability |
| k | subsoil settlement |

Figure C.1 — Overview of failure modes for rubble mound breakwaters (Burcharth, 1997, with modifications)

NOTE 2 Failure modes ‘h’ and ‘k’, i.e. core and subsoil settlement, can be due to liquefaction induced by seismic excitation of fine soil prone to this process.

(3) The failure modes described in (1) and (2) should be checked along with modes associated to 3D effects, in particular at lee-side parts of roundheads and at structural transitions.

NOTE For more information, see The Rock Manual (2007), 6.1.4.3.

* + 1. Fault tree

(1) In order to be able to conduct a sensible design of a conventional mound breakwater one should comprehend the failure mechanisms at play that induce damage to the structure.

(2) These mechanisms should involve the failure modes (C.3.1) along with their combinations and interrelationships, if any.

NOTE An illustrative mapping of the failure modes along with their causality linkages is obtained by the so-called fault-tree of the structure. Fault-trees are based on cause-consequence diagrams and are quite helpful for applying probabilistic, reliability, and resilience analyses.

(3) For practical applications a fault-tree for conventional mound breakwaters, simplified to contain only the critical failure modes mentioned in C.3.1, as presented in Figure C.2, may be used.

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Figure C.2 — Simplified fault-tree for a conventional mound breakwater

NOTE 1 Crest instability due to wave overtopping (mode a) normally takes place when no capping wall is present.

NOTE 2 Erosion of leeside armour (mode d) normally is induced by excess wave overtopping and possibly by a venting process taking place through the upper part of the permeable layers of the mound.

(4) In geotechnical modes of failure, soil liquefaction should be considered when applicable.

(5) The analysis of existing coastal structures should establish physical or statistical relations between the various modes, i.e. in general terms which failure mode is affected by the realisation of another one or is triggered only by a driver common to another mode.

EXAMPLE Hydraulic action.

NOTE 1 The simplified diagram in Figure C.2 shows at each mode panel the corresponding failure mode index depicted in Figure C.1.

NOTE 2 Figure C.2 also embraces two main origins of the failure mechanism: hydraulic and geotechnical. These two main drivers may not be regarded as acting independently from one another, hence the inclusion of the most significant geotechnical aspects in this document.

(6) The mound breakwater may be regarded as a system comprising sub-systems that contribute to the total probability of failure of the system through their corresponding probabilities of failure that are made up from the failure probabilities of their associated individual failure modes.

NOTE 1 There are two fundamental types of these sub-systems: those that have their failure modes operating in series and those where the modes operate in parallel.

NOTE 2 For further guidance, see Annex H.

(7) The fault tree of the mound breakwater system may be decomposed into series and parallel sub-systems.

NOTE When decomposing, the fault tree in Figure C.2 can yield the sub-systems and their failure calculation line in Figure C.3.

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Figure C.3 — Sequence of probability of failure calculation based on decomposition of the fault tree into sub-systems: all sub-systems shown are of the series type, except for sub-system h-k (for notation, see Figure C.2)

(8) Since the evaluation of the failure probability depends largely on the decomposition of the fault tree, the decomposition should be done with care, such that only the critical failure modes would be allowed to directly cause system failure.

EXAMPLE In Figure C.3 modes b (outer armour) and S (geotechnical settlement) could be selected to cause direct system failure.

* + 1. Design approaches and formulae

(1) For any design approach related to conventional mound breakwaters, formulae needed to describe the response of the structural elements associated to the failure modes presented in C.3.1 may be found in the technical literature and specifically in the following documents:

* The Rock Manual (2007);
* Coastal Engineering Manual (2003-2011);
* EurOtop Manual (2018).

NOTE 1 Design approaches for conventional mound breakwaters largely depend on the consequence class of the specific structure under consideration and the uncertainty associated to both the hydrodynamic estimate and the responses with associated formulae.

NOTE 2 Guidance on the selection of the design approaches can be found in Clause 4.

NOTE 3 The Norwegian Breakwater Construction Recommended Practices (Norwegian Coastal Administration, 2019) is a useful recent compilation of best practices related to the design and implementation of breakwaters.

(2) Given that response formulae for various failure modes are continuously updated through research the recommendations in the following subclauses should only be adopted in such a temporary context.

(3) When seaward slopes of a mound breakwater are armoured by rocks or by prefabricated concrete units, the required mass of the prefabricated concrete units may be estimated for several unit shapes/properties through the stability number *H*s/Δ*D*n or the stability coefficient *KD* in the Hudson formula.

NOTE Other parameters likely to be in a formal way assigned to the side of the hydraulic limit state conditions opposite to the action side are the damage parameters, such as the allowable percentage of eroded volume, the percentage of displaced units, etc.

(4) Design values of the stability numbers may be in a very formal way considered as “design resistances” appearing in the hydraulic limit state conditions reflecting the failure modes into consideration.

(5) Structural ULS of crown-wall or another monolithic concrete element may apply when the monolithic element is so large that it cannot deform without significant fracture.

NOTE It reflects failure mode c.

(6) Typical informative values of damage parameters are displayed in Table C.1 for some usual armour units, under failure mode b, may be used.

NOTE 1 The 'start-of-damage' and 'failure state' are similar to the SLS and ULS presented in 7.2.2 and 7.2.3, respectively.

Table C.1 — Typical informative values of damage parameters

|  |  |  |  |  |  |
| --- | --- | --- | --- | --- | --- |
| Sub-system | Unit | Damage parametera | Slope | Start of failureb | Failure statec |
| Two-layer armour | Rock | *D*er | 1:2 – 1:3 | 0 – 5 % | 20 % |
|  |  | *S*d | 1:1,5 – 1:2 | 2 | 8 |
|  |  | *S*d | 1:3 | 2 | 12 |
|  |  | *S*d | 1:4 – 1:6 | 3 | 17 |
|  | Tetrapod | *N*d |  | ~0 % |  |
|  |  | *N*od | 1:1,5 | 0-0,5 | 1,5 |
|  | Cube | *N*d | 1:1,5 – 1:2 | ~0 % |  |
|  |  | *N*od | 1:1,5 | 0-0,5 | 2 |
|  | Dolos | *N*d | 1:1,5 | 0 – 2 % | 15 % |
|  |  | *N*od |  |  | 2,8d |
|  | Accropode | *N*od | 1:1,33 | 0 | 0,5 |
|  | Rockf | *S*d | 1:1,3 – 1:1,4 | 2 | 8 |
| Single-layer armour | Accropode | *N*d | 1:1,33 | 0 | 10 % |
|  | Cube | *N*od |  | 0 | 0,2 |
| Toe berme | Rock | *N*od |  | 0,5 | 4 |
| a *D*er: percentage of eroded volume from the middle of the crest down to 1*H*s below still water level (SWL).  *S*d or *S*: , where *A*e the eroded area around SWL, *D*n50 the median nominal diameter of the armour stones.  *N*d: ratio of number of units moved out of place between levels:  SWL ± 6*D*n for cubes and Dolosse;  SWL + 5*D*n & SWL -9*D*n for Accropodes;  where *D*n nominal diameter of unit.  *N*od: ratio of number of units displaced out of armour layer to the width of tested section normalized with respect to *D*n, or equivalently to *D*n50 for rock.  b Few units are displaced.  c The underlayer (or filter layer) is exposed to direct wave attack.  d For a packing density coefficient *φ* = 0,83 and a waist-to-height ratio *r* = 0,32.  e For a typical toe size of about 3 – 5 stones wide and 2 – 3 stones high.  f Typically for revetments. | | | | | |

NOTE 2 Values for the packing density coefficient of concrete armour units can be found in the Rock Manual (2007), Table 3.47.

NOTE 3 Characteristic and design values of concrete parameters and resistances as well as relevant partial factors are indicated in EN 1992 (all parts).

NOTE 4 Characteristic and design values of geotechnical parameters and resistances as well as relevant partial factors are indicated in EN 1997 (all parts).

(7) Under three-dimensional conditions, such as in roundheads, the values of the damage parameters may be adjusted to cope with the modified reference area.

* + 1. Wave action on the seaward rock-armoured slope

(1) Formulae covering the failure mode <b> in Figure C.1 for wave action on the seaward rock-armoured slope may be the:

* Van Gent formula (Van Gent et al., 2004);
* Van der Meer formula (Van der Meer, 1988; Van Gent et al., 2004);
* Hudson formula (Hudson, 1953, 1959).

NOTE Specific information on several aspects of the formulae, including range of applicability, can be found in The Rock Manual (2007), 5.2.2.2.

(2) In broad terms the Van Gent formula shall be applied only to shallower waters, i.e. in depths less than 3*H*s where *H*s the significant wave height at the toe of the structure.

NOTE 1 The design experience with the Van Gent formula is so far limited.

NOTE 2 Studies (Guler et al. 2013) have shown that the Van Gent formula may be applied if two more constraints are valid, namely *H*2%/*H*s0 < 1,4 and *H*s,toe/*H*s0 < 0,9.

* + 1. Wave action on the seaward slope of artificial units

(1) Care should be taken in some types of artificial units in seaward slopes since milder slopes do not always imply increased stability due to imperfect interlocking of the units.

NOTE The units can be placed uniformly or randomly in double or single layers.

(2) Single layers of units should be designed based on enhanced stability criteria.

(3) Physical model tests should be undertaken to evaluate the stability of the crest of single layer armour types, especially its offshore corner where a weak spot can easily develop.

NOTE 1 The design of the armour layer has a direct impact on the design of the associated underlayer, especially when the armor layer is of the single layer type.

NOTE 2 Design “resistance values” of the stability number *H*s/Δ*D*n or the stability coefficient *K*D in the Hudson formula are suggested along with other design information for concrete armour layers in The Rock Manual (2007), 5.2.2.3.

* + 1. Wave actions on the seaward toe

(1) The toe should be checked as an individual system against sliding due to pressures exerted by the armour layer, especially if the latter is quite steep as in some armours made of precast units.

(2) Empirical formulae for assessment of toe block stability, based on model tests, may be used within their validity range for standard toe solutions, where no wave breaking takes place on the toe.

(3) In cases of limited water depth it should be examined to have the toe protected by embedding it in the natural seabed.

(4) The stability of the seaward toe under wave action may be estimated through the Van der Meer et al. formula 1995.

NOTE Design issues including toe berms in shallow water and gently sloping foreshores are dealt with in The Rock Manual (2007), 5.2.2.9.

* + 1. Wave run-up and wave overtopping

(1) Estimation of the run-up on armoured slopes may be performed by use of formulae based on extensive data analysed in EurOtop (2018), 6.2.

NOTE Wave run-up is a parameter through which knowledge of the number of overtopped waves can be obtained in a conventional mound breakwater.

(2) Also, the Van Gent formulae (Van Gent 2004, Schüttrumpf & Van Gent 2004), and the TAW equations coupled with a roughness reduction factor (TAW 2002) may be used.

NOTE For more information, see The Rock Manual (2007), 5.1.1.2.

(2) Empirical formulae to calculate wave overtopping discharge based on physical model and field experiments may be used for conventional mound breakwaters.

NOTE 1 Wave overtopping is nowadays the prime parameter to decide on the crest elevation of a conventional mound breakwater with or without a crest wall.

NOTE 2 For more information, see EurOtop (2018), 6.3.1.

(3) Also, wave overtopping discharge may be calculated by TAW method (TAW 2002) or by Owen’s method (Owen 1980) when the case considered lies outside the validity range of the former.

NOTE Further details can be found in The Rock Manual (2007), 5.1.1.3.

(4) The effects of a wide crest berm and of oblique wave incidence may be estimated through empirical formulae given in the EurOtop (2018), 63.2 and 6.3.3.

(5) Other routes may also be exploited to estimate wave overtopping discharges in breakwaters of very low permeability or with crown walls.

NOTE Such routes include use of the EurOtop database (4.4) and the EurOtop Neural Network prediction tool, 4.5, see EurOtop (2018).

(6) For more complex cases physical model tests should be undertaken.

NOTE The effect on wave overtopping rates of currents co-existing with waves can be significant only for quite high current velocities, their actual threshold values depending on the associated wave height, e.g. more than 1 m/s for *H*s around 2 m; for more information on this subject visit the EurOtop (2018), 5.4.5.

* + 1. Wave action on the rear armour slope

(1) The required armour of a rear-side slope in conventional mound breakwaters with no crown wall may be calculated by the formula of Van Gent and Pozueta (2005).

NOTE Further guidance on this topic can be found in The Rock Manual (2007), 5.2.2.11.

* + 1. Wave actions on roundheads

(1) Structural response in roundheads may be taken care of by modifying the stability number *H*s/Δ*D*n or the stability coefficient *K*D in Hudson’s formula.

NOTE 1 Wave actions on roundheads are quite complex to model. The stability coefficient decreases along the roundhead from the section of wave attack through to about an angle of 3*π*/4. Also, the stability decreases with the surf similarity parameter.

NOTE 2 Roundheads with armour units are more vulnerable, due to the three-dimensionality of the layout, especially for units relied more on interlocking than on mass. Also, single-layer armour units are quite prone to instabilities at roundheads. Design requirements for the radius of a conventional mound breakwater are, therefore, being proposed. For more guidance on this subject, see The Rock Manual (2007), 5.2.2.13.

* + 1. Wave action on crown walls

(1) The strength of a wall-unit foundation on the mound crest against settlements of the latter should be checked under short-term and long-term conditions.

NOTE 1 Typically, crown walls are designed at ULS, while the mound part of the structure at SLS-LDi.

NOTE 2 Guidance on how to combine the probabilities of failure of the failure mechanisms of a coastal mound breakwater can be found in C.3.2.

NOTE 3 For further guidance on the structural response of crown walls, see The Rock Manual (2007), 5.2.2.12.

(2) Wave action on crown walls, including uplift, may be calculated through Martin’s method (1999) for waves that do not break directly on the crown wall.

(3) For other cases Pedersen’s method (1996) may be employed.

* + 1. Local seabed and underlayers erosion

(1) The increase of the impact of the general-field sediment transport on an erodible bed due to the presence of a conventional mound breakwater cannot be estimated with high credibility, but physical large-scale tests should be performed.

NOTE A level of the erodibility potential of waves on the foreshore of the breakwater can be obtained by the required toe protection, see C.3.6. Wave action on the seaward breakwater slope will normally increase the free-field erosion potential close to the offshore slope, due to the reflection process. Reducing the slope reflectivity can have a positive effect on the local seabed scour.

(2) Stability of the seabed or exposed underlayers under current attack may be estimated through formulae, such as by Escarameia and May (1995) presented along with other formulae in The Rock Manual (2007), 5.2.3.1.

(3) For preliminary estimates the current velocity close to the seabed or the underlayer may be modified by the presence of waves through the procedure proposed in Soulsby (1997).

NOTE Further guidance on this topic can be found in Sumer and Fredsøe (2002).

* 1. Berm breakwaters
     1. Introduction
     2. Failure modes

(1) Failure modes in berm breakwaters, which are of prime importance for the overall stability, may be basically those of conventional mound breakwaters, presented in C.3.1, with the following modifications (see Figure C.1):

* add failure mode b2 dealing with main-berm instability (type a berm breakwaters);
* add failure mode Rec dealing with excessive berm recession (type b berm breakwaters);
* add failure mode Ab dealing with abrasion and stone crushing (type b berm breakwaters).

NOTE 1 A berm breakwater can be either non-reshaping stable or allowed to reshape into statically stable profile:

1. non-reshaping statically stable berm breakwater, i.e. only a few stones can move similarly to what is allowed in a conventional rubble mound breakwater for the storm-representative wave conditions (Icelandic type);
2. partly reshaping statically stable berm breakwater, i.e. the profile is reshaped into a stable profile, where the individual stones are also stable for the storm-representative wave conditions.

NOTE 2 Type a) structure is associated to a stability number *N*s = *H*s/Δ*D*n50 less than 2, whereas type b) to 2 < *N*s < 2,5. The stone diameter *D*n50 refers to the homogeneous cover layer or to the uniform material in single-layer structures.

NOTE 3 Berm breakwaters with stability number higher than 2,5 are not dealt with in this document. A typical cross-section of type (b) berm breakwater is given in Figure C.4. Point A is more or less fixed for any value of the recession Rec.

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**Figure C.4 — Type (b) rubble mound berm breakwater with recession Rec**

NOTE 4 The berm breakwater has been traditionally constructed with a berm that is allowed to reshape into an S-shape. This is because it is cheaper to construct the breakwater with an ordinary berm rather than with the S-shape directly. A more stable design has been developed in Iceland in close cooperation between all stakeholders, Sigurdarson et al. (1998). One reason for this development is the fear that the reshaping process may eventually lead to excessive crushing and abrasion of individual stones as they move on the berm breakwater. The question of allowing reshaping or not has obviously to do with the stone quality and the stones ability to withstand impact crushing and/or abrasion. It is though clear that even a non-reshaping berm breakwater requires cover stones with significantly less mass than required on a conventional rubble mound breakwater. There are methods available to evaluate the suitability of quarried stones against crushing for reshaped statically stable berm breakwaters.

NOTE 5 In cases where not large enough quarried cover stones for a conventional rubble mound breakwater can be provided, a berm rubble mound breakwater can be an economical alternative to rubble mound breakwaters with concrete armour units.

* + 1. Fault tree

(1) A simplified fault tree applicable to berm breakwaters can be drawn based on the diagram of Figure C.2 with the additions of failure modes noted in C.4.2.

NOTE All additional modes belong to the hydraulic cause of failure and are connected, one at a time, i.e. under an <OR> gate, with the rest failure modes of hydraulic origin.

(2) The reliability assessment presented in C.3.2 may be used also for the structures dealt with in Clause C.4.

* + 1. Design approach and formulae

(1) The guidelines on design approach and the applicable manuals given in C.3.3 may be used for berm breakwaters.

NOTE 1 Related material found in Van der Meer and Sigurdarson (2016) and PIANC Report of WG40 (2003) can be useful.

NOTE 2 Characteristic and design values of geotechnical parameters and resistances as well as relevant partial factors are indicated in EN 1997 (all parts).

* + 1. Wave action on the seaward face

(1) The recession up to the 100-year condition may be as given by Van der Meer & Sigurdarson (2016).

NOTE 1 Further guidance on geometric design of berm breakwaters can be found in Van der Meer & Sigurdarson (2016).

NOTE 2 The recession of a given berm armoured by stones of *D*n50 depends on the stability number *H*s/Δ*D*n50, hence on the wave action.

NOTE 3 Breakwaters of type a) with orderly placement of the stones in the berm are of improved stability over type b) structures. Type a) is characterized by very limited mobility of the armour stones. Information on oblique waves, stone velocity, and redistribution of stone size can be found in PIANC (2003). In general terms in type a) berm breakwaters the recession is negligible, while in type b) it can be up to 5Dn50, for the 100-year design condition.

(2) The depth function *h*t (Figure C.4) may be estimated by the formula by Tørum et al. (2003).

NOTE For more information, see The Rock Manual (2007), 5.2.2.6.

* + 1. Rear side stability

(1) Necessary stone size *D*n50 on the rear side may be determined using the relation from Andersen et al. (1992), which is based on:

* the breakwater crest elevation,
* the spectrum-based significant wave height,
* drag and lift coefficients,
* the effective slope on the front side,
* the slope of rear side, and
* a friction factor.

NOTE 1 The rear side stability of a berm breakwater is influenced by the relevant wave action.

NOTE 2 For more information, see PIANC (2003).

* + 1. Stability and reshaping of the berm breakwater head

(1) Due to the reshaping of the berm breakwater head, wave actions onroundheads should be assessed as type a) rather than type b) berm breakwaters.

NOTE 1 Wave action on berm roundheads can induce loss of armour stones away from the offshore profiles receiving the wave attack and accumulate them at the rear profiles of the roundhead. This process does not, however, reach an equilibrium state as in the trunk section profiles, where moving stones remain on their profile.

NOTE 2 Further guidance can be found in The Rock Manual (2007), 5.2.2.13 and in Van der Meer and Sigurdarson (2016).

* + 1. Wave overtopping

(1) The average front slope and the influence of the berm thereupon may be calculated as suggested in the EurOtop (2018), 5.4.6.

NOTE 1 Berm breakwaters can be overtopped by waves depending on their characteristics, mainly crest elevation, average slope, berm width.

NOTE 2 For the effect of currents on wave overtopping, see C.3.7(4).

(2) When the slope is 1:2 or steeper than the berm influence is minimal and wave overtopping may be estimated as in the conventional mound breakwaters (see C.3.7).

(3) For milder slopes than 1:2 the formula by Van der Meer & Sigurdarson (2016) may be used.

NOTE See also EurOtop (2018), 6.3.4 where other design material can also be found.

* + 1. Abrasion and crushing of stones

(1) The probability of breaking of the stones may be evaluated by considering two variables:

1. the impact energy;
2. the breaking energy required to break the stone.

NOTE 1 When a berm breakwater reshapes under the wave and/or current action the stones suffer impacts as they roll on the berm. These impacts may eventually lead to abrasion and/or breaking of the stones. This possibility, however, is quite limited in the types of berm breakwaters dealt with in this document, more so for type a) structures.

NOTE 2 Tørum and Krogh (2000) and Tørum et al. (2002) developed a method to evaluate the suitability of stones from a specified quarry from the stone breaking point of view, when the stones roll on a partly reshaping berm breakwater. For this condition a stone will basically move once down the breakwater slope and come to a rest at a lower level. The speed of the stone will vary as it moves along the slope, but it is anticipated that it will be subjected to one major impact if it hits another stone at the maximum velocity.

(2) The velocity and the strength of the stone may be assumed to be independent of one another.

NOTE For further information, see PIANC (2003).

* + 1. Local scour and scour protection

(1) Assessment of local scour should preferably be based on experience on local scour conditions.

(2) If this is lacking, which is typically the case with new projects, then validated semi-empirical formulae or sediment transport theory may be used.

NOTE Type a) berm breakwaters appear to be more protected from scour at their toes due to commonly exposing heavier material there than type b) berm breakwaters. Useful guidelines on scour and scour protection can be found in the Coastal Engineering Manual, Sumer and Fredsøe (2002), and The Rock Manual (2007), 5.2.2.9. See also 7.3.10 and C.3.11.

* 1. Low-crested and submerged mound breakwaters
     1. Failure modes

(1) Failure modes in low-crested (emergent, no capping block) and submerged mound breakwaters may be basically those of conventional mound breakwaters, presented in C.3.1, with the following modifications (see Figure C.1.):

1. mode b1 is quite seldom;
2. modes c, and f are not present;
3. modes h, and j are not present in single-layer structures.

NOTE In these types of mound breakwaters wave overtopping and transmission processes are quite predominant. Hence crest stability becomes of prime importance, especially in submerged structures.

* + 1. Fault tree

(1) A simplified fault tree applicable to low-crested and submerged mound breakwaters may be drawn based on the diagram of Figure C.2, modified as noted in C.5.1.

(2) The failure probability assessment presented in C.3.2 may be applied also to the structures dealt with in Clause C.5.

* + 1. Design approach and formulae

(1) The guidelines on design approach and the applicable manuals given in C.3.3 may be used also for low-crested and submerged mound breakwaters.

NOTE Characteristic and design values of geotechnical parameters and resistances as well as relevant partial factors are indicated in EN 1997 (all parts).

* + 1. Wave action on the seaward rock-armoured slope

(1) For structural response formulae exist for wave action on the seaward rock-armoured slope, the graphs by Kramer & Burcharth (2004) and by Burger (1995) for the evaluation of the stability of low-crested and submerged breakwaters may be used.

(2) For uniform stone structures results by Ahrens (1987) and Van der Meer (1990) may be used for the stability of the single-layer units in relation to the structure’s height.

(3) When that height results from wave and current action on piles of uniform stones the structure may be called dynamically stable.

NOTE Further guidelines can be found in The Rock Manual (2007), 5.2.2.4.

(4) In highly permeable structures the seaward armour is subject to an extra pulsating flow of the waves partially transmitted through the body of the mound, and physical model tests should be used for such cases until formulae credible for practical applications are developed.

NOTE Studies have shown that this wave action is dependent on the wavelength ratio to a length measure of the structure.

* + 1. Wave action on the crest and rear armour slope

(1) The crest should receive extra care in submerged structures, since wave action there can increase due to wave breaking.

(2) A design graph by Burger (1995) may be used for crest and rear side stability, with caution though in the case of high freeboards.

NOTE 1 For more information, see The Rock Manual (2007), 5.2.2.4.

NOTE 2 C.5.4(2) is applicable also to the rear armour.

* + 1. Wave overtopping in low-crested mound breakwaters

(1) Wave overtopping should be assessed for low-crested mound breakwaters.

NOTE For guidance on this topic, see C.3.7.

* + 1. Wave transmission

(1) Calculation of the coefficient of wave transmission may be assessed through formulae based on the DELOS database.

NOTE 1 Wave transmission over and through low-crested and/or submerged mound breakwaters is a complex process:

* In low-crested structures, i.e. of positive crest elevation, transmitted waves are generated by the overtopped water impinging on the sea surface to the lee of the structure plus a wave transmission through structures with sufficient permeability;
* In submerged breakwaters waves are transmitted above and through the structure.

NOTE 2 Waves transmitted above the crest can be the result of a wave breaking process.

(2) Apart from the freeboard (relative to wave height) that has a prime effect on the value of the coefficient of wave transmission, the contribution of crest width should also be assessed.

NOTE Both these effects are captured by the data analysis in (1). For more information, see The Rock Manual (2007), 5.1.1.4 and the EurOtop (2018), 4.2.5.

* 1. Qualitative cumulative damage assessment of mound breakwaters loaded by waves and currents

(1) A qualitative idea about the level of resilience may be obtained with the following procedure:

* incorporation of short-term wave statistics into the long-term ones to build an appropriate suite of sea-states in a realistic way;
* repetitive application of a sound design formula can provide quantitative indication about the total damage at the end of storm-suite, provided no repair or maintenance of the structure takes place in-between the storms.

NOTE 1 This procedure assumes a relation between the damage level and the number of waves present in a given sea-state.

NOTE 2 Regarding the cumulative damage estimation to a breakwater armour layer after several storms, methods propose empirical formulae that can calculate the total damage level at the end of a storm or a sea-state as a function of Hs, the number of waves of the storm/sea-state, and the damage level at the end of the previous storm/sea-state.

NOTE 3 Information on the cumulative damage of a coastal structure after several storms/sea-states can be useful for timely decision making on the maintenance and repair of the structure.

1. (informative)  
     
   Additional guidance for vertical face and composite breakwaters
   1. Use of this annex

(1) This informative annex provides supplementary guidance to Clause 8 for wave and current actions on vertical face breakwaters, in particular to 8.3.2 and 8.3.3 for:

* horizontal impact pressure;
* uplift due to wave action;
* impulsive pressure;
* wave overtopping;
* wave transmission.

(2) This informative annex provides supplementary guidance to Clause 9 for wave and current actions on composite breakwaters, in particular also to 9.3.1, 9.3.2 and 9.3.6 for vertical-composite structures corresponding to:

* critical action on and response of the mound armour;
* wave overtopping;
* wave and current actions on the vertical face toe;

and to 9.4.1 for horizontal-composite breakwaters with respect to:

* wave actions on the mound;
* wave action on the vertical face.

NOTE National choice on the application of this informative annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

* 1. Scope and field of application

(1) This informative annex covers issues related to wave and current actions on vertical face and composite breakwaters, i.e. subjects covered in Clauses 8 and 9. It applies to structure types presented in 8.1 and 9.1, namely breakwaters with vertical face extending down to the seabed, vertical-composite, and horizontal-composite breakwaters with vertical face.

* 1. Vertical face breakwaters
     1. Failure modes

(1) The stability of vertical face breakwaters shall be considered.

NOTE 1 Typical overall stability failure modes for vertical face breakwaters shown in Figure D.1.

NOTE 2 Commonly vertical face breakwaters are made of caissons.

NOTE 3 Mode <m1> occurs seldom.

(2) Failure mode <g> may be regarded as quite frequent in triggering failure of vertical face breakwaters.

(3) Geotechnical-type cause of failures is quite common and should be carefully assessed.

NOTE In the <st> failure mode no foundation layer is shown for clarity. This mode of failure can be triggered by soil liquefaction.

(4) Local failure modes should also be considered.

EXAMPLE An example for such a mode in a caisson breakwater is a localised breakage of front wall (br).

(5) The assumption in all above failure modes is that the structure responses as a monolithic body. This assumption should be fulfilled in all cases.

(6) Additional failure modes associated with 3D effects, such as structure length and alignment effects, should also be considered.

NOTE These failure modes are discussed in CEM VI-5-4b(3).

  

a) Shoreward sliding b) Seaward sliding c) Inshore overturning around heel

Ein Bild, das Logo enthält.

Automatisch generierte Beschreibung  

d) Offshore overturning around heel e) Inshore slip failure to subsoil f) Offshore slip failure to subsoil

  Ein Bild, das Pfeil enthält.

Automatisch generierte Beschreibung

g) Seabed settlement h) Seabed overtopping movement i) Push out base material due to rocking

 

j) Toe and seabed scour, tilt and settlement k) Local front wall breakage

Key

|  |  |
| --- | --- |
| 1 | rock |
| 2 | crushing |
| 3 | slip surface |
| 4 | push out |
| 5 | scour |

Figure D.1 — Overall stability failure modes for a caisson breakwater (PIANC 2003, modified)

* + 1. Fault tree

(1) The failure modes presented in D.3.1 may be combined in a fault tree representation in order to illustrate the failure mechanisms and their interrelationships that can cause damage to the structure; thus, sensible verifications can be performed.

NOTE 1 A fault tree for a caisson breakwater is shown in Figure D.2.

NOTE 2 The fault tree in Figure D.2 contains only OR-gate combinations of the individual failure modes, i.e. only one mode in a sub-system can result in failure. It is quite common in coastal structures to have failure mechanisms in a series (OR-gate) rather than in a parallel (AND-gate) format.



Figure D.2 — Fault tree for a caisson breakwater (PIANC 2016, modified)

* + 1. Design approach and formulae

(1) For conventional vertical face breakwaters, formulae needed to describe the response of the structural elements associated to the failure modes presented in D.3.1 should follow widely accepted technical literature and specifically of the following documents:

* “Coastal Engineering Manual” (2003-2011);
* “EurOtop” (2018);
* “PIANC MarCom WG 196” (2016):
* “PROVERBS” (2001).

NOTE 1 Design approaches for conventional vertical face breakwaters largely depend on the consequence class of the specific structure under consideration and the uncertainty associated to both the hydrodynamic estimate and the responses with associated formulae.

NOTE 2 For guidance on the selection of the design approaches, see Clause 4.

NOTE 3 (3) Geotechnical limit states indicated in EN 1997 reflect the failure modes G, “overturning” and “sliding”.

NOTE 4 Structural limit states indicated in EN 1992 (all parts) reflect failure mode br.

NOTE 5 Hydraulic limit states indicated in Annex D reflect failure modes ov and r.

NOTE 6 Characteristic and design values of concrete parameters and resistances as well as relevant partial factors are indicated in EN 1992 (all parts).

NOTE 7 Characteristic and design values of geotechnical parameters and resistances as well as relevant partial factors are indicated in EN 1997 (all parts).

(2) In the following sections more specific guidelines regarding response formulae are given for the various failure modes mentioned previously. Given that such formulae are continuously updated through research these recommendations should only be taken as indicative and temporary.

* + 1. Nonbreaking waves on vertical wall

(1) For nonbreaking or quasi-breaking waves, the Goda formula (Tanimoto et al. 1976) may be used.

NOTE 1 This formula deals with irregular waves and oblique incidence.

NOTE 2 Guidance on application of the above can be found in CEM VI-5-4b.

(2) Further on, the older formulations by Sainflou (1928) as modified by Miche (1944) and Rundgren (1958) may also be used.

NOTE 1 This latter formula is not to be used for breaking or overtopping waves.

NOTE 2 Application guidelines for the older formulae can be found in the Shore Protection Manual (1984).(3) Sainflou may be used to better estimate negative pressures.

* + 1. Breaking waves on vertical wall

(1) The most promising design formulae to approximately estimate actions due to breaking waves, which may be used, are the following:

1. Kortenhaus & Oumeraci (1998, 1999). For waves with incident significant wave height larger than 0,35*h*s, *h*s water depth in front of the structure, the maximum horizontal impact force *F*h(per unit length) can reach a value (5/8)*ρ*∙*g*∙*H*∙*b*2, *H*b the breaking wave height.

NOTE 1 For further information, see PROVERBS vol. IIb, 5.1.

1. Goda's formula modified by Takahashi et al. (1994). The formula is developed for regular waves and for a range of tested parameters.

NOTE 2 Formula and parameters are given in CEM VI-5-4b.

1. A probabilistic procedure to estimate the maximum wave force on the front face of the breakwater by using a generalized extreme value distribution.

NOTE 3 This procedure is presented in PROVERBS vol. I, 2.5.1.1.

NOTE 4 When waves are forced to break on the vertical face of a breakwater, they exert high impact pressures that are difficult to evaluate. The peak wave action lasts a tiny fraction of a second, e.g. 0,05 sec for plunging waves with no air entrapped. In case of breaking waves with large air pockets even higher two-peaked forces are developed on the wall. Usually, the former are called impact and the latter impulsive forces or pressures. To judge whether a structure experience impact or impulsive forces by breaking waves is not possible, since this depends on uncontrolled parameters, such as curvature of breaking face, exact distance from the wall, blowing wind, etc.

* + 1. Uplift force

(1) Since the uplift hydrodynamic force is directly dependent on the associated pressure distribution on the front face and the permeability of the foundation soil, the references in D.3.4. and D.3.5. should be used for the assessment of the uplift force.

(2) The dependence on the soil permeability may be considered using Goda’s formula for both nonbreaking and breaking waves.

* + 1. Wave overtopping

(1) For estimating wave overtopping rate over a vertical-face breakwater a formula based on experimental data may be used.

NOTE Guidelines on this can be found in EurOtop (2018), 7.3.2.

(2) A distinction should be made between non-breaking wave conditions over an influencing or non-influencing foreshore and impulsive wave action.

(3) A non-influencing foreshore may be regarded one with deep water at the structure toe.

(4) Under impulsive conditions the actual freeboard should be considered to decide which formula is applicable.

NOTE This freeboard threshold equals roughly 1,35*H*m0, where *H*m0 the spectrum based significant wave height.

* + 1. Wave transmission

(1) To obtain an estimate of the wave transmission at vertical face breakwaters the Goda (2000) formula may be used.

NOTE The formula is applicable to freeboards less than 1,25*H*m0.

(2) For higher freeboards the wave overtopping rate may be used:

* to obtain an order of magnitude of the transmitted wave; or
* revert to an appropriate numerical or physical model.

NOTE Application guidelines for this wave transmission can be found in the EurOtop (2018), 4.2.5.

* + 1. Wave reflection

(1) For irregular head-on waves the reflection coefficient may be obtained by the simple formula provided by Allsop et al. in the MAST Programme (1994).

NOTE 1 Experiments with impermeable vertical walls have shown that the reflection coefficient is rather insensitive of wave short-crestedness or wave obliquity.

NOTE 2 Further details on wave reflection by vertical walls can be found in CEM VI-5-2-C(3).

* + 1. Stability of seabed against geotechnical failure

(1) The bearing capacity of the seabed foundation shall be calculated in accordance with EN 1997 (all parts).

NOTE The seabed foundation of a vertical face breakwater can experience geotechnical failure by the eccentric and inclined loading from the main body of the breakwater under wave action.

(2) The strength parameters of rubble stones composing the breakwater foundation shall be estimated in accordance with EN 1997 (all parts).

* + 1. Local scour and scour protection

(1) Assessment of local scour due to waves and currents should preferably be based on experience on local scour.

(2) If such experience is lacking, then validated semi-empirical formulae or sediment transport theory may be used.

NOTE To this effect useful guidelines on scour and scour protection can be found in CEM (2002), Sumer and Fredsøe (2002), OCDI (2002).

(3) The ambient current velocity close to the bed should be modified to account for the presence of waves.

NOTE 1 This can be achieved by considering the interaction between waves and current.

NOTE 2 A simplified procedure for such interaction is proposed in Soulsby (1997).

* 1. Vertical-composite breakwaters
     1. Failure modes

(1) The most significant failure modes in a vertical-face breakwater on a rubble mound foundation may be considered to follow in general the modes presented in Figure D.1 with the following modifications:

1. modes l, l1, i.e. slip failures in subsoil and associated settlements, are replaced by the corresponding ones lm and lm1, i.e. slip failure in the rubble foundation (including possibly subsoil) and associated settlement inducing tilt inshore and offshore, respectively;
2. scour in seabed, mode g, is much more infrequent as compared to scour in rubble mound crest, mode c, and may be replaced by the latter for ease of use.

(2) The above three failure modes may be applied to this type of vertical breakwaters are shown in Figure D.3.



Figure D.3 — Additional failure modes for vertical-face breakwaters on rubble mound foundations

* + 1. Fault tree

(1) The fault tree for vertical breakwaters sitting on the seabed given in Figure D.2 may in principle be applied to vertical-face breakwaters sitting on rubble mound foundation with the following modifications in the graph:

1. failure modes l, l1 are replaced by lm, lm1 that refer to the slip failures associated mainly with the foundation mound to the inshore, offshore, respectively;
2. failure mode g is replaced by mode c that refers to the scour in rubble mound crest and the subsequent tilting of the structure.

NOTE For guidance on the combination of failure mechanisms, see C.3.2.

* + 1. Design approach and formulae

(1) Provisions in C.3.3 is also applicable to vertical-composite breakwaters with offshore plane vertical face and should be followed in respect to the mound part of the structure.

(2) Provisions in D.3.3 may be followed for the vertical face part of the structure.

(3) The provisions in D.3.3(3), D.3.3(4) and D.3.3(5) shall be applied.

* + 1. Wave actions on the vertical face

(1) Since impact loads on that face should be avoided, extra care should be taken to achieve nonbreaking conditions.

NOTE The wave actions on the vertical offshore face of a vertical-composite breakwater are dependent on the water depth at the vertical face relative to the incoming wave height.

(2) Considering the irregular nature of wave trains the above-mentioned depth should be at least 2*H*1/3 or the maximum wave height allowed by the water depth at the base toe.

(3) In cases where this cannot be met due to depth limitations the structure should be designed with a front wall to the seabed.

NOTE See Clause 8 and Clause D.3.

(4) The alternative may be to design a rubble mound foundation such to provoke wave breaking upfront of the vertical face.

NOTE 1 In this approach the structure’ s front can experience breaking, slightly breaking, or broken waves depending on the berm width in front of the structure.

NOTE 2 Estimation of that width can be found in CEM VI-5-4a(7). However, it should be appreciated that such an estimation refers only to the assumed design conditions represented by *H*1/3 and the associated definition of the surf zone. Hence if the structure lies outside that zone but inside an adjacent zone where waves higher than *H*1/3 should break due to depth limitation, it can experience breaking waves on its face and not on the berm if its width is calculated through *H*1/3.

NOTE 3 A caution relates to designing a narrow berm in order to achieve slightly breaking instead of impact waves. By narrowing the berm eventual failure of its offshore corner, a frequent possibility, can impair the stability of the caisson structure by weakening the foundation strength and inducing eventually offshore tilt.

(5) Calculation of the wave forces upon the vertical front should be done by the methods given in D.3.4 for non-breaking waves and D.3.5 a) and b) for breaking waves.

NOTE The classification of wave actions refers typically to the design conditions representing one or two sea states. However, in real life a coastal structure can experience both breaking and non-breaking waves, depending on its location with respect to the surf zone and on its berm width relatively to any wave height attacking the structure.

* + 1. Wave and current actions on the mound armour

(1) The wave action on the armour layer of the foundation mound should be estimated in the framework of a submerged rubble mound, commonly of a low permeability.

NOTE 1 An approximate method to describe the interaction between wave action and response of the armour units on the mound slope is to follow the respective guidelines for submerged mound breakwaters C.5.4.

NOTE 2 Regarding wave action on the front berm of the mound it is noted that due to non-overtopped but rather reflected wave energy by the vertical front, the wave action at this part of the structure can easily overrun the corresponding action on the mound slope. Similar consideration applies to the currents of any origin present in the area, see also D.4.6.

(2) The formula to quantify the response of the berm armour to wave action in the MAST II project (Madrigal and Valdés 1995) may be used.

(3) When outside the range of validity of its various parameters the results of tests by Tanimoto et al. (1982) and Takahashi et al. (1990) may be applied.

NOTE It is common to extend berm stone-size on the mound slope if the latter requires lighter armouring, based on the approximate approach mentioned above.

* + 1. Wave and current actions on the vertical face toe

(1) The protection against this failure mode should be designed jointly with the design of the berm armour, see D.4.5.

NOTE 1 The vertical face toe is an area vulnerable to scour due to the reflection process imposed by that face on waves and currents. Toe scour can impact adversely the stability of the vertical structure.

NOTE 2 Guidance on this can be found in D.3.11 where the corresponding problem at the toe of a vertical front to the seabed is addressed.

* + 1. Wave overtopping and transmission

(1) Wave overtopping for vertical-composite breakwaters should be assessed through the procedure stated in D.3.7 for an insignificantly influencing mound, i.e. with:

* deep water above the mound berm; or
* a depth *d* more than 60 % of the total water depth at the mound’s toe (*h*), assuming the offshore berm width is up to *H*m0.

(2) For a depth *d* less than 60 % of the total water depth at the mound’s toe (*h*) and assuming the offshore berm width is up to *H*m0, wave overtopping can be associated with wave impulsive conditions or not wave impulsive conditions.

NOTE For wave impulsive conditions, see D.3.7 while for not wave impulsive conditions , see EurOtop (2018), 7.3.4.

(3) The waves transmitted or generated in the lee of a vertical-composite breakwater may be estimated by the same procedure given in D.3.8.

* + 1. Wave reflection

(1) Quantification of the reflection coefficient may be achieved by the formula of Tanimoto et al. (1987), see CEM VI-5-2-C(3).

NOTE In general, vertical walls on rubble mounds present a reduced reflection coefficient as compared to reflection by vertical walls extending to the seabed.

(2) The comment in D.3.9 on wave obliquity and wave short crestedness may be applied also to this type of structure, since a major part of the wave reflection process takes place in the upper part of the water column covered in most cases by the vertical face of the breakwater.

* + 1. Stability of foundation and seabed against geotechnical failure

(1) Geotechnical design shall be performed in accordance with EN 1997 (all parts).

* 1. Horizontal-composite breakwaters
     1. Failure modes

(1) The most common failure modes in a horizontal-composite breakwater may be assumed to follow in general the corresponding modes presented in Figure D.1 with the following amendments:

1. include a failure mode pa associated to the erosion of the outer layer of the protective prism;
2. include a failure mode pt associated to the scour of the toe of the protection slope.

NOTE For guidance on the combination type of failure mechanisms, see C.3.2.

* + 1. Fault tree

(1) The fault tree of a horizontal-composite breakwater may be built on the one given in Figure D.2 by adding an independent hydraulic mode pa and an independent geotechnical mode pt.

(2) Mode g in Figure D.2 should refer to seabed scour and the corresponding settlement underneath the protective mound including the vertical face toe, whereas mode pa to the toe scour of the mound.

(3) Compact versions of the fault tree of horizontal-combined breakwaters may be used by considering only the more significant failure modes.

(4) The more significant failure modes in (3) should include the modes s, m, g, pa, st, pt and br.

* + 1. Design approach and formulae

(1) D.3.3 is applicable and should be followed also to horizontal-composite breakwaters with vertical face and sloping protective prism.

(2) The provisions in D.3.3(3), D.3.3(4) and D.3.3(5) shall apply.

* + 1. Wave action on the vertical face and the protective mound

(1) The Goda formula as modified by Takahashi et al. (1990) may be used to calculate the wave action on the vertical face when protected by artificial interlocking units.

NOTE 1 In general terms impact loads on the unprotected face can be reduced by around 80 % and by half of this amount actions due to non-breaking or slightly breaking waves.

NOTE 2 Details on those modifications can be found in CEM Table VI-5-58, while information about the damping effect of artificial armour is presented in PROVERBS vol. I, 2.8.2.

NOTE 3 Results on the vertical face wave action when a less permeable protective structure is used, e.g. as in a conventional rubble mound, are not available, whereas the wave action on the vertical face could be reduced even further.

(2) Results for the required armour and underlayers stability are not available, but to a first approximation the corresponding results for the mound breakwaters may be used, applying the required adjustment on the stability coefficient depending on the permeability of the protective mound.

NOTE Guidelines on the armour and underlayers stability can be found in Clause 7 and Annex C while information on the required adjustment is contained in the Rock Manual (2007), 5.2.2.2.

(3) D.5.3 may be followed to estimating stresses to individual armour units due to wave action.

(4) In cases not covered above, hydraulic model tests should be undertaken.

NOTE For design approach DA4, see Clauses 4 and 12.

1. (informative)  
     
   Additional guidance for coastal embankments
   1. Use of this annex

(1) This informative annex provides supplementary guidance to Clause 10 for wave and current actions on coastal embankments and in particular to the following aspects related to revetments:

* wave action on seaward slope (10.3.2);
* wave action on seaward toe (10.3.3);
* wave overtopping (10.3.4);
* influence of wave action on geotechnical failure (10.3.5);

and also in particular to the following aspects related to seawalls:

* wave reflection (10.4.2);
* wave overtopping (10.4.4);
* wave induced forces on the wall (10.4.5).

NOTE National choice on the application of this informative annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

* 1. Scope and field of application

(1) This informative annex covers issues related to wave and current actions on coastal embankments, i.e. subjects covered in Clause 10 of this document. It applies to structure types and components presented in 10.1, 10.3.1, and 10.4.1, namely to revetments in E.3 and seawalls in E.4. It is understood that in revetments the larger part of the incoming wave energy is dissipated through wave breaking, whereas in seawalls the larger part of that energy is reflected.

* 1. Revetments
     1. Wave and current actions



Figure E.1 — Overview of relevant hydraulic processes, wave actions and failure modes for revetments (based on ISO 21650); TF: transfer function

(1) Figure E.1 shows that the hydraulic processes in the foreshore of the revetment may be transferred to processes describing the loading at the structure and the overflow or wave overtopping rate.

NOTE 1 Coastal revetments are man-made sloped structures parallel to the shore built to armour embankments and protect the coastal land zone against erosion and flooding. Revetments are typically made of concrete slabs, prefabricated armour units, rubble, asphalt or combinations thereof; other materials can also be used. Berms can be installed on revetments. A summary of relevant hydraulic processes and failure modes for coastal revetments is given in Figure E.1.

NOTE 2 Current actions are typically of secondary importance as compared to wave actions and they are not shown in Figure E.1. However, they can contribute to producing failure at the seaward toe and the slope of the structure.

* + 1. Fault tree

(1) The wave and current actions in E.3.1 may be used to describe the failure mechanisms developed at:

* the seaward side;
* the interior of the structure;
* the revetment crest;
* above the structure.

(2) Combination of these failure modes may provide a fault tree for revetments.

NOTE 1 For a more detailed description of the fault tree and the combination of the failure mechanisms, see C.3.2.

NOTE 2 A simplified fault tree for revetments containing only the major failure modes developing under typical conditions is given in Figure E.2.



Figure E.2 — A condensed fault tree for revetments

NOTE 3 Symbols of failure modes have been kept the same with the corresponding modes of mound breakwaters for ease of reference, see C.3.2.

(3) The most critical failure modes are those on the seaward side and those due to wave overtopping. These should be considered as modes capable of directly causing system failure, see C.3.2 and Table H.1.

* + 1. Design approach and formulae

(1) Design approaches for revetments largely depend on the consequence class of the specific structure under consideration and the uncertainty associated to both the hydrodynamic estimate and the responses with associated formulae. Guidance on the selection of the design approaches can be found in Clause 4.

(2) Formulae needed to describe the response of the structure elements associated to the failure modes presented in E.3.2 may be found in the technical literature and specifically in the following documents:

* The Rock Manual (2007);
* Coastal Engineering Manual (2003-2011);
* EurOtop Manual (2018);
* EAK 2002, Empfehlungen für Küsteschutzwerke: “Die Küste”, Korrigierte Ausgabe (2007)

(3) The following document on revetment may also be used:

* Pilarczyk, K.W. “Design of revetments” in Tsinker G.P. (Ed.) Port Engineering, John Wiley & Sons (2004).

(4) In the following subclauses more specific recommendations regarding response formulae are given for the various failure modes mentioned previously. Given that such formulae are continuously updated through research these recommendations should only be taken as indicative and temporary.

* + 1. Wave actions on the seaward slope

(1) Variation of water depth at the toe of the structure, wave breaking, angle of wave approach, steepness of the foreshore should be carefully examined along with other considerations.

NOTE 1 The definition of the environmental conditions, in particular that of the wave field, is of great importance in revetment design.

NOTE 2 For more information, see the Rock Manual (2007), 5.2.2.2

NOTE 3 Run-up on revetment slopes can cause wave overtopping, whereas the process of both run-up and run-down can cause erosion damages on the seaward slope depending on the material used for the armour and underlayer.

(2) For the determination of wave run-up height, the widely used definition of *R*u,2% may be used which is defined to be the height of the run-up tongue above still water level which is exceeded by only 2 % of all waves.

NOTE It can be determined using e.g. the Van der Meer formula (1998).

(3) The influence of a berm or of the angle of wave attack should be considered.

(4) Run-up due to broken waves may be initially calculated using CEM § VI-5-4-b(7).

(5) For shallow foreshores wave breaking on the foreshore can occur which changes the type of the wave spectrum. Under these conditions the spectral mean energy wave period *T*m‑1,0 for the runup calculations should be used.

NOTE Details can be found in the Rock Manual (2007), 5.1.1.2.

(6) Wave run-down may be determined using the EurOtop (2018), 6.2.

(7) The thickness of the water layer on the slope is changing with elevation over the still water level and may be determined using Schüttrumpf (2001).

(8) Run-up and run-down velocities may be determined by methods available for sea-dikes or revetments.

NOTE For more information, see the Rock Manual (2007), 5.1.1.3.

(9) Very little information is currently available for critical velocities which are linked to initiation of erosion on the slope so that hydraulic model tests should be undertaken wherever erosion failure can become critical for the investigated cases.

(10) Wave impact loads on the seaward side of the revetment can cause the revetment to fail. These loads may be estimated using results from Führböter (1994).

NOTE 1 The associated failure mode is denoted by <b> in the fault tree of Figure E.2.

NOTE 2 The material to be used as armour of the seaward slope is dependent on:

* the wave climate;
* the required hydraulic performance;
* visual and access aspects;
* maintenance;
* other functional characteristics.

NOTE 3 The following materials have been used as porous slopes:

* rock armour;
* riprap;
* concrete armour units;
* gabion mattresses;
* open-stone asphalt revetments.

NOTE 4 Various constructions are feasible to be used as non-porous slopes comprising;

* stepped slopes;
* concrete slabs or blocks;
* many others.

(11) The armour layer of the seaward slope may be designed using Hudson’s formula or similar approaches as described in Clause C.3 or in CEM VI–5–3(a).

(12) Hydraulic model tests should be undertaken for special geometries of revetments where these formulae are not directly applicable or whenever the selection of material on the slope is not covered by any of these empirical formulae.

NOTE 1 For a berm above MSL, see Clause C.4.

NOTE 2 For composite slopes, seethe Rock Manual (2007), 5.2.2.8.

(13) Wave induced uplift forces underneath the cover layer are very relevant for armour removal leading to failure of revetments and therefore should be duly considered.

(14) When estimating seaward loading, the sea level should be determined.

NOTE 1 The associated failure mechanism is denoted by <up> in the fault tree of Figure E.2.

NOTE 2 Uplift actions can be estimated using e.g. Bezuijen and Klein-Breteler (1996) as modified by Kortenhaus et al. (2000).

* + 1. Wave and current actions on seaward toe

(1) The stability of the toe should be considered to guarantee the overall stability of the revetment.

NOTE The main purpose of the toe is to prevent undermining of the body of the revetment.

(2) Structural response of the mound toe may be estimated for wave and current actions as proposed in the Rock Manual (2007), 5.2.2.3 and 5.2.3.3 respectively.

NOTE 1 Various types of toe constructions such as sheet-piles, concrete aprons or rubble toes are described together with some design guidelines in Thomas and Hall (1992), CEM VI-5-3(d) and in the Rock Manual (2007), 6.3.4.1.

NOTE 2 Design methods corresponding to failure mode g are given in C.3.6.

* + 1. Wave overtopping

(1) Mean or storm-representative wave overtopping discharges per unit length of the revetment are the main drivers of failure mode <ov> and may be calculated by following EurOtop (2018), 5.3.1, 5.4.2 and 5.4.3, for relatively smooth slope faces, ranging from placed revetment blocks to asphalt.

NOTE Wave overtopping rate is a principal verification criterion for revetments.

(2) In the case of very heavy wave breaking on the foreshore of the revetment the wave spectrum is flattened and the associated modification to the wave overtopping rate may be calculated by formulae presented in EurOtop (2018), 5.3.2.

(3) The effects of oblique wave approach or of a curved revetment on wave overtopping rates may be evaluated by adjustments presented EurOtop (2018), 5.4.4.

(4) The effect of a composite slope or a berm may be estimated through guidance given in EurOtop (2018), 5.4.6.

(5) For coarser faces such as rubble mounds provisions in C.3.7 may be followed.

(6) The effect of currents on wave overtopping should be considered for strong currents, higher than about 0,8 m/s.

NOTE 1 This effect is concentrated mainly in modifying the angle of wave attack.

NOTE 2 For furhter guidance, see EurOtop (2018), 5.4.5.

NOTE 3 Excessive wave overtopping can give shoreward pressures on the revetment, especially when the landward crest of the revetment is permeable.

(7) Wave spray overtopping has not yet been simulated in hydraulic model tests and therefore may be considered by engineering experience, whenever needed.

NOTE 1 Spray overtopping results from wave breaking, where spray is being carried over the revetment. This process can be greatly enhanced by onshore winds, causing hazards, e.g. by reducing visibility to drivers.

NOTE 2 For other overtopping issues use can be made of the EurOtop (2018).

* + 1. Influence of wave action on geotechnical failures

(1) The influence of wave action on geotechnical failures should be considered.

NOTE 1 The design of the underlayer of the slope needs special consideration due to its desired functions such as:

* filtration;
* erosion control:
* drainage;
* energy dissipation.

NOTE 2 Some advice on the design of granular underlayers (for riprap, rock armours, and concrete armour units) and geotextiles is given in the Rock Manual (2007), 6.3.3.6, and the references therein; also in 7.3.8.

NOTE 3 The associated failure mechanism is represented by the mode j of Figure E.2.

NOTE 4 Infiltration in the structure body can result from excessive wave overtopping or wave run-up. It has been shown to be mostly dependent on the mean thickness of the water body on the revetment.

(2) The time required for seepage through the embankment body may be estimated using, e.g. Kortenhaus et al. (2000).

NOTE The phreatic water level in the structure body will influence the geotechnical parameters in the long term and is therefore more relevant for long lasting water levels in front of the revetment.

(3) The slope stability, failure mode e, and the other geotechnical failure modes shall be considered under the provisions of EN 1997.

* + 1. Local scour and scour protection

(1) Assessment of local scour should preferably be based on experience on local scour.

(2) If lacking, then validated semi-empirical formulae or sediment transport models may be used.

NOTE Additional guidelines on scour and scour protection to the literature cited in E.3.3 can be found in Sumer and Fredsøe (2002), OCDI (2009) Clause 3, EAK (2007).

* 1. Seawalls
     1. Failure modes and fault tree

(1) The hydraulic processes in the foreshore and the nearshore of a seawall transferred to processes describing wave and current actions, may be used to describe the failure mechanisms in the interior of the seawall body and at the seawall structure itself.

NOTE Seawalls are onshore structures parallel to the shoreline. In this document they are assumed as vertical or near vertical face structures such as gravity concrete walls, steel or concrete sheet pile walls, stone filled cribwork, etc. The principal function of seawalls is to reinforce a part of the coastal profile and to protect land and infrastructures from the action of waves and flooding, mainly by reflecting wave energy.

(2) Combination of these mechanisms may provide a fault tree for seawalls.

NOTE For a description of the fault tree and the interdependence of the failure modes, see C.3.2.

(3) A simplified fault tree for seawalls may be based on the corresponding fault tree for revetments (Figure E.2) with the following modifications:

1. a further mode gw of wall-toe instability is added to the crest processes of Figure E.2;
2. failure modes c, e, gw are far more pronounced over modes b, g and up in seawalls, reversing thus their relative importance holding in revetments.
   * 1. Design approach and formulae

(1) Formulae needed to describe the response of the structure elements associated to the failure modes presented in E.4.1 may be found in the technical literature and specifically in the following documents:

* The Rock Manual (2007);
* Coastal Engineering Manual (2003-2011);
* EurOtop(2018);
* Die Küste EAK 2002, Korrigierte Ausgabe (2007).

NOTE 1 Design approaches for seawalls largely depend on the consequence class of the specific structure under consideration and the uncertainty associated to both the hydrodynamic estimate and the responses with associated formulae.

NOTE 2 For guidance on the selection of the design approaches, see Clause 4.

(2) In the following clauses more specific guidelines regarding response formulae are given for the various failure modes mentioned previously. Given that such formulae are continuously updated through research they should only be adopted in such temporary context.

* + 1. Wave reflection

(1) Vertical face structures should be avoided whenever a sloping rather than a vertical face structure can provide a satisfactory armouring of the specific coastal embankment.

NOTE 1 Wave reflections from seawalls can have a significant influence on the coastal regime in front of the structure. Vertical or almost vertical constructions can nearly reflect totally the waves increasing the wave height in front of the wall quite significantly, see Allsop and McBride (1996).

NOTE 2 Together with increasing wave heights the sediment transport potential can increase and so does the potential danger of scour.

NOTE 3 Generally, rough and porous walls will result in lower reflection coefficients due to an increased energy dissipation at the structure.

NOTE 4 Wave reflection is dependent on wave steepness and wavelength resulting in longer waves being more reflected than shorter ones.

NOTE 5 An overview of wave reflection formulae for vertical walls is available in the Rock Manual (2007), 5.1.1.5.

* + 1. Wave overtopping

(1) The wave overtopping rate should be considered for seawalls.

(2) Mean or storm-representative wave overtopping discharges per unit length of the seawall are the main drivers of failure mode ov and may be calculated by following EurOtop (2018), 5.3.3.

(3) Design graphs may be found in Goda (2000), 5.1.

(4) For vertical walls under nonbreaking conditions D.3.7 may also be used.

(5) In case of composite seawalls sitting on structures with slopes E.3.6 and D.4.7 may be used.

(6) Generic prediction formulae for different types of seawalls are however not yet available so that hydraulic model tests should be performed in configurations of high uncertainty.

NOTE For vertical walls Mach stem waves can increase wave overtopping at specific angles of wave attack.

(7) Wave spray overtopping has not yet been simulated in hydraulic model tests and therefore should be considered by engineering experience whenever needed.

NOTE 1 Spray overtopping results from wave breaking, where spray is being carried over the seawall. This process can be greatly enhanced by onshore winds, causing hazards, e.g. by reducing visibility to drivers.

NOTE 2 Only limited information for special cases is available.

(8) The influence of oblique waves and other factors on wave overtopping may be calculated as proposed in EurOtop (2018), 5.4.

(9) The wave overtopping related to a composite wall sitting on a sloped face protection may be estimated by reference to E.3.6 and D.4.7.

* + 1. Wave and current loading forces on the wall

(1) The distinction between broken, breaking, and non-breaking waves is essential and shall be considered during the design of seawalls.

NOTE 1 A seawall can be exposed to wave-induced forces acting horizontally and vertically endangering thus its stability.

NOTE 2 Formulae predicting wave breaking are available only for simple seawall constructions and vertical walls with and without berm, see CEM, Oumeraci et al (2001, Ch. 2), Environment Agency (1999).

(2) Wherever the situation is unclear, hydraulic model tests should be performed to determine the wave breaking distinction.

(3) Hydraulic model tests shall also be performed for more complex shapes of seawalls.

(4) When a seawall is sitting on a small mound, wave and current actions on the mound may be estimated by following E.3.4.

(5) Oblique wave attack and its effect on wave-induced forces has only been investigated for vertical walls, for this Oumeraci et al. (2001, Ch. 2) may be consulted.

* + 1. Wave and current on seawall toe

(1) The wall-toe shall prevent undermining the body of the seawall.

NOTE 1 For guidance on the required wall-toe protection, see the Rock Manual (2007), 5.2.2.9.

NOTE 2 For guidelines on the mound toe protection from wave and current actions for seawalls sitting on small mounds, see E.3.5.

* + 1. Local scour and scour protection

(1) Assessment of local scour should preferably be based on experience on local scour.

(2) If lacking, then validated semi-empirical formulae or sediment transport models may be used.

NOTE Additional guidelines on scour and scour protection to the literature cited in E.3.3 can be found in Sumer and Fredsøe (2002), OCDI (2009) Clause 3, EAK (2007).

(2) Scour effects along the seawall may be enhanced due to increased reflection (compare with vertical-composite breakwaters).

NOTE These can undermine the wall toe and put at risk the safety of the structure, see the Rock Manual (2007), 5.2.2.9.

1. (informative)  
     
   Additional guidance related to floating structures
   1. Use of this annex

(1) This informative annex provides supplementary guidance to Clause 11 for wave and current actions on floating structures.

NOTE National choice on the application of this informative annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

* 1. Scope and field of application

(1) This informative annex includes informative guidance supplementing Clause 11.

* 1. Further guidance related to floating structures

(1) Further guidance related to floating structures are found in references to literature and technical documents.

NOTE 1 For more guidance on floating structures related to oil & gas or energy production, see American petroleum Institute (2005), Bureau Veritas (2012), DNV GL (2019), DNV GL (2018), ISO 19901-7.

NOTE 2 For more guidance on assessment of current actions, see DNV GL (2019), DNV GL (2018), Overseas Coastal Area Development Institute of Japan (OCDI) (2009) and American petroleum Institute (2005).

NOTE 3 For more guidance related to the use of the Morison Formula to assess waves and current actions on slender structures, see DNV RP-C205.

NOTE 4 For more guidance related to the assessment of mean drift for some floating structure geometries, see American petroleum Institute (2005).

NOTE 5 For more guidance related to very large floating structures mooring behaviour, see Publications of 16th International Ship and offshore Structures.

NOTE 6 For further guidance, see also Ancres et lignes d’ancrage (1987), British Standards (1989), Chakrabarti (1987), Faltinsen (1990), ISO 19901-7, Molin Bernard (2002).

1. (informative)  
     
   Additional guidance related to physical modelling of coastal structures
   1. Use of this annex

(1) This informative annex provides supplementary guidance to Clause 12 on physical modelling of coastal structures. It also discusses main strengths and weaknesses of physical models.

NOTE National choice on the application of this informative znnex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

* 1. Scope and field of application

(1) This informative annex includes informative guidance supplementing Clause 12.

* 1. Strengths and limitations of coastal physical models

(1) When using coastal physical models, the strengths and limitations of the models should be taken into account.

NOTE 1 Three main attributes are highlighted by Kamphuis (2010), see Note 2 – Note 4.

NOTE 2 Qualitatively, physical models are close simulations of the prototype because they are normally carried out with water subjected to gravity forces as in nature. They naturally integrate governing physical processes without simplifying assumptions made in analytical or numerical models.

NOTE 3 Viewing a physical model in operation adds to the physical understanding of the problem. For example, large circulations, refraction and diffraction are more obvious in the model because of its reduced scale. In addition, it enhances physical intuition to improve the design of structures.

NOTE 4 Physical models can be used even if not all the details of the relevant processes are clearly understood or not well described by theory. Multi-phase phenomena involving combination of air, water, rock and sediment can all be studied in physical models even though we do not know the equations that govern the interactions. Design of coastal structures in the surf zone is a good example of such combination.

(2) To minimize scale effects, the largest possible scale should be used.

NOTE 1 As a consequence, physical models cannot simulate large-scale processes (*O* > 1 km2)

NOTE 2 Scale effects are the main limitations of physical models.

NOTE 3 Detailed discussion of similitude laws and scaling could be found in classical textbooks of Hughes (1993) and Kamphuis (2010). It is never possible to achieve complete similarity between model and prototype because some quantities cannot be scaled down. By using water, gravity, fluid viscosity and density are the same in model and prototype, and gravity-driven and viscosity-related phenomena cannot be simulated simultaneously.

(3) Mechanical devices generating and absorbing waves and currents produce only simplified boundary conditions generating laboratory effect which should be minimized as much as possible.

(4) Composite modelling which combines physical and numerical models in different ways to improve the reliability of the results, may be used.

NOTE 1 Composite modelling is an extension of hybrid modelling (Hughes, 1993).

NOTE 2 In most cases, the boundary conditions for a physical model comes from a regional numerical model. These models are run in sequence from large scale to small-scale. A step further was to establish a two-ways flow of information between the physical model and the numerical in order to generate and absorb properly flows at the boundaries of the physical model (Barthel and Funke, 1989).

NOTE 3 More recently, composite modelling has been explored within the Hydralab European research project and reported in Frostick et al. (2013, clause 6). It is defined as the integrated and balanced use of physical and numerical models and based on an inclusive approach including also field experiments and theoretical analysis.

NOTE 4 It combines the best out of both physical and numerical models for a given problem providing more quality results at the same cost or increasing capabilities to model more complex problems which individual physical or numerical models cannot.

* 1. Uncertainty in measurement

(1) The evaluation of measurement data shall follow the Guide to the expression of Uncertainty in Measurement (GUM) and related documents published by the Joint Committee for Guides in Metrology (JCGM, 2009).

* 1. Viscosity forces in internal flows of rubble-mound structures

(1) At protype scale, internal flows in the underlayer and core of rubble-mound structures are partially to fully turbulent whereas they are laminar at model scale leading to lower permeability and overestimated head losses (Hughes, 1993, 5.2.2), and a correction of the grading of the core should be performed to minimize this scale effect.

NOTE The basic principle is to equate the hydraulic gradients in model and prototype modelled with the extended Forchheimer equation including current and wave contributions. A recent development of this approach has been published by Vanneste and Troch (2012, 2013).

* 1. Viscosity forces on floating structures

(1) Since scaling of viscosity forces are function of a Reynolds number, the Reynolds number may be defined as in Formula (G.1):

(G.1)

where

|  |  |
| --- | --- |
| *L* | is *h* at shallow and intermediate depths or *L*=*L*o/2 at undefined depths |
| *h* | is the depth at the model reference zone and *Lo* is the wavelength in deep waters; |
| *V*c | is group waves celerity. |

NOTE For more information, see Sutherland and Evers (2013).

(2) To make sure that the forces of viscosity are sufficiently representative, it shall be checked that the model Reynolds number reaches values of several hundred in deep water and thousands or greater than 104 in shallow water.

* 1. Impulsive wave load with air effects

(1) Overestimation of impulsive wave loads at model scale when air compressibility effects are generated due to air entrapment, leakage or entrainment should be considered.

NOTE 1 For more information, see Bullock et al. (2001).

NOTE 2 It is understandable by the fact that when a model is scaled with Froude similitude, a distortion is created with Cauchy similitude. Correction methods have been published by Cuomo et al. (2010) and Bougis et al. (2016) to properly scale such impulsive impacts.

1. (informative)  
     
   Wave and current actions in reliability analysis
   1. Use of this annex

(1) This informative annex provides supplementary guidance on the application of the reliability-based design (DA2) dealt with in Clause 13. As such it should be used in conjunction with Clause 13, as well as with Clauses 4, 5, and 12 and their annexes thereof.

NOTE National choice on the application of this informative annex is given in the National Annex. If the National Annex contains no information on the application of this informative annex, it can be used.

* 1. Scope and field of application

(1) This informative annex covers issues related to the reliability analysis of coastal structures built in the marine part of the coastal zone, i.e. subjects covered in Clause 13.

NOTE The basic background material on reliability analysis applicable to all structures dealt with in this ensemble of Eurocodes can be found in EN 1990:2023, Annex C and is not repeated here. Nevertheless, additional information relevant specifically to coastal structures is included herein.

* 1. Calculation of the probability of failure of an element
     1. General guidelines

(1) The probability of failure of an element, i.e. associated to an individual failure mode, should be calculated by a well-established method or approximations thereto.

EXAMPLE Examples of probabilistic methods appropriate for the full application of DA2 are the Direct Integration Method (DIM) and the Monte Carlo Method (MCM).

NOTE DIM requires more resources than MCM for its application.

(2) The assumptions used should be duly stated.

(3) For the full application of the reliability analysis the probabilistic distributions of all resistance and load variables should be known, as well as the correlation level between them, if any.

NOTE The probability distributions of the load variables along with their correlations are provided by the relevant Hydrodynamic Estimate Approach (see Clauses 4 and 5). That information comes normally in the form of joint probability density function between the relevant core load variables.

(4) In a full application of DA2, calculation of the probability of failure shall be implemented for all limit states prescribed in Clause 13.

* + 1. The direct integration method

(1) The core element in this method of calculation of the probability of failure *P*f should be applied the conditional probability model for the hydrodynamic loads in order to capture the correlation between variables.

(2) The said model may be illustrated by the following probability law applied to three core wave action variables, see Formula (H.3):

(H.3)

where

|  |  |
| --- | --- |
| *H*s | is the long-term significant wave height; |
| *T*m | is the long-term mean wave period; |
| *θ*m | is the long-term mean wave direction. |

(3) Application of the above law may yield for a single arbitrary sea state and for a given realisation of the vector of time invariant parameters ***x*** (including model uncertainties, geometrical parameters, resistance parameters) and short-term time variant parameters **y** (including water movement, wave forces, etc., see 13.2), as given in Formula (H.4):

(H.4)

where

|  |  |
| --- | --- |
| min g(x,y; t) | indicates the minimum value of the limit state function over the duration of the sea state under consideration; the time dependency of g(..) follows from the fact that the water movement within a sea state is conceived as a random process; |
| *SL* | is a variable that denotes the sea-level modification due to storm-surge effects including wind set-up; |
| *C* | is a variable that denotes the current velocity due to storm-surge. |

NOTE Approximation (H.2) can also be formulated to calculate the annual failure probability by taking the marginal distribution of the annual maximum of the dominating variable and conditional distributions for all athers. In many cases *H*s will be the dominating variable.

(4) It is evident from Formula (H.2) that a wide range of data is required. In some cases, the calculations may be simplified by considering:

1. The effects of wind waves and storm surges to be independent;
2. The variables *SL* and *C* act independently of one another.
   * 1. The Monte Carlo method

(1) Since the Monte Carlo method is a method based on a large number of *N* simulations, a part of which *N*f lead to structure’s failure, it may be assumed that provided *N* is a high enough number, see Formula (H.5):

(H.5)

NOTE Each simulation starts by drawing a random number from a uniform probability density function between zero and unity. Through this number a value (*x)* of each variable (*X)* can be determined by the latter’s inverse cumulative distribution function .

(2) *N* should be sufficiently large for *P*f to attain acceptable convergence.

(3) The cumulative distribution function *F*X(*x*) may be calculated from the probability density function *f*X(*x*) as follows in Formula (H.6):

(H.6)

NOTE 1 The statistical correlation between two variables *Xi*, *Xj* can be obtained by considering the conditional distribution .

NOTE 2 *P*f is associated to the time period that underlies .

* + 1. An easy-to-use method

(1) An artificial sample of environmental data may be generated based on the advanced fully probabilistic methods direct integration method and Monte Carlo method.

EXAMPLE The artificial sample of environmental data can contain data of at the structure’s location, e.g.:

* significant wave height *H*s;
* mean wave period *T*m;
* mean wave direction *θ*m;
* sea level *SL*.

(2) The artificial sample of environmental data should be so selected to validly represent the full dataset covering a certain time period.

(3) The method can be easily applied when wave data in deep waters are available, but in this case a wave propagation model should be applied.

EXAMPLE A fast and adequately accurate model via integration of short- and long-term wave statistics, to transfer the probabilistic information to the structure’s location.

(4) The steps of the methodology applied at correlated environmental parameters in deeper waters than the structure’s location may be the following.

1. Application of a relatively high threshold (e.g. corresponding to 95 % quantile of *H*s) to *H*s data, which are available in deep waters, to filter the most significant sea states that will be transferred to the structure’s location.

NOTE In this way, not only the most critical sea states are distinguished from the total sample, but also the amount of data to be transferred to the structure location has been significantly reduced.

1. Grouping of the filtered (over threshold) data into joint classes of adequately small bin size containing all correlated parameters and estimation of the frequency of each joint class. In the case of a couple of correlated environmental variables, e.g. *H*s and *T*m, an approximation to their joint probability is to proceed with only the most critical parameter (*H*s) and relate to its design value the most probable value of the other variable, based on their corresponding scatter diagram (*H*s - *T*m).
2. Selection of the reduced sum of the filtered data of the new dataset to optimise computational demands and attain further data reduction. The size of the new dataset can range at about 15 % of the filtered dataset but not less than 5 % to avoid elimination of rare and extreme events in the reduced sample.
3. Calculation of the new frequencies of the joint classes of the new dataset using proportionate stratification to the size of the new dataset.
4. To obtain the new dataset, the reduced sample can be produced by randomly generating an integer number of groups of parameters between the limits of each joint class, matching the new frequency of each class calculated previously.
5. Estimation of the sample’s parameters at the structure’s location by propagating the new dataset (step d)) from deep waters to the structure’s location.
6. Application of the conditional model of the above correlated parameters to the sample of the previous step in order to estimate their joint pdf at the structure’s location.
7. Calculation of the new frequencies of each joint class, by rounding to the nearest integer the product of the joint probability of each class (step f)) by an adequately large number *N*.
8. To obtain the artificial data set of parameters at the structure’s location, a sample of *N* size can be provided by randomly generating an integer number of groups of parameters within the range of each joint class, equal to the new frequency of each joint class (step g)).

(5) The easy-to-use method may be used in order that the procedure of transformation of the probabilistic information of the long-term sea conditions, from deep waters to the structure’s location, be adjusted to the available computational capacities.

NOTE The selection of the size of the reduced sample depends on the size of the filtered data, which is related to both the time period that the initial data covers and the time step between the sequential initial data.

(6) If the available data in deep waters covers only a period of about 1 decade, only step 4 a) may be applied and all filtered data be transferred to the structure’s location.

(7) Similarly to Monte Carlo method, *P*f is also estimated here by Formula (H.3), where *N* should be sufficiently large for *P*f to attain acceptable convergence.

NOTE A comparative analysis of this method’s results with those by Monte Carlo method and direct integrating method shows acceptable agreement for engineering applications.

* 1. Failure probability analysis of failure mode systems

(1) To check if the reliability of the system meets the target reliability, which is a design requirement, the failure probability of the system should be calculated.

NOTE 1 The target reliability of a marine coastal structure refers to the allowable failure probability of the system.

NOTE 2 The failure probability of a system depends on the failure probability of its individual failure modes (including post failure behaviour) and on their correlations and dependency. A fault tree is often constructed that describes the aforementioned relations. A system can split into two types of fundamental systems depending on the fault tree, namely series sub-systems and parallel sub-systems.

NOTE 3 In series systems/subsystems, failure occurs if any of the elements fails then the sub-system failure probability is equal to the probability of the union of the elements failure.

NOTE 4 In parallel sub-systems, failure occurs only if all the elements fail, thus the sub-system failure probability is equal to the probability of the intersection of the elements failure.

(2) Since the union or the intersection of the single failure modes depends on their correlation, the latter should be taken into consideration during the calculation of the failure probability of the system.

* 1. Correlation of failure modes

(1) When the correlation between individual failure modes lies between full correlation and no-correlation, the correlation should be quantified, and the result should be put into the relevant probability formula.

NOTE Quantification of correlation is usually performed through the probability associated to the intersection, i.e. the common domain, of the individual failure modes and via the identification of the causal relationship between them.

(2) To accommodate this kind of correlation between individual failure modes, all failure mechanisms should be checked to assess whether they lead to failure under the same actions.

(3) The failure of the system should then be checked by a procedure dependent on its type.

NOTE In this way the individual failure modes dependence on common parameters can be considered and the system probability of failure calculated.

EXAMPLE An illustrative example of this procedure based on Monte Carlo simulation is presented in Table H.1.

Table H.1 — Estimation of system’s failure frequency via consideration of the statistical correlation of individual failure modesa

|  |  |  |  |  |
| --- | --- | --- | --- | --- |
| Sea state simulation number | Seaside armour failure | Toe failure | Rear side  armour failure | System failure |
| 1 | 1 | 0 | 1 | 1 |
| 2 | 0 | 0 | 0 | 0 |
| 3 | 0 | 1 | 1 | 1 |
| **.** | **.** | **.** | **.** | **.** |
| **.** | **.** | **.** | **.** | **.** |
| **.** | **.** | **.** | **.** | **.** |
| *N* | 1 | 0 | 0 | 1 |
| Total | *N*s | *N*t | *N*r | *N*f |
| a Numbers 0 or 1 are only for illustration purposes. | | | | |

* 1. Outline example application of reliability analysis of a coastal structure

(1) The methodology steps a) – e) described below may be used to formulate a reliability analysis of a marine coastal structure under wave action.

EXAMPLE E.g. a rubble mound breakwater considered as a system, given its fault tree (Annex C).

1. Split the fault tree into parallel and series subsystems that consist of individual modes of failure.
2. For each subsystem, choose a suitable limit state functions (design formula) for every mode of failure of the sub-system to examine its reliability and formulate the reliability function of each mode, e.g. the sea-side armour stability.
3. Collect input data of the load and resistance parameters that affect the reliability function of each element. Referring to the armour stability, input data on the long-term wave climate should be collected at the site. Such data will include variables *H*s, *T*m, *θ*m, or similar. This can be implemented by:
4. measurements at the structure’s location for a period of, say, one year; or
5. measurements of the same type but in deep water if the location of the structure is in shallower water; or
6. hindcasting methods for a period of several years (Coastal Engineering Manual, 2006). Since a required input to these methods is wind data, it is advised to extend the time period applied to match the available wind records length;
7. Applying the climate change component on the results of Step a). This change can relate to wind direction or/and intensity modifications;
8. Regarding case c) 1) ii), the steps a) to f) of the easy-for-application method (H.5.4) may be applied in order that the joint pdf of *H*s, *T*m, *θ*m, be estimated at the structure’s location. This procedure is useful when there is a need to optimise computational demands during the long-term statistics transformation from deep waters to the structure’s location. Either linear or non-linear models may be used, the latter being superior to the former in that they account for wave interactions that can have a considerable share in the final outcome –of the order of 10 % or more- depending on water depth; however, these latter models are quite demanding in terms of computing resources. The variables sought after during this process are *H*s, *T*m, *θ*m, and the final result will be their joint probability density at the structure’s location.
9. Applying a fully probabilistic method (see H.3.2 - H.3.3) to estimate the failure probability of each specific element.
10. assess the sub-system’s reliability by estimating the sub-system probability of failure. The latter can be calculated by checking whether the elements of a subsystem fail under the same environmental conditions. Thus, the statistical correlation of the individual modes of failure can be taken into consideration, and the sub-system failure probability can be extracted;

the total system’s probability of failure can then be estimated by considering the failure probability of all sub-systems and their linking to the total system.

* 1. Qualitative resilience assessment of coastal structures

(1) Until credible formulae are established capable to assess the resilience of specific structures, the recommended way to cover this lack of information should be through physical model testing.

NOTE 1 Approach DA4 can also capture correlations between failure mechanisms of the physical interaction type that are important in assessing a coastal structure’s resilience.

NOTE 2 A qualitative resilience assessment of armoured mounds is outlined in C.5.

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None

**References contained in permissions (i.e. “may” clauses)**

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